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EARHTDAM CONSTRUCTION PRACTICE IN TANZANIA

by

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ABSTRACT

In this report the nature and types of failures of earth-dams has been reviewed. Examples of such failures in Tanzania are indicated. Construction methods as per current practice are discussed. The properties of soils in general and tropical soils in particular are presented. The hydrological aspects of damsites are briefly discussed and some methods of analysis discussed. Finally it is recommended that the deficiencies which have been pointed out in the various dam failures should be corrected so that these structures can serve the purpose for which they are meant.

1. INTRODUCTION

The construction of earthdams has been in practice in many parts of the world for a long time. Prior to the understanding of the mechanics of soils, these structures were mainly constructed using the rule of thumb, and often without compaction. With the introduction of soil mechanics in the analysis of earth embankments a rational approach to the design, and construction has evolved. This has greatly reduced the number of earthdam failures. However, failures continue to occur mainly due to poor design, poor construction control or inadequate hydrometeorological data. Almost all dam failures which have occured in Tanzania in recent times have been caused by one or a combination of the above. Unfortunately no proper records exist of the investigations carried out after the failures. The common measures have been to reconstruct the failed structure without giving consideration to the cause of the failure. For example, the Wiyenzele Dam in Dodoma region failed after reconstruction, possibly because the recommendations for reconstruction were not followed.

The failure of a dam is in most cases accompanied by destruction of lives and property, besides the termination of the service for which it was intended. The investment is wasted and reconstruction may cost much more than the original cost.

As the dams under consideration are mainly the small to medium size type meant for water supply and irrigation, there is a tendency to ignore the necessity for proper investigations. The fact that failures of these structures have led to some areas being cut off from other areas due to washed-away bridges should make the need for proper investigations necessary. The design should also follow guidelines outlined in the design manual, which has just been approved for publication.

The construction phase should also follow the assumptions adopted in the design, and if possible the design engineer should follow-up the construction as well. At present there doesn't seem to be enough contacts between the design section and construction section. The soil technicians who are seconded to the projects from the soil laboratory are sometimes ignored and their opinion is not followed.

Most earthdam projects were initiated in Tanzania after the Second World War. The main purpose then was to create sources of water for villagers and their livestock, and to reduce pastoralism. This was not very successful as it created areas with very large concentrations of livestock. After the villagization programme of 1973 an urgent need was felt to provide a dependable source of water for people and their livestock. This was necessary especially in regions like Dodoma, Arusha, Tabora, Shinyanga, Mwanza and Mara which have big numbers of livestock.

In 1975-1976, five national earthmoving teams were established with capacity to construct small to medium size dams. In addition to the national teams, eighteen small earthmoving teams were established in eight regions to construct dams and charcos on a self-help basis. Besides these teams donor agencies established teams in Arusha and Dodoma regions for the same purpose. The output from these teams leaves much to be desired, but as it is outside the present discussion, it shall not be touched. However, it is important to mention that only dams constructed by the national teams get the services of the soil mechanics laboratory. As such many of the dams continue to be constructed, using the rule of thumb. The establishment of zonal laboratories which has started, together with the availability of staff and laboratory and field equipment may help alleviate the present situation.

After constructing the dam, it should be used and maintained. Many dams have been left unused due to the failure of a proposed irrigation project to take-off. Cattle troughs are not constructed, slow sand filters are not constructed in time and this leads to watering of livestock in the reservoir itself. The fact that domestic water has to also be drawn from the reservoir, makes it, a potential health risk. There is a need to carry out inspection and repair to the dam and spillway when required. Unfortunately there seems to be a problem of maintenance in most waterworks. Frequent inspection should be carried out to assess the status of the works. Financial allocations should be made to enable repairs to be carried out promptly. Cracks should be filled in and unwanted vegetation both on the embankment and in the spillway should be cleared. As these problems are cumulative, if left unattended, they may lead to the eventual failure of the dam.

2. CAUSES OF EARTHDAM FAILURES

2.1 General

The main causes of earthdam failures and damages may be summarized as due to poor quality of construction materials, or poor workmanship of dam design and construction, or inadequate hydrometeorological and/or geological data of dam site, or a combination of the above. The material selected for building the embankment has to confirm to certain predetermined specifications, otherwise assumptions used in the design may not be applicable. For example, the material selected for use as core material must be of low permeability, whereas that for the shell has to be of a higher permeability.

The construction process must also meet the requirements, otherwise a good design might not give the required quality. Hydrometeorological data may be a problem as this is scarce, but the designer should use all means at his disposal to assess the necessary parameters. Geological data is rarely collected in the case of small dams, and natural phenomena such as earthquakes are difficult to predict.

Earthdam failure may be due to: overtopping, piping, sloughing, sliding and so on. These are described in the following sections.

2.2 Failure Due to Overtopping

The flow of water over the crest of a dam is termed as overtopping. When this occurs over an earthfill dam failure is imminent. This occurs during great floods when the spilling capacity is inadequate or when it is not functioning properly. Overtopping can also be caused by an unforeseen intensity of wind or wave action.

When an earthfill dam is overtopped due to one of the above causes a breach will form and grow gradually under the erosive action of the waters. There is, however, very little knowledge about the inception, shape and development of the breach /20/. To properly describe the gradual breaching, the geometry of the breach must be related to the hydraulics of the flow and bed material properties. Johnson and Illes state that the geometry is directly related to the duration and shape of the overtopping flood wave.

The typical breach is between 1 and 3 times as wide as it is deep /20/. A mathematical model, the overall objective of which is to achieve a physical realistic simulation of the gradual failure of an earth embankment caused by an overtopping flood event, has been formulated, developed and tested with real life data by V.M. Ponce and A.J. Tsivoglon. They have concluded that a significant feature of the model is its ability to account for the growth of the breach and the eventual draining of the reservoir behind the embankment /20/.

Failures due to overtopping are not deficiencies in the design of the earthdam itself but rather the result of inadequate hydraulic design. This may be due to inadequate hydrometeorological data.

2.3 Failure Due to Piping

Embankment and foundation piping has caused the next biggest number of failures after overtopping /22/. Piping is caused by improper design or poor construction or both. Strict requirements for uniformly compacted embankments with emphasis on control of construction water and density have been developed tp provide dense and homogenous cores which reduce the incidense of concentrated leaks and resist piping when leaks do develop. Because of such requirements and the introduction of graded filters in the downstream portions of dams, there have been extremely few piping failures in important modern dams /22/.

The damage by piping failure leaves no trace at the embankment or foundation in which the piping developed. To reconstruct the causes and mechanics of failure, construction and design records and eyewitness accounts are required.

2.3.1 Mechanics of Piping

The mechanics of piping is explained by the effect of hydrodynamic force of seeping water upon the stability of soil. Depending upon the direction of flow of water through the soil, the hydrodynamic force can alter the unit weight of soil considerably.

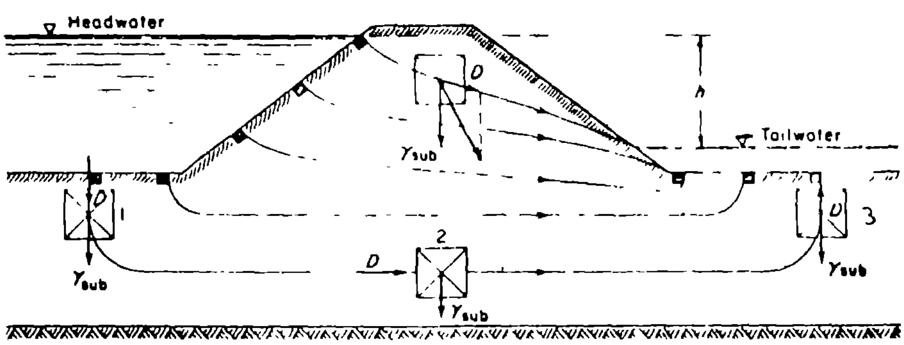

Figure 1. Hydrodynamic pressure conditions of seeping water in soil.

The accompanying figure illustrates this:

D = hydrodynamic force (tangent to the flowline)
= γ_{wi} (force/unit volume)
i = hydraulic gradient
γ_w = unit weight of water
γ_{sub} = submerged unit weight of soil
γ_{eff} = effective unit weight of soil

At point No. 1, the effective unit weight of the soil is

$$\gamma_{eff} = \gamma_{sub} + D$$

indicating densification of the soil.

At point No. 2, the two vectors D and γ_{sub} act perpendicularly to each other. The effective unit weight is

$$\gamma_{eff} = \gamma_{sub}$$

At point No. 3, the hydrohynamic force acts vertically upwards against the submerged unit weight of the soil. Thus,

$$\gamma_{eff} = \gamma_{sub} - D$$

Here when D = $_{sub}$, the soil appears to be weightless and instability of the soil mass is impending. At this point the soil is said to be in an incipient state of fluidization. Such a case occurs when the critical hydraulic gradient, i_c, is attained and consequently, a critical velocity, v_c; then

$$D = \gamma_w i_c$$

When the flow velocity, v, exceeds the critical velocity, v_c, then $D > \gamma_{sub}$, and $\gamma_{eff} < 0$.

This means that the soil particles are loosened, buoyed and lifted up, resulting in a "boiling" or "quick condition".

This phenomenon is associated with the concentration of seepage quantity which is true with non-uniform seepage media. After "boiling" has started, erosion would progress backwards along the flowline until a "pipe" would be formed to the reservoir storage and subsequent failure of the dam.

This action can be slow and accumulative and the resulting failure can be a sudden upheaval.

Some engineers refer to this type of piping failure as "failure by heave". Others describe it as "blowouts". In the case of foundation piping, if the foundation soil is non-uniform, the fine material may be carried away, leaving the coarse material behind, thus tending to produce a reservoir filter which will prevent further piping. However, it is difficult to determine whether piping will result in failure, or will produce an eventual stabilization in any specific case /6/.

Piping can develop inside a zoned dam where the seeping water discharges from the finer material of the core into the adjoining previous zone by migration of soil particles.

2.3.2 Leaks

The size and development of leaks is very much variable and enough knowledge has not been acquired to predict when a leak starts and whether it causes failure or not. Some leaks flow clear for years without any harm while others cause a rapidly progressing piping failure. The following origins of leaks have led to piping failures:

- loose embankment soil
  (Poor construction control results in inadequate compaction in the embankment, adjacent to conrete outlet pipes or other structures or adjacent to the foundation and abutments.)
- differential settlement cracks
- cracking in outler pipes which can be due to
  * foundation settlement, or
  * spreading of base of dam, or
  * deterioration of the pipe.

There are two modes of occurence of this type of leak. If the water flowing in the pipe is under pressure, it may be forced into the embankment and downstream, otherwise the eroded soil may be lost into the pipe.

- natural foundation soils; natural soils are more erratic and often less dense than embankment materials. Thus, leakage is more common here.

- animal burrows and drying cracks can sometimes cause leaks which may lead to piping failures.

2.3.3 Cracks

Cracks are formed when a portion of the embankment is subjected to tensile strains during the deforming of the dam by differential settlement. They may be localized or continuous. They may open as much as 15 cm in width, but widths of 2,5 cm or 5 cm have been more common /22/. Cracking has developed most commonly in low dams of about less than 30 m and in the upper portions of high dams. In the lower portions of high dams the heavy weight of the dam counteracts cracking.

Cracks may be transverse, longitudinal or intermediate to the dam axis. The plane on which they are formed may be vertical, horizontal or inclined in any direction.

Transverse cracks are the most dangerous causes of earth dam failures since they create a path for concentrated seepage through the core. The following are some of the mechanisms of formation of transverse cracks:

- a narrow valley with rock abutments causes the arching of the upper portion of an embankment during settlement and this results in roughly horizontal cracks

- at sites where the foundation is compressible, compression of the embankment, even when it is well constructed, can cause dangerous transverse cracking

- embankment subjected to differential settlement produces the most dangerous transverse cracks.

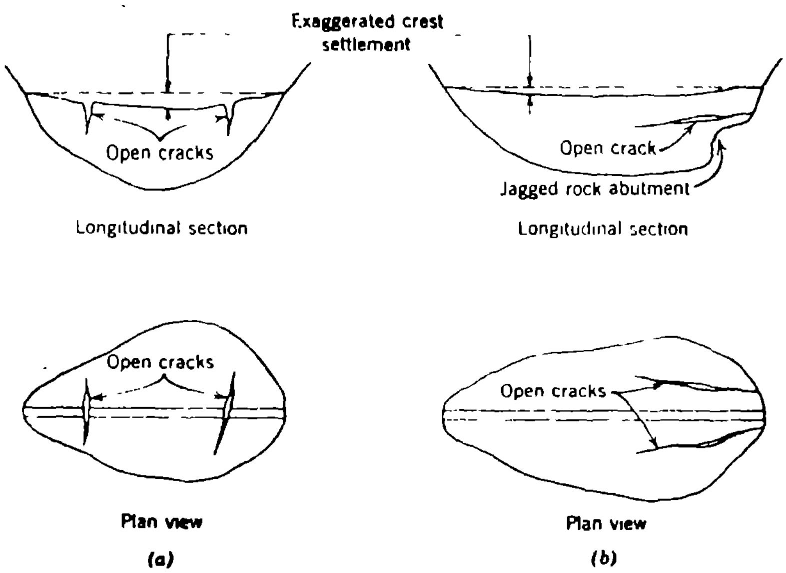

Figure 2. Typical transverse differential settlement cracks.

When section of rolled-earth embankments have been placed in trenches through compressible foundations for the purpose of supporting outlet conduits, severe transverse cracks are formed due to the differential settlement of the embankment. Figure 3 illustrates this.

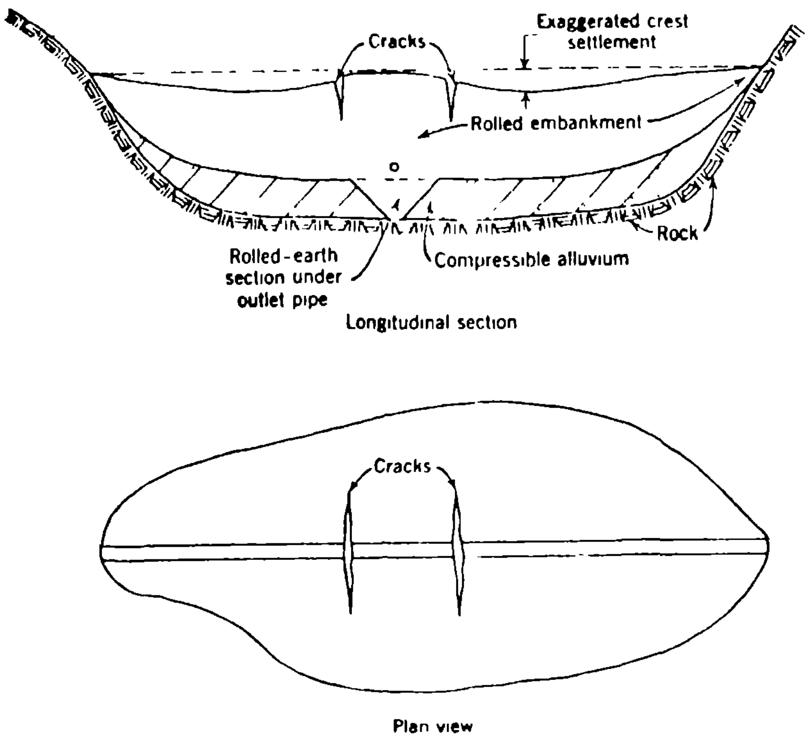

Figure 3. Cracking due to differential settlement between natural foundation soil and rolled-earth support under outlet pipe (or other discontinuity in the foundation).

Longitudinal cracks are not as much dangerous as transverse cracks. Usually the openings are not very deep, although some have been found to have depths of about 6 m. Like in transverse cracks, longitudinal cracks are formed due to different types of differential movement. Two possible cases are shown in figure 4. Longitudinal cracks may occur in conjunction with other unseen cracks running transversely through the core and this exposes the dam to a more dangerous situation.

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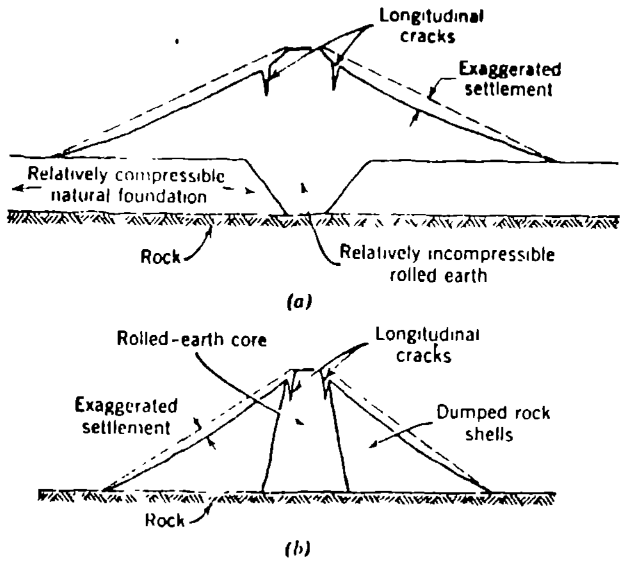


Figure 4. Longitudinal cracks.
a) cracking caused by differential foundation settlement
b) cracking caused by differential settlement between embankment sections of dumped rock and rolled earth.

In addition to the types of cracks discussed in the preceding paragraphs, there are other categories which are not visible at the dam surface and their existence can only be inferred. One of these types occurs with narrow central cores of compressible and impervious material. During contruction the core tends to compress more under the weight of the overlying fill than the shells do, so that part of the weight of the core is transferred to the shells by shear stresses and by arching. This effect can result in horizontal cracks in the core as shown in figure 5a.

A second category of these cracks is formed when a relatively short length of the embankment is underlain by a more compressible foundation material than that which exists under the rest of the dam, figure 5b. Another category is localized internal cracks formed in impervious zones adjacent to interior concrete structures, figure 5c.

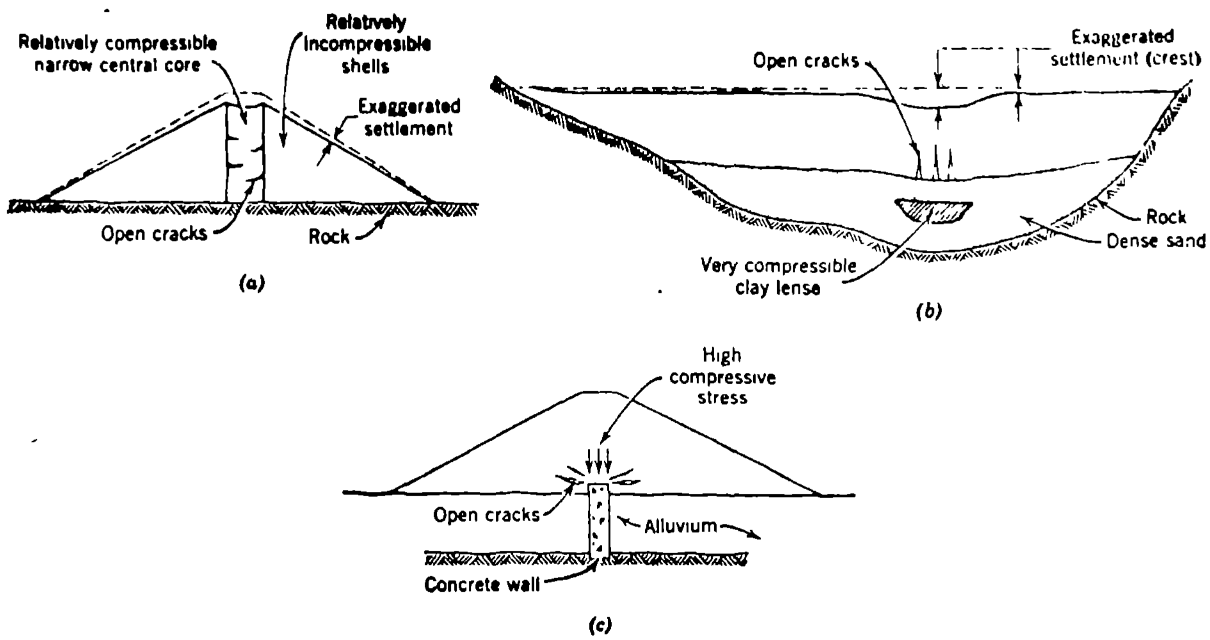


Figure 5. Internal embankment cracking.

From the foregoing discussion it is apparent that the magnitude of cracking depends on the tensile deformation occuring in a dam as related to the stress-strain characteristics of the fill materials. But the extent of damage depends on the leakage and erosion properties of the soil. Therefore cracking, leakage and erosion properties are in conjunction responsible for the stability of a dam.

These properties have been studied in physical models made of well graded silty-gravely sand and silty-clayey fine sand. The well graded silty-gravely sand has a self healing effect.

Erosion starts at a certain deformation and diminishes ultimately within an interval of time. The blow-off occurs when the tensile deformation approaches the maximum particle size. The silty-clayey fine sand has no self-healing effect. Erosion starting at a certain deformation continues and increases with time along with increase in leak discharge and finally makes the blow-off /10/.

As an example of a dam that failed due to piping, Wiyenzele Dam in Dodoma region, Central Tanzania is cited. The dam was completed in 1962. The total length is 720 m and maximum height above natural ground level is 12 m. At the filling of the reservoir in December 1962 a section of the dam, about 30 m in length, was washed away because of piping immediately above the river bed. Layers of sand material passing through the clay core were visible in the breach in the dam and in inspection trenches excavated afterwards. Also the embankment was to some extent damaged by cracks /25/.

2.4 Failures by Sloughing

Sloughing is a type of damage which is closely related to piping. It occurs in a few older homogenous dams. It starts with a miniature slide resulting in a relatively steep face.

Saturation by seepage helps a continued sloughing. After every sloughing a relatively higher and unstable face is formed until a thin portion of the dam remains. Finally the reservoir breaks through the thin portion causing a complete catastrophic failure. Failure of this type has taken place only when the whole downstream portion of the dam has been saturated.

2.5 Failures by Sliding

Embankment and foundation slides cause frequent failures of earthdams. Slides occur when the average stress along any potential sliding surface becomes greater than the average strength of the construction material. Slides in an earthdam can be classified into three categories:

1. slides during construction
2. downstream slope slides during reservoir operation
3. upstream slope slides after reservoir operation.

2.5.1 Slides During Construction

These slides may involve the upstream or downstream slope or both. No loss of life or property damage is encountered other than the dam itself. According to Sherard et al, in every case of construction slide the dam was underlain by a foundation of either soft, brittle, or sensitive clay, usually of high plasticity, and a large portion of the sliding surface passed through the foundation /22/. Construction slides can be slow or rapid.

A slow slide starts gradually and continues sliding for 1-2 weeks. The vertical and horizontal components of movement are 5 - 15 % of the height of the dam which is regarded as a small percentage.

While the sliding never completely stops, movement slows down at the end of the initial period to an unimportant rate in a creeplike action. Slow slides occur when the foundation is composed homogenous deposit of soft clay which is not sensitive /22/.

A rapid slide happens very suddenly with a larger magnitude of movement which is usually equal to one-half of the height of the dam. The major part is over in a few minutes and the movement either stops or slows down to a creeplike rate within several hours /22/. Rapid slides result when the foundation clay contains horizontal bedding planes, lenses, or layers of silt or fine sand through which the high pore water pressures developing under the centre of the dam can be transmitted outward towards the more lightly loaded areas under the toe.

2.5.2 Downstream Slope Slides During Reservoir Operation

Downstream slope slides which occur during reservoir operation may be deep slides which generally pass through a clay foundation or shallow surface slides.

Deep slides usually take place during full or nearly full reservoir condition and reduce free board by extending further upstream than the upstream edge of the crest of the dam, figure 6.

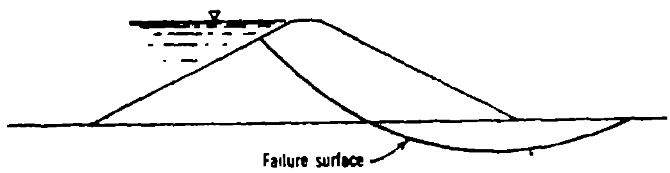


Figure 6. Deep downstream slides frequently extend to the upstream slope below the reservoir water level.

Deep slides are caused by pore water pressure which is developed from seeping water from the reservoir through or under the dam.

There is no relief of the pore pressure after the slide has taken place. The unstable scrap left standing slides again until it breaches the dam and a great flood wave results. Deep slides move at about the same rate or somewhat faster that slow slides during construction. A typical movement might be 8 to 10 cm per day in the first day or so and then approximately 3 cm per day for several weeks /22/.

Most shallow slides follow heavy rainstorms and do not extend more than 5 to 8 cm into the embankment in a direction normal to the slope /22/. Shallow slides following rainstorms have often occured where there are heavy layers of downstream rock rip-rap which retains the rainwater in many small puddles and produces greater saturation of the downstream slope. Poorly drained downstream berms or poorly drained roads on the downstream slope also lead to slides. Shallow surface slopes involving only the upper few centimetres have sometimes occured when the embankment slope have been poorly compacted. But this does not endanger the safety of the dam. It can often be embarassing to the engineers.

2.5.3 Upstream Slope Slides Following Reservoir Operation

Although occurrence of upstream slope slides during full reservoir operation is not impossible, most of the known cases have been following reservoir drawdown. Thus, the most critical operation condition so far as the stability of the upstream slope is concerned is a rapid drawdown, following a long period of high reservoir level.

Figure 7 shows the effect of rapid drawdown on the pore water pressures measured in Alcova and equal pressure lines under full reservoir conditions, while figure 7b shows the pressures under drawdown condition.

If special provision are not made for a dam subjected to such a condition, it will most likely end up with an upstream slope slide.

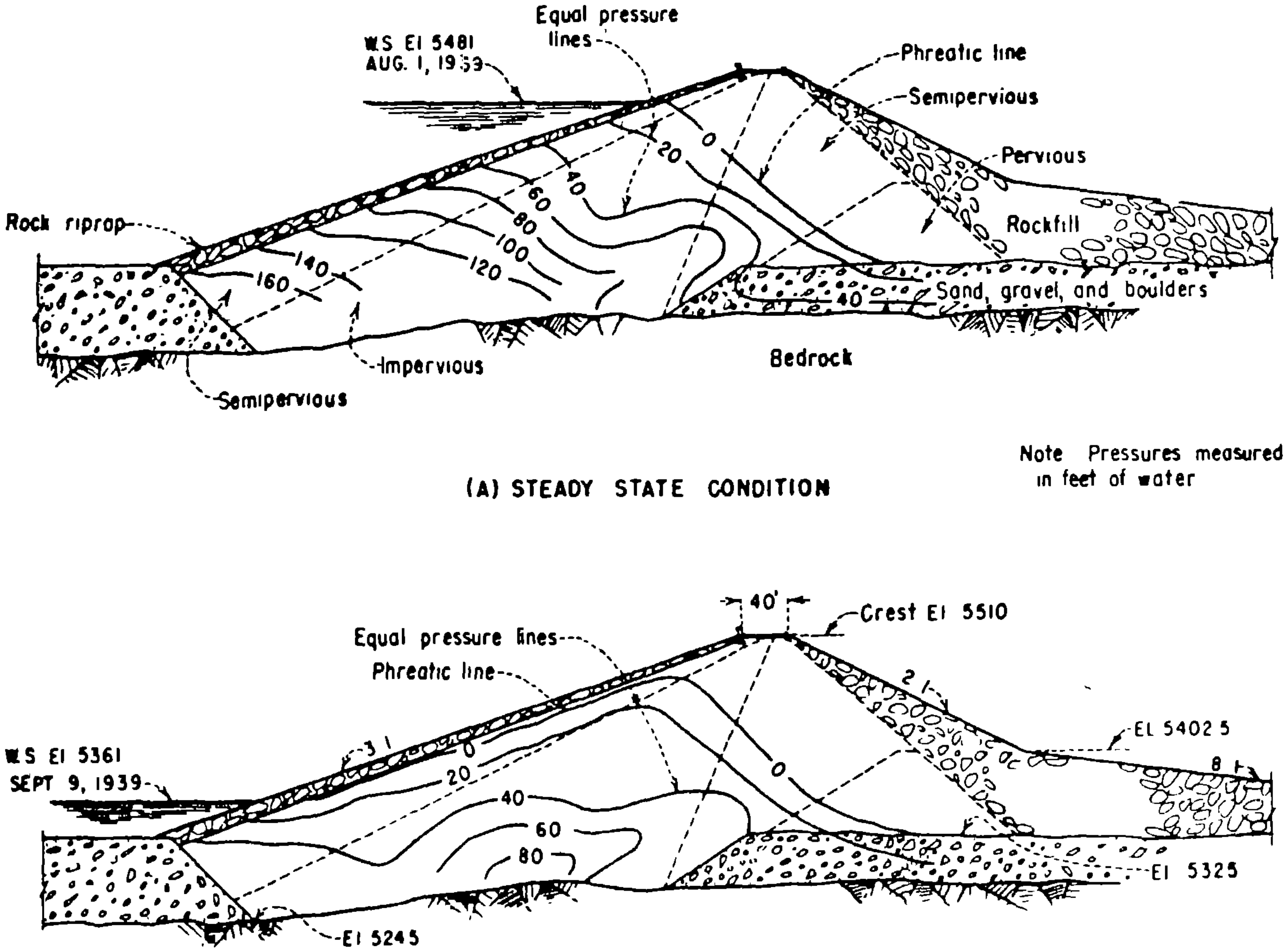


Figure 7. Effect of rapid drawdown on pore pressures at Alcova Dam, an earthfill structure on the North Platte River in Wyoming.

Upstream slides have not caused complete failure or loss of water from the reservoir, although they have occasionally blocked the entrances to outlet conduits and made these useless for further lowering of the reservoir, sometimes creating a very awkward and dangerous situation. Following an upstream slope slide caused by reservoir drawdown, the excess pore pressures within the embarkment soil adjacent to the surface of sliding are dissipated to a large extent. Thus, there is a lesser tendency for continued sloughing and sliding than there is in the downstream slides, in which the pore pressure are not likely to be diminished. Since the slide comes to equilibrium at low reservoir stage, there is small likelihood of catastrophic failure even though a large earth movement has taken place.

Extremely rapid rates of drawdown are not necessarily involved. A study has shown that the majority of the slides have occurred between average rates of 9 and 15 cm/day. The majority of upstream slides have been deepseated with the sliding surface passing well into the clay foundation. The upper edge of the sliding surface normally extends only to the upstream edge of the crest /22/.

In some dams constructed for hydroelectric power, where the reservoir essentially remains full all the time, the rip-rap wave protection is placed only at the top of the dam in the range of elevations over which the reservoir will operate. If the upstream slope is relatively steep, trouble-some surface slides may develop and become progressive as the reservoir is being filled and the water level is still below the bottom of the rip-rap. These slides are caused by erosion from wave action on the unprotected lower slope of the dam.

2.6 Earthquake Damages

There are no complete instrumental records of the response of an earthdam to a strong earthquake; even limited records for smaller shocks are scarce. By "complete" is implied records capable of an adequate definition of input ground motion and dam structural response; a better picture of behaviour requires records of soil and pore water pressure as well /1/.

One of the few dams known to have failed under seismic forces was the Sheffield Dam in Santa Barbara, California, earthquake of June 29, 1925 which had a Richter magnitude of 6,3 and a epicentre about 11 km from the damsite.

This 7,6 m high earthfill was 220 m long and had slopes of 2,5 to 1. Founded on a silty sound layer about 2 metres thick, the embankment consisted mostly of this same material, except a clay blanket on the upstream slope. Seepage had saturated the foundation alluvium and the lower part of fill. In response to the earthquakes' estimated maximum acceleration of 15 % of gravity (i.e. 0,15 g), and 15 to 18 seconds of significant vibration, the relatively loose saturated silty sand near the base failed in liquefaction.

The consensus of the investigators who viewed the broken dam was that the earthquake "had opened vertical fissures from base to top" and had "formed a liquid layer of sand under the dam, on which it floated out, swinging about as if on a hinge". A 91 metre length of the embankment at its centre slid downstream about 30 metres. As a consequence, approximately 113 000 m^3 of water spilled into the city of Santa Barbara /5/.

Earthquakes may effect dams in various ways. Seismic forces may be transmitted directly from the foundation to the structure. Overtopping water waves may be generated by landslides or oscillation of the reservoir or sudden movement of the dam. As demonstrated at the Sheffield Dam, foundations and embankments under certain circumstances may suddenly weaken when subject to prolonged vibration. Liquefaction of fine-grained cohesionless soils under such condition can place a dam in jeopardy /5/.

2.7 Slope Protection Failures

The upstream slopes of most earthdams are protected with one of the following materials in decreasing order of frequency:

- dumped rock rip-rap
- hand placed rock rip-rap
- articulated concrete pavement consisting of individual slabs
- monolithic reinforced concrete pavement.

2.7.1 Dumped and Hand-placed Rip-Rap

During a heavy storm the waves on the surface of a reservoir beat repeatedly against the slope just above the reservoir water level, and their energy is dissipated in turbulent action on and within the rocks of a rip-rap layer. As a wave strikes the slope, the water rushes upward into the rip-rap and filter layer and then, in the lull before the next wave strikes tumbles back downward. This action may damage dumped rip-rap layers in two main ways:

- First, if the filter material is too fine, the wave water moving in and out of the rip-rap may gradually wash the filter out; in an extreme case where the filter is completely removed, the individual rocks in the rip-rap layer settle and expose the embankment to wave erosion.

- Second, if the average size of rock comprising the rip-rap is not heavy enough to resist the hydraulic forces generated by the waves, rocks may be literally washed out of the layer.

Failure of the filter layer is more common. The larger rocks are more likely to be moved out of place because they have been undermined when the filter layer was washed out. Beaching of the type shown in figure 8 is the typical result of failure of dumped rip-rap.

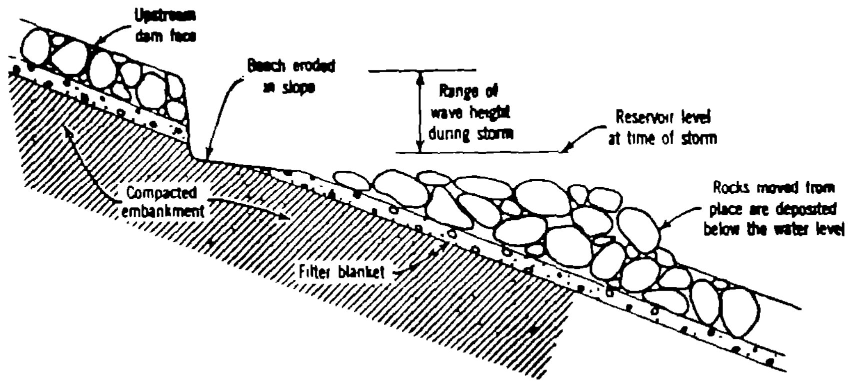


Figure 8. Typical failure of rock rip-rap.

Another kind of damage occurs when rip-rap layers are constructed with very large rocks of uniform size and without enough small rocks to make a well graded blanket.

The Itamuka Dam is Singida region which was completed in 1961 was seriously damaged by wave erosion in 1962. The dam had to be emptied by cutting a trench through the embankment. The trench revealed some construction defects as well /25/.

2.7.2 Articulated Concrete Pavement

Slope protection of this type has been the least successful of all the principal methods used. Almost invariably the failures have occured because wave water has washed large quantities of the filter or embankment material through the cracks between the individual slabs. The loss of material has caused serious settlements of the slab and the development of more cracks through the wave can wash off the soil. A few failures have occured from deterioration of the concrete.

2.7.3 Monolithic Reinforced Concrete Slabs

This kind of slope protection has a life of at least 30 years and probably many more. One of the disadvantages of concrete slab faces is that waves run up the smooth slope further than they do on a rough rock surface and a considerable amount of water can be thrown over the dam crest.

2.8 Damage Due to Burrowing Animals

Burrowing animals have been responsible for piping failure in a number of small earth dams and dikes but have not caused trouble in major dams because animal holes do not penetrate to a great depth. In the US the worst pests are muskrats and ground squirrels. Muskrats burrow into embankments either to make homes or to dig passages from one pond, figure 9. In Tanzania, termite holes have been noticed in some earthfill dams /25/.

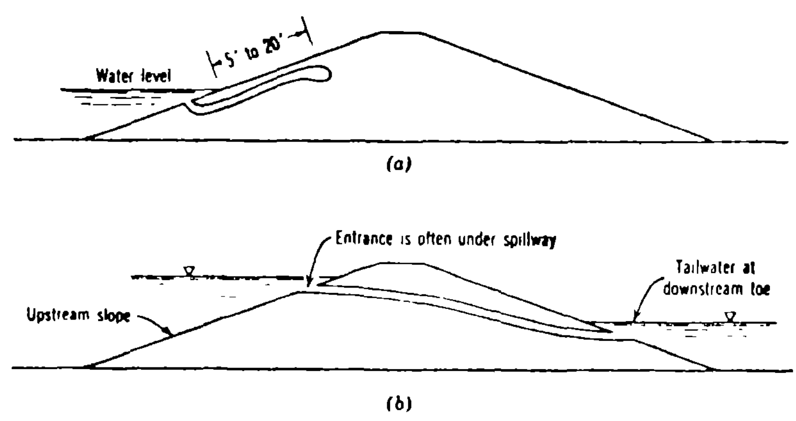


Figure 9. Holes by burrowing animals in earth dams.
a) Typical muskrat hole for nest.
b) Typical muskrat hole used as passage between the reservoir and the downstream pond.

2.9 Damage Due to Water Soluble Materials

2.9.1 Soluble Materials in Foundation and Abutments

Leaching of natural deposits of water soluble materials from abutments and foundations has caused difficulty at some dams. Gypsum which is gradually dissolved by seepage water from the reservoir has been particularly troublesome in this respect.

The leaching of soluble salts creates cavities and solution channels which may be dangerous for the safety of a dam. The deposition of soluble material previously leached from the natural soil may tend to plug specially designed fitters and pervious drains.

2.9.2 Soluble Materials in Embankment Soils

Solutions of salts have been the suspected cause in the failures of a few old dams constructed in arid regions where the soil had a high percentage of soluble materials.

However, these failures occured many years ago when embankments were built with what would now be considered rudimentary construction control. Small quantities of soluble salts are not as troublesome in embankment materials as they are in foundations and abutments because:

- the water in a river running through a region where the soil contains much soluble salt has a high salt content itself, and consequently the reservoir water will show no great tendency to leach salts out of an embankment composed of such soils

- in the construction of modern rolled-earth dams the water which is added to the borrow material to bring the construction water content up to optimum probably dissolves much of the soluble salt.

One the other hand, Cedergren /4/ reports that waters high in sodium tend to flocculate soils high in alumina, thereby reducing the permeability. Fresh water may deflocculate clay soils, rendering them more permeable. The failure of a number of small dams in Australia has been attributed to post-construction deflocculation that led to piping when fresh river water was suddenly turned into reservoirs that had been kept filled with saline water. Compaction at optimum or slightly wet of optimum is recommended as a means of avoiding piping failures in clay soils.

In extremely arid climates where little or no water is available for compaction, chemical aids are recommended. Maintenance of a low sodium adsorption ratio in the percolating water is essential to prevent undesired changes in the soil sodium adsorption ratio following leaching /4/.

2.10 Damage Due to Surface Drying

Surface drying cracks have caused a constant maintenance problem on a few low dams constructed with homogenous sections of clayey soil. Usually the main cracks in extreme cases have been several centimetres wide, develop near the top of the dam parallel with the crest /22/. The worst conditions develop when some combination of the following factors occurs:

- hot, dry climates in which the reservoir remains empty for long periods
- embankment construction materials of highly plastic or extremely fine silty soil, and
- embankments not compacted to high densities.

3. HYDROLOGY OF DAMSITES

3.1 Introduction

When designing a dam one has to have an idea of the expected floods in order to ascertain whether the reservoir will fill up and provide a dependable flow to satisfy the expected demand. Secondly the maximum probable flood has to be estimated so that sufficient spillway capacity can be provided to safely pass the flood. In addition the magnitude and frequency of floods should also be found from statistical analysis of stream flow records. These are primarily for use in connection with estimating diversion requirements during construction, evaluating flood damage risks and also for devising rule curves for operation of the spillway after commissioning of the project.

3.1.1 Stream Flow Data

The hydrological data most directly useful in determining flood flows are actual stream flow records of considerable length at the location of the dam. Such records are rarely available. Efforts should be made to obtain stream flow records available in the vicinity of the damsite. There should be such records although they are in most cases of very short duration - in most cases less than ten years duration. Where stream flow records are not available, the inhabitants of the vicinity may have information about high water-marks caused by specific historic floods.

If stream flow records are available at or near the damsite for 20 years or more, they can be analysed to provide flood frequency values and notable flood events can be used to find the runoff factors for use in determining the maximum probable flood. If such a record is available but covers only a few years, it may not include any flood of great magnitude within its limits and, if used alone it would give false indication of flood potential.

However, analysis may give some or all of the runoff factors needed to compute the maxium probable flood. Frequency values obtained from a short record should not be used without analysis of data from nearby watersheds of comparable runoff characteristics.

When stream flow records are available on the stream itself, but a considerable distance from the damsite, they can be used to obtain unit hydrograph characterictics and frequency data which may be transferred to the damsite by using area and basin-characteristics coefficients. This transfer can also be made from one drainage area to another if the areas have comparable characteristics /6/.

When there are no stream flow records on the stream and on comparable drainage basins use of high water marks can be used. These should be used with caution in estimating flood magnitudes. They are much more dependable when they are several near the damsite and especially if some local authority e.g. roads department, has a record of these marks. These records may be used to determine the water cross-sectional area and the water surface slope for the flood to which they refer, and from these data an estimate of that particular flood peak may be obtained /6/.

In the design of the Mindu Dam, records were available for a maximum of 15 years /18/. It was found that the duration of the data is inadequate for a sensible prediction of extreme flood flows on the Ngerengere River. The extreme flood was therefore derived from an examination of all notable flood events recorded in the hydrological year books of Tanzania for the periods 1950-59 and 1960-65. Values of maximum flows were plotted against catchment area, and a limiting line drawn. On this basis a design flood intensity of 1,82 m^3/s per km^2 was adopted for Mindu Dam, which for its catchment area of 303 km^2 gives a peak discharge of approximately 552 m^3/s. See figure 10.

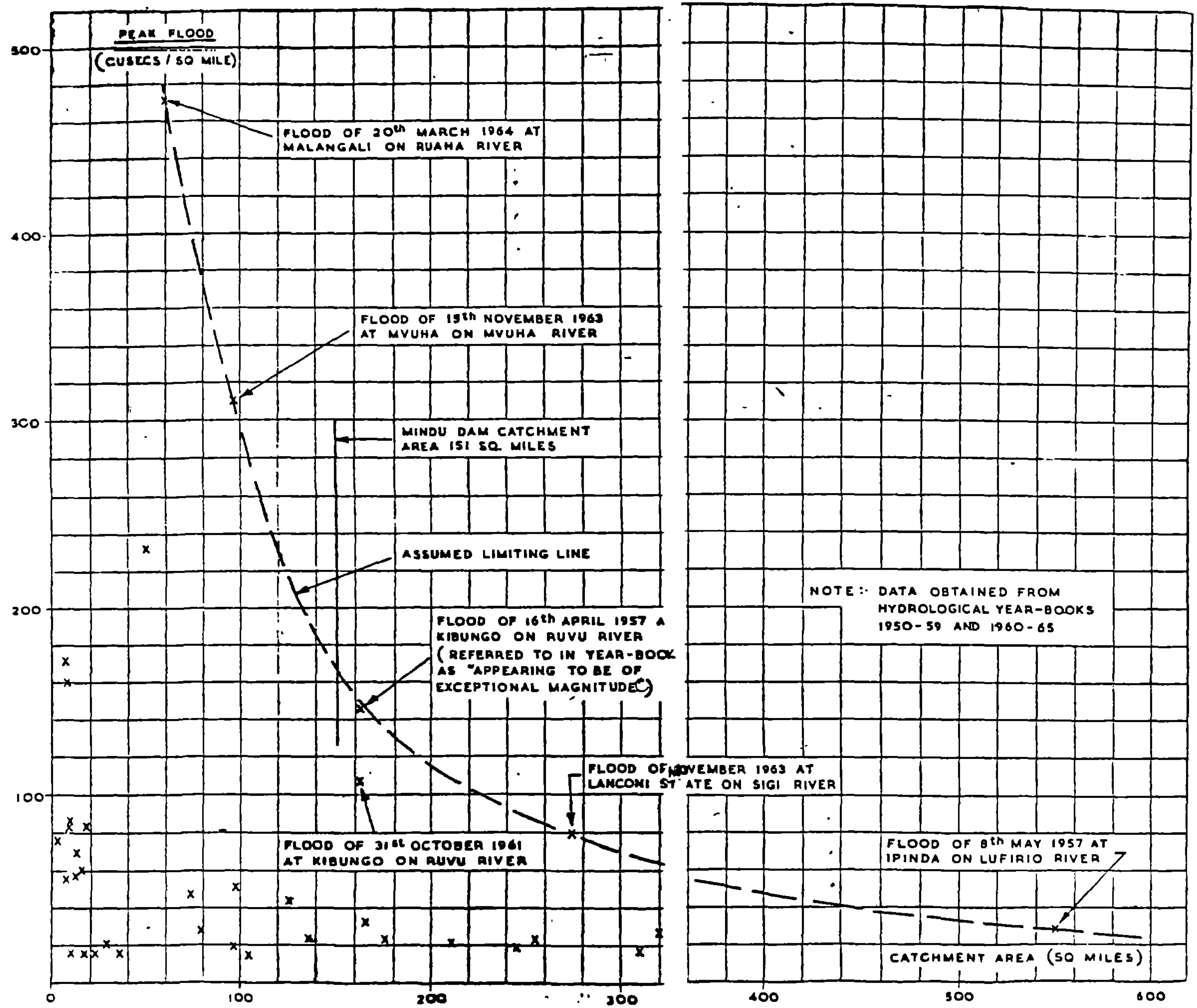


Figure 10. Notable flood events in Tanzania.

3.1.2 Precipitation Data

Precipitation data is required in all the preceding cases for evaluating factors for use in computing the maximum probable flood. Information on precipitation should be assembled especially those which show great storms for which runoff records are also available.

In Tanzania most small catchments lack sufficient rainfall and runoff data and the designer has to sometimes adopt characteristics developed elsewhere for choosing the design flood discharge. This has led to some dams failing due to overtopping on the one hand and quite uneconomical spillways on the other. We shall cite here Kilimi Dam in Tabora as an example:

Project:	Kilimi Dam
Location:	Nzega District, Tabora Region
Dam height:	15 m above original bed level
Spillway:	
Discharge:	300 m^3/s
Bed width:	150 m

The excavation of the spillway was almost completed when somebody raised an alarm about the spillway size. After some communication it was found that the maximum flood could not be more than 100 m^3/s and most likely around 50 m^3/s, which agrees quite well with nearby drainage basins. The spillway which is almost completed has yet to have its size fixed todate. This may be due to lack of enough studies during the design period which should have enabled fixing of these dimensions before the project was approved for construction.

The selection of the design flood for dams in Tanzania has always been a matter of debate before a final figure is fixed.

It is recommended that whenever it appears that there will be one or more flood seasons between the selection of the damsite and construction of the dam, facilities for securing a stream flow record for the project should be set up as promptly as possible. This is of particular importance in order to obtain watershed data directly applicable to the computation of the inflow design flood for the dam, although a record usable for frequency computation cannot be secured. Such measurements may produce basic data which would justify "eleventh hour" revision of the plans, thus improving the design of the dam.

3.1.3 Use of Data

The objective of analysing stream flow and precipitation records is the development of procedures whereby the hydrograph that will result from a given amount of rainfall may be estimated. Another objective is the computation of a flood magnitude-frequency relationship based on experienced events. The primary use of recorded flows and precipitation records in the computation of an inflow design flood is the determination of a unit hydrograph and of retention loss characteristics of the watershed.

3.1.4 Watershed Data

Besides the stream flow and precipitation data it is necessary to assemble all other relevant data for the watershed. A map of the area upstream of the damsite should be prepared showing the drainage system, contours if available, drainage boundaries, and locations of any precipitation stations and stream flow gauging stations. Available data on soil types, cover, and land use provide valuable guides to judgement. An inspection trip should be made over the watershed to verify drainage area boundaries and soil and cover information, and to determine if any non-contributing areas are included within the drainage boundaries.

Just to give an idea about the necessity of ascertaining the catchment area: the Mindu Dam in Morogoro was designed for a catchment area of 390 km^2. The design flood intensity is 1,82 m^3/s per km^2, which gives a peak discharge of 710 m^3/s. Subsequent measurements indicated that the actual catchment area is 303 km^2. Using the same design flood intensity this gives a peak discharge of 552 m^3/s. This means that other factors remaining the same, the provided spillway is overdesigned by 22 %. The implication of this overdesign on the cost of spillway excavation is obvious. Therefore there is a need to actually correlate data from maps with field reconnaisance.

3.2 Methods of Estimating the Inflow Design Flood

Most of the damsites in this discussion fall outside the catchments characterized in Tanzania as of national importance, and this means that there is rarely a river gauging satation for the particular basin. As such the estimation of the design flood is mostly derived from rainfall data, and the methods for arriving at this will be discussed.

Further it is worth mentioning that rainfall stations have been operating for a long time compared to gauging stations even on the important catchments and therefore it may be more appropriate to use the rainfall data of a longer duration than the 5 - 10 years gauging data for estimating the design flood. However, the density of these stations is rather low and therefore the obtained values may only give order-of-magnitude estimates due to predominating orographic factors.

3.2.1 USSCS Method

The US Soil Conservation Service's Manual on "Hydrology Guide for Use in Watershed Planning" /11/ gives a method of finding the inflow design flood for ungauged areas.

The resulting inflow design flood hydrograph represents direct runoff from precipitation in the form of rain over a watershed having no unusual characteristics. This method shall be illustrated with data from Mindu Dam, Morogoro.

a) Catchment Characteristics

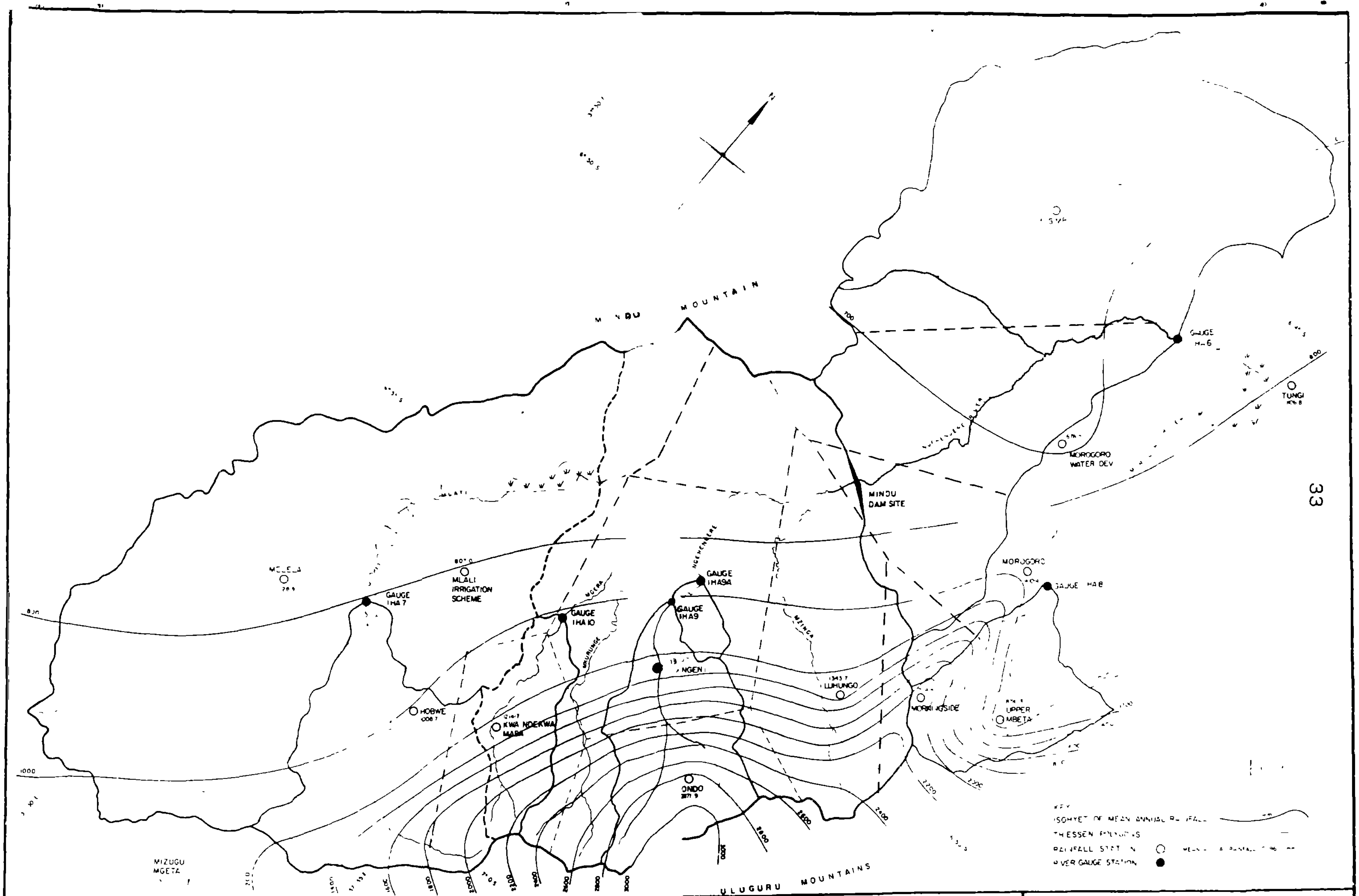
The following catchment information is always needed:

1. Geographical location. This should provide information about the location of the dam together with the variation of altitude within the catchment.

2. Map showing topography, streams and runoff contributing drainage area.

   For Mindu Dam the total catchment area is 303 km^2. Out of this some 169 km^2 is contributed by the catchment of the Mlati River which drains the southern part of the catchment and which discharges to an area of seasonal swamps in the upper part of the Ngerengere Valley. Due to evaporation losses from the swamps and possible ground water recharge, the Mlati River makes a disproportionately small contribution to the total surface flow in the Ngerengere.

   There is certainly an effect of attenuation in the marshes which will reduce the flood peak in the Mlati River. Therefore for flood studies it is considered necessary to examine not only the flood contribution from the total catchmen, but also the more flashy conditions that might be expected from the limited catchment drained by the other rivers exluding Mlati, amounting to 134 km^2 in area (figure 11).



REVISIONS	Figure 11.	SIR ALEXANDER GIBB & PARTNERS (AFRICA) CONSULTING ENGINEERS	UNITED REPUBLIC OF TANZANIA MINISTRY OF WATER ENERGY AND MINERALS MOROGORO WATER SUPPLY CONTRACT No 1 — MINDU DAM	CATCHMENT MAP Date OCT 1979 — DRG NO 7655/10

3. Rainfall distribution

As already mentioned Tanzania is in an area of summer rainfall, contributed principally by the southern zone of intertropical convergence. Two seasons of rainfall are therefore experienced.

At Morogoro there occur in November-December and March-April. However, being a coastal region significant rainfall brought in by the prevailing south east winds occurs throughout the year and the incidence of this rainfall in the Morogoro area is accentuated by the presence of the Uluguru Mountains. The catchment lies in an area of rain shadow on the lee side of the mountains, and therefore exhibits an extreme rainfall gradient varying from in excess fo 300 mm mean annual rainfall at the top of the mountains to a general level of about 700 mm in the valley bottom.

This rainfall gradient and the random nature of rainstorms makes it particularly difficult to assess storm runoff by conventional unit-hydrograph techniques /Gibb/.

4. Catchment time of concentration

Various empirical methods are available for estimating the catchment time of concentration, which is defined as the time taken for water to travel from the hydraulically most distant point of the watershed to the point of exit from the catchment.

Most methods relate time of concentration to mean channel gradient, obtained by dividing total fall in channel level by total channel length.

Applying such methods give times of concentration of the order of three and one half hours for the whole Mindu Catchment and one and one quarter hours for the reduced catchment excluding the Mlati River. Mean channel velocities obtained with these values are abnormally high (2,0 and 2,9 m/s respectively).

This indicates that the above methods may not be strictly applicable to catchments in which channel gradients vary substantially throughout their length.

Table 2. Seasonal distribution of 24 hour rainfall (mm).

Month	Return Period (Years) 2	5	10	20	50
January	28	43	53	63	76
February	31	53	67	80	98
March	39	56	67	78	93
April	36	50	59	67	78
May	20	29	34	40	47
June	8	18	25	31	39
July	4	10	13	17	21
August	5	11	16	20	25
September	8	20	28	35	45
October	11	23	31	38	48
November	20	40	53	66	83
December	27	45	57	68	83

An alternative method using a guide prepared by the Texas Highway Department was therefore used /6/. The guide gives typical mean channel velocities and using these the travel times in each stage of the river can be obtained and sumed up to give an aggregate time of concentration. By this method the time of concentration for the catchment with and without the Mlati River are estimated as 16 hours and 6 hours respectively.

5. Catchment area reduction ractor

In studies carried out by the UK Transport and Road Research Laboratory (TRRL) /9/ factors for reducing storm rainfall intensity by areal extent of storm were computed based on data from four catchments in East Africa.

Due to limited data, firm estimates for catchment area reduction factors could not be obtained, but a formula based on the available data was used by the TRRL.

$$ARF = 1 - 0{,}044A^{0{,}275}$$

where A = catchment area (km^2)

Factors based on this formula are then used for adjusting catchment rainfall for the areal extent of the catchment.

b) Design Storm Rainfall

1. Storm frequency analysis

In order to analyse the recurrence interval of rain storms (and therefore of floods) of specific magnitudes it is necessary to examine rainfall records of a length suitable for extrapolation.

The data is fitted to a probability distribution, usually the Gumbel probability distribution and the extrapolation for the required return periods is done.

2. Seasonal distribution of rainfall

Analysis of the seasonal distribution of rainstorms is carried out by applying the Gumbel probability distribution to the series of annual maximum of daily rainfall extracted separately for each month. Since this method may not be truly applicable to partial series abstracted in this way care should be taken not to extrapolate the distribution beyond the period of record /18/.

3. Short duration rainfall

The relationship between rainfall intensity and duration is normally expressed in the general form:

$$I = \frac{a}{(T + b)^n}$$

where I = rainfall intensity in mm/hr
T = rainfall duration in hours
a, b and n are constants.

The only recent analysis of rainfall intensity and duration for East Africa is that carried out by a team from the British Transport and Road Research Laboratory /9/.

Using data derived from twenty three autographic rainfall stations, and using an average value for b of 0,33 hours, values of "a" and "n" were computed for various return periods.

Values of "n" were found to fall into three general groupings for:

- inland stations
- coastal stations
- eastern slopes of the Kenya Aberdate Range.

As the primary interest in the analysis is for storm intensities of low frequency in index of 0,85 was used throughout for Mindu Dam. Using this value of "n" the relationship can then be used to relate 24 hours rainfall with rainfall of any shorter storm duration.

4. Design storm sequence

For unitgraph analysis the design storm is assembled as a hypothetical sequence of rainfall events distributed symmetrically in unit time. Suitable increments of time are taken, for example, for the Mindu Catchment one hour was adopted for the whole catchment and half an hour for the sub-catchment excluding the Mlati River. The design storm at any intensity is limited in duration only by the relationship stated in (3.) above.

Table 3. Average values of index "n".

	Return Period (years) 2	5	10
All Coastal Stations	0.82	0.76	0.76
Dar es Salaam	0.91	0.86	0.84
Dodoma	0.95	0.91	0.88

Table 4a. Peak flood estimate from whole catchment.

Return Period Years	Catchment mean 24 hr. Rainfall mm	Peak Rainfall Intensity mm/hr	Peak Excess Rainfall mm/hr	Peak Flood m^3/s
10	83.5	61.2	47.2	266
50	103.4	75.8	63.8	363
100	112.3	81.4	71.2	408
500	131.5	96.4	87.4	507
1 000	140.1	102.8	94.6	552
10 000	168.0	123.2	117.8	699

Table 4b. Peak flood estimate from sub-catchment excluding Mlati River.

Return Period Years	Catchment mean 24 hr. Rainfall mm	Peak Rainfall Intensity mm/hr	Peak Excess Rainfall mm/hr	Peak Flood m^3/s
10	107.8	79.2	67.4	413
50	133.5	98.0	89.0	550
100	145.0	106.4	98.8	612
500	169.8	124.6	119.4	746
1 000	181.0	132.8	128.8	806
10 000	216.9	159.2	156.2	992

c) Catchment Retention

1. Flood runoff in the catchment results from storm rainfall not retained by surface ponding or by infiltration within the soil structure. The principal factors affecting the relationship between rainfall and runoff are the intensity and duration of the storm, the soil type and the antecedent moisture conditions of the soil.

The United States Soil Conservation Service /11/ developed a mass flow model of the form:

$$Q = \frac{(P - Ia)^2}{P - Ia + s}$$

where Q = direct runoff in mm
P = storm rainfall in mm
s = maximum potential difference between P and Q at time of storms beginning
Ia = the initial abstraction which on the evidence of data from watersheds in various parts of the USA is taken as 0,25.

The above relationship can be plotted as a family of curves against a curve number having the form:

$$CN = \frac{1000}{10+S} \quad \text{(S converted to inches)}$$

One unique curve number therefore defines the cumulative relationship between rainfall and runoff for any particular storm.

2. Calibration of model

A mass flow form of model is adopted in order to allow calibration using the available 24 hour rainfall and river flow records. For calibration purposes, river flow and rainfall data is used for common periods of record in order to relate the rainfall and runoff.

From the records a series of extreme runoff events are extracted. These are defined as 24 hour periods when the runoff exceeds three times the runoff in the preceeding period and the preceeding five periods are resonably uniform. These single extreme runoff events are compared in each case with rainfall for the preceeding five days.

For Mindu Dam, the model was expressed in the form:

Runoff = Excess rainfall + base-flow from ground water

$$\text{i.e.} \quad = \frac{(P - Ia)^2}{P - Ia + S} + B$$

and S and B are related directly to the Antecedent Precipitation Index (API) where

$$API = \sum_{i=1}^{5} K^i P_i$$

K being a constant and P_i the precipitation i days previously.

Coefficients of correlation of the order of 0,4 to 0,7 were obtained, the higher figures being obtained only by adopting what were considered to be unrealistic values for the parameters. It was concluded that calibration on the basis of 24 hour records is not possible because:

- The catchment time of concentration is only some 1 1/2 hours so that 24 hour records may include several storm records having very different characteristics.

- The time of reading of rainfall and river flow gauges is not known, so that the records may not be coincident.

- The river flow gauges are of the cipolletti weir type, intented primarily to measure low flows. It is therefore doubtful whether the records truly represent the peak of short duration floods.

3. Model parameters

In the absence of a direct calibration for the catchment model, parameters were adopted in accordance with guidelines presented by the US Soil Convervation Service. It must be recognised that these guidelines, prepared for soil catchment characteristics obtained in the US are not strictly relevant to conditions in Africa. However, in the absence of more authorative data it was considered reasonable to accept them.

The US Soil Conservation Service /6/ classifies catchment runoff characteristics in accordance with:

1. Hydrologic soil groups

   Four groupings classified A to D are listed.

   Group A. (Low runoff potential)
   Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well-to excessively-drained sands or gravels. These soils have a high rate of water transmission.

   Group B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

   Group C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

   Group D. (High runoff potential)
   Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly imprivous material. These soils have a very slow rate of water transmission.

The Mindu Catchment would probably fall within the group C classification having mainly fine grained silty clays (mostly laterites).

2. Land use patterns

This again classifies the catchment according to the soil-cover complexes which are in turn used in estimating direct runoff. Types of land use are classified on a flood runoff-producing basis.

The greater the ability of a given land use or treatment to increase total retention, the lower it is on a flood runoff-production scale /6/.

Three distinct zones within the Mindu Catchment can be defined. The forest reserve at the top of the catchment having, it is believed, a substantial depth of humus, would be classified as good woodland. The lower slope, where intensive cultivation is carried out and where although some contouring and terracing are practiced, general overgrazing leads to hill-wash, would generally fall within the classification of poor pasture. The valley bottom would be classified as good close seeded agricultural land.

3. Antecedent moisture conditions

Three groups of antecedent moisture conditions are listed:

AMC-1 A condition of watershed soils where the soils are dry but not to the wilting point, and when satisfactory ploughing or cultivation takes place.

AMC-II The average case for annual floods, that is, an average of the conditions which have preceeded the occurrence of the maximum annual flood on numerous watersheds.

AMC-III When heavy rainfall or light rainfall and low temperatures have occured during the 5 days previous to the given storm, and the soil is nearly saturated.

The curve numbers are computed and presented for the average conditions AMC-II. These can be converted to the other antecedent moisture conditions using tables.

In the case of Mindu Catchment, in considering extreme flood events AMC-III conditions were adopted and this follows curve No. 90 which was used for the model. For rainstorms of a higher frequency such as would be experienced under AMC-II conditions, curve No. 78 was adopted.

d) Flood Estimates

1. Unit hydrograph

The unit hydrograph is defined as the hydrograph of storm runoff at a given point that will result from an isolated event of rainfall excess occuring within a unit of time and spread in an average pattern over the contributing drainage area.

For Mindu Dam, since autographic rainfall and run-off records are not available it is necessary to assume a unit hydrograph shape consistent with unit hydrographs for similar catchments elsewhere.

The systhetic unit triangular hydrograph adopted by the US Soil Convervation Service is presented here. The dimensions of the triangular hydrograph are defined by the following three equations:

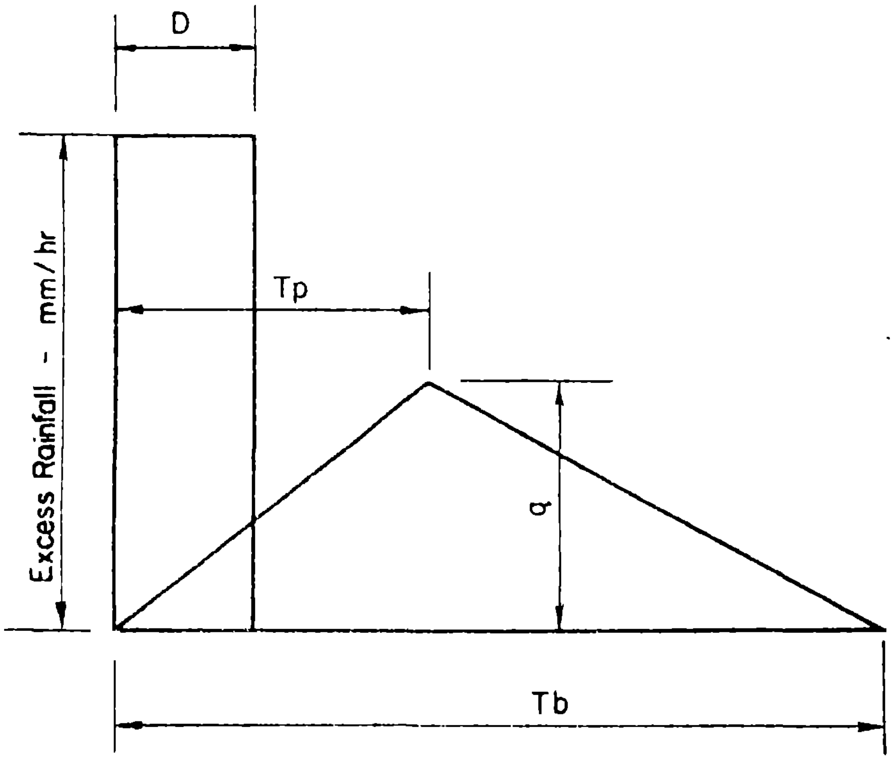
$$Tp = \frac{D}{2} + 0{,}6\ Tc$$

$$Tb = 2{,}67\ Tp$$

$$q = \frac{0{,}208\ AQ}{Tp}$$

where Tp = time to peak (hours)
Tb = time length of base of hydrograph (hours)
q = discharge at peak of hydrograph (m^3/s)
D = unit increment of time of hydrograph (hours)
Q = unit increment of excess rainfall for hydrograph (mm)
A = catchment area (km^2)
Tc = catchment time of concentration (hours)

The relevant values are shown in the figure 12 /18/.



	WHOLE CATCHMENT	SUB CATCHMENT EXCLUDING MLATI RIVER
D (hrs)	1	1/2
Tp (hrs)	10·1	3·85
Tb (hrs)	27·0	10·28
q (M^3/s)	6.24	7·24

Figure 12. Unit hydrograph dimensions for Mindu Dam Catchment.

2. Method of analysis

The method of analysis follows closely that presented in the USBR Design of Small Dams /6/.

The design storm sequence derived in B (4.) is converted to a sequence of excess rainfall by use of the catchment model. Adoption of the mass flow model presupposes that after a prolonged rain storm the unit increment of runoff approaches in value the unit increment of rainfall, giving unrealistically small values for catchment retention losses. To resolve this anomaly a minimum retention loss rate of 3 mm/h is adopted, this being the recommended mimimum rate for soils in hydrologic soil group C.

Each increment of excess rainfall is then converted to a unit triangular hydrograph of runoff. The total flood hydrograph is defined by the arithmetic sum of the coordinates of all of the triangular hydrographs.

The effect of base flows in the river on the flood peak is ignored as these flows are too small to be significant.

3. Peak flood estimates

Estimates of the peak flood that may occur at various return periods are shown in the following tables both for the whole catchment and for the sub-catchment excluding the Mlati River. As the flood peak is measured at inflow to reservoir, the effects of reservoir attenuation are not considered here. Since the flood from the sub-catchment is greater than that from the whole catchment, further discussion will be on the sub-catchment only.

Table 5. Seasonal variation in peak flood (m^3/s).

Month	Return Period (Years)				
	2	5	10	20	50
January	7	38	67	102	153
February	11	67	117	170	251
March	20	60	93	131	189
April	41	96	139	180	242
May	3	21	36	56	135
June	0	0	2	8	39
July	0	0	0	2	8
August	0	3	18	38	71
September	0	12	40	74	132
October	2	48	103	160	252
November	22	140	244	359	519
December	14	71	123	178	260

Table 6. Gauges 1HA9 + 1HA10* (1954-1959) Flood discharge fitted to Gumbel probability distribution (m^3/s).

Month	Return Period (Years)				
	2	5	10	20	50
January	5	7	9	11	13
February	14	24	31	38	46
March	55	91	115	138	168
April	60	115	152	187	233
May	32	63	82	102	126
June	5	8	11	13	16
July	2	3	4	5	6
August	3	6	8	10	12
September	14	46	68	89	116
October	3	5	7	8	10
November	6	10	13	15	18
December	8	18	24	30	38

* Adjusted pro rata to the respective effective catchment areas to represent flood discharge at Mindu damsite.

4. Seasonal variation in flood risk

Based on the seasonal distribution of rain storms (table 4, page 37, 38) the risks of flood in any month of the year are estimated. As these estimates are for floods of a frequency closer to that of the annual flood, soil moisture conditions are assumed as being equivalent to AMC-II. The following table gives the seasonal flood estimates for the sub-catchment excluding the Mlati River.

It should be noted that base flow, which is not included in the figures shown in the table, is likely to provide the only significant river discharge during the dry months of the year.

To corroborate values obtained by the unit hydrograph method, river gauge readings were plotted on Gumbel probability distribution.

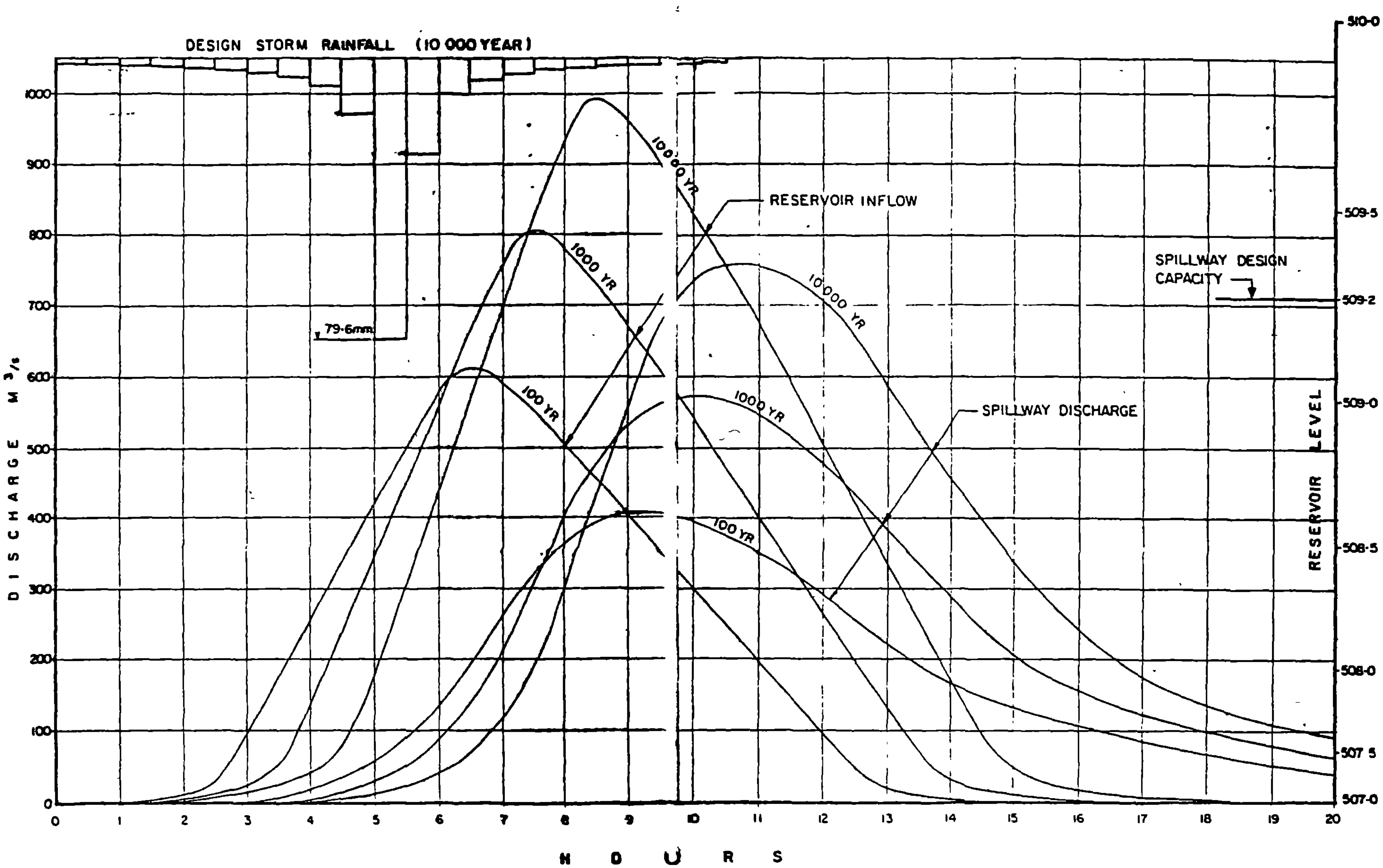


Figure 13. Discharge hydrographs for Ngerengere River at Mindu Dam.

It can be noted from the above table that the values predicted by the unit hydrograph method are at least of the same order of overall magnitude, though with substantial and significant differences apparent in individual months. For future flood prediction, direct river gauging at the damsite will provide the required data base. However, until such data is available, the figures in table 6 above are adopted for planning purposes during construction of Mindu Dam.

3.2.2 Kobalyenda Method

Kobalyenda /16/ used a computer aided method to simulate stream flow from precipitation data and other hydrological parameters on a case study of Little Ruaha River, Southern Tanzania.

A mathematical model was developed which correlates the land phase of the hydrological cycle as a component of the cycle to the total cycle. The basic system for the hydrological model is shown in figures 14 and 15.

The main hydrological processes are: precipitation, evapotranspiration, surface runoff, infiltration, percolation, recharge, base flow and stream flow. These processes in the model form a system.

The model needs a set of hydrological parameters to be used in the computation. Besides the stream flow coefficients, the variables used in the model include: interception storage, interception storage capacity, soil field capacity, soil wilting point and soil storage.

Essentially what is done is selection and use of these parameters to see if, with minor adjustments, they fit the model.

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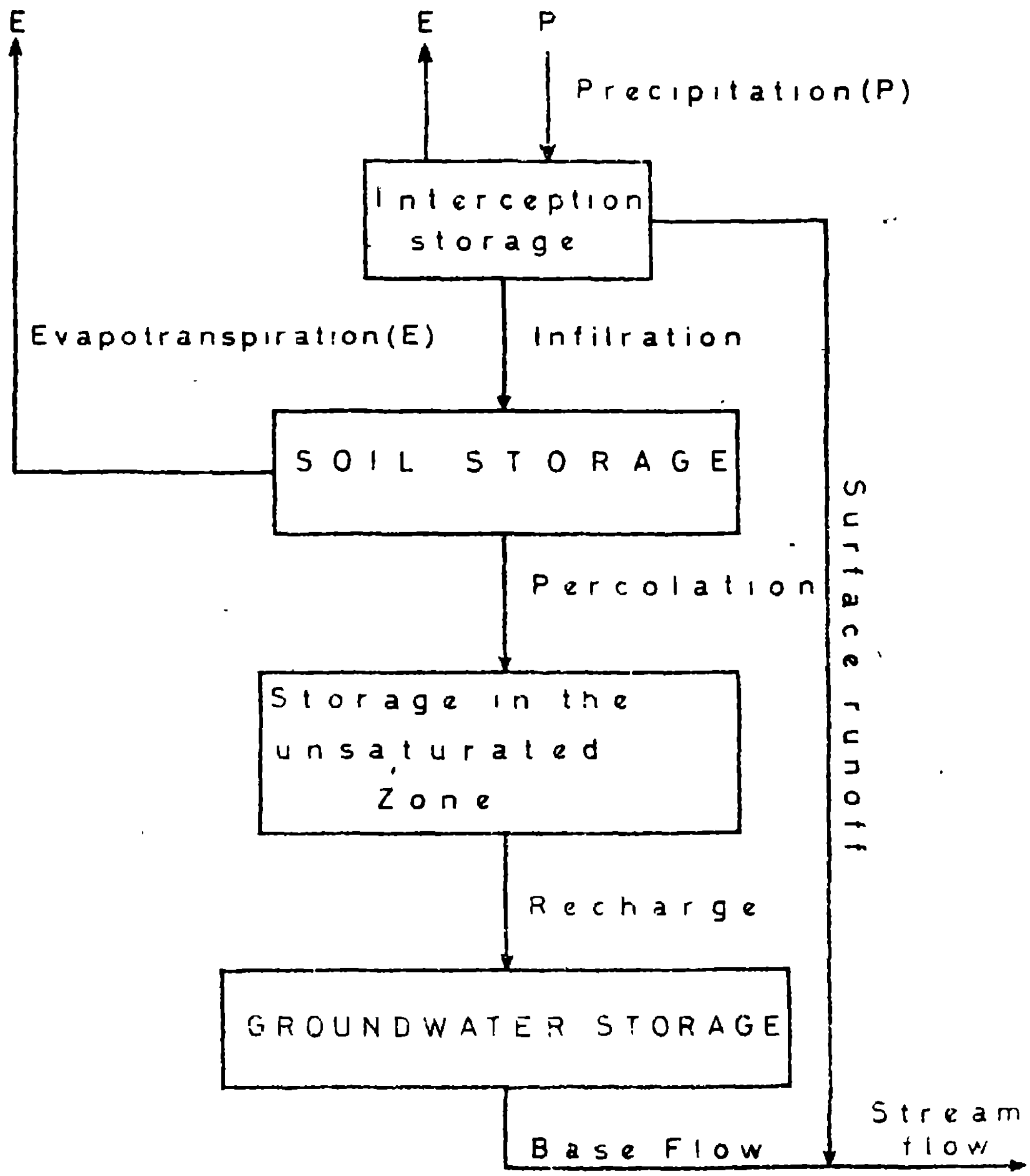


Figure 14. The basic system for the model.

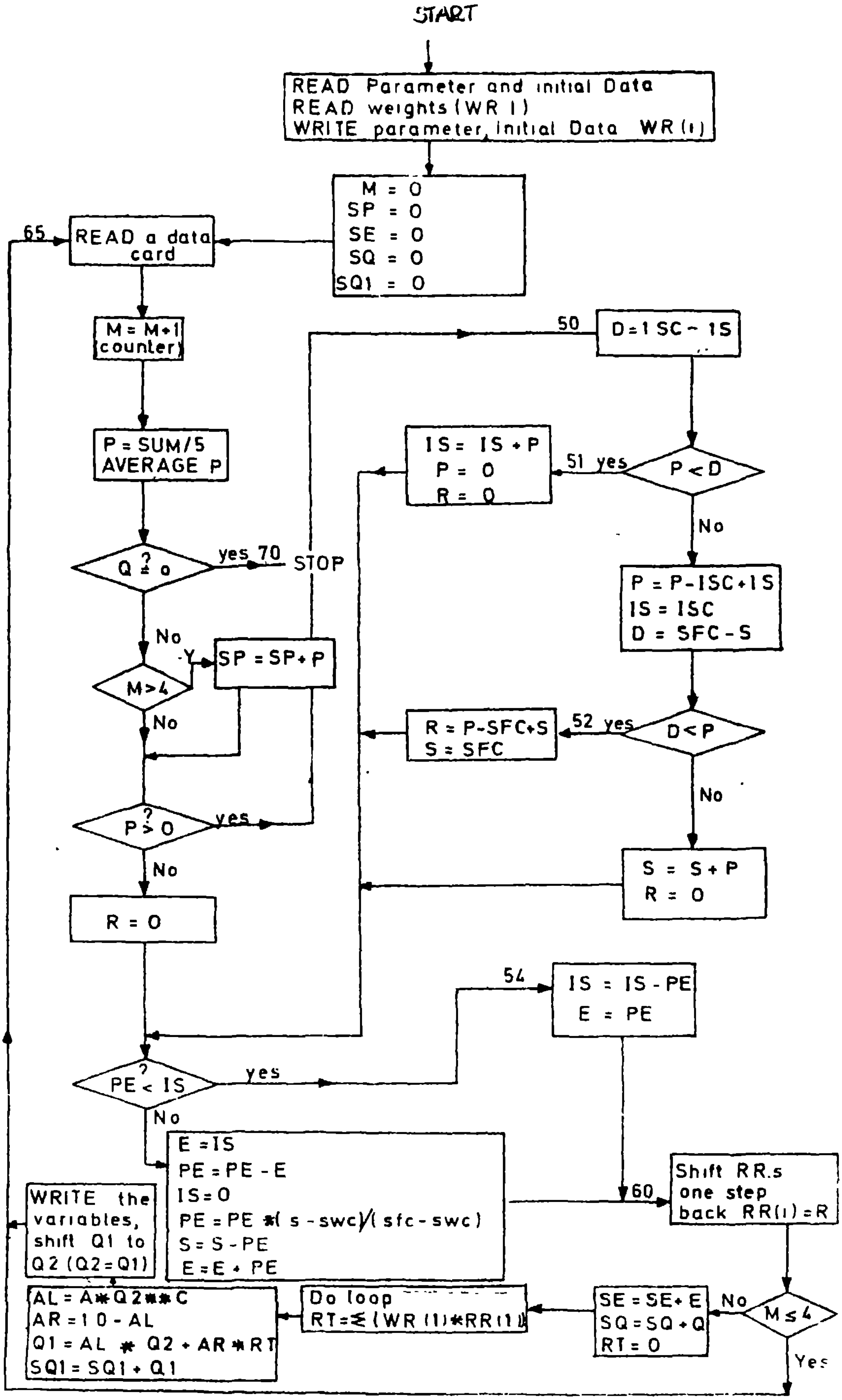


Figure 15. The flow chart.

When the observed and simulated hydrographs are compared, it is found that the latter reacts after a few pentades.

The obtained results are rather poor mainly due to:

- the data from the rain gauge network may to a certain damage be non-representative

- the catchment is too big (2 920 km^2) and the estimated set of parameters may not be truely representative values for the whole area.

3.2.3 BRALUP Method

Wingard and Riise /3/ used partial regression analysis for synthesising hydrographs from rainfall data. This was used as a case study of Kilombero River Basin which is a part of the Rufiji River Basin, potentially one of the most important water resources in Tanzania. The area of the catchment to the gauging site is 33 400 km^2. The results were not satisfactory due to

- too large a catchment area

- at the time of their study only four rainfall stations covered the whole of this area

- due to the pattern of scattered tropical rain storms this is obviously not an adequate network.

4. CONSTRUCTION MATERIALS

4.1 Introduction

Theoretically earthdam embankments can be designed in such a way that soils of any type can be used. Practically, highly organic fibrous soils (e.g. peat) are not chosen because of low shear strength and high compressibility. Because of the construction difficulties, inorganic clays of very high plasticity are not desirable except where no other materials are available, or in regions where water content control is not excessively difficult /22/.

4.2 Definitions

4.2.1 Soil

In the engineering sense, soil is any naturally occuring loose or soft deposits forming part of the earth's crust, particularly where they occur close enough to the surface of the ground to be encountered in engineering works, but excluding topsoil. The term covers such deposits as gravel, sands, silts, clays and peats. It should not be confused with the agricultural or pedological soil which embrace only the topsoil and subsoil. Pedological soil may come within meaning of the word in the engineering sense when it is excessively deep as in some tropical soils such as mbuga.

Subsoil is the weathered portion of the earth's crust that lies between the topsoil and the unweathered material below.

Topsoil is the superficial skin of the deposits forming the earth's crust that has, by processes of weathering and the action of organic and other agencies, been transformed into material capable of supporting plant growth. The term thus embraces the upper or humus bearing horizons of the soil of pedology./2/

4.2.2 Rocks

In engineering rock is hard and rigid deposits forming part of the earth's crust. Example are sandstones, limestones, metamorphic formations and igneous masses. Geologists define rock as any naturally occuring deposits be they hard of soft, but excluding topsoil /2/.

4.3 Major Soil Groups (Unconsolidated Sediments)

The major soil groups which may be encountered in the region under study are: Marine clay, marine and beach sands, estuarine muds and silts, alluvium, lacustrine sediments and pyroclastic deposits.

4.3.1 Marine Clays

a) Deposition: Slow settling in relatively deep slow-moving saline water, usually very flocculant, with high moisture content when first deposited.

b) Mineralogical and mechanical composition: Mostly clay minerals with fine detrital quartz, sometimes calcite and/or sulphides. Usually clay content is more than silt, whereas silt content is much more than sand (i.e. clay > silt > sand). Shaley bands may be present and bituminous organic matter can be an important constituent. Tecture clay or silty clay, class CH.

c) Characteristics:
Swelling, plasticity and cohesion, strength dependent on whether or not pre-consolidated. Hydraulic conductivity very low, no clear water table but surface water in humid climate in absence of artificial drainage. Topography generally low with low amplitude.

4.3.2 Marine and Beach Sands

a) Deposition from rapidly moving water or on beaches by wave action where often transitional, to coastal dunes. An example is modern beach sands.

b) Mineralogical and Mechanical Composition:
Mostly detrital quartz, sometimes with iron oxides, calcite etc. The sand content is much more than silt and clay. There are virtually no stones but iron oxide concentrations may occur. Texture sand, class SP but beach sand may pass into shingle, marine sands are usually better graded than beach sands.

c) Characteristics:
No plasticity or cohension. Hydraulic conductivity generally very high although cemented layers may occur; water table well defined if present.

4.3.3 Estuarine and Coastal Muds and Silts

a) Deposition: in water moving at low velocity, saline or brackish.
Example: estuarine and deltaic marshes and tidal flats, e.g. mangrove swamps.

b) Mineralogical and mechanical composition:
Sand and silt fractions mainly detrital quartz with variable accessories such as muscovite, iron oxides. calcite; often black iron sulphides below surface.

Clay minerals variable according to source area. Particle size very variable: silt and very fine sand usually dominant, coarse sand virtually absent. Textures typically silt loam, silty clay loam, fine sandy loam etc, classification ML, possibly CL, with variable grading.

c) Characteristics:
Plasticity, cohesion and swelling variable according to clay content and state of dispersion. Hydraulic conductivity intermediate, may fall if clay disperse after reclamation.

4.3.4 Alluvial Deposits

a) Deposition: from water no longer moving fast enough to keep the load in suspension. The main circumstances are:

- from the overflow of modern perennial streams in their flood plains (fine or coarse according to rate of flow)
- from formely existing streams, evidenced by valley-side terraces or buried channels (coarse or fine)
- from seasonal torrents as alluvial fans or wadi-fall (coarse or very coarse)
- from seasonal sheet wash across broad gently sloping sediments into playas or mbugas (fine).

Examples: fine grained alluvium of major rivers such as the Nile, alluvial fans and wadi-fills of North Africa etc, mbugas of East and Central Africa.

b) Mineralogical and mechanical composition:
detrial quartz, feldspars, micas, iron oxides, calcite, clay minerals etc, very variable according to terrain drained. Particle size depends on both source area and speed of water flow, texture ranging from silty clay to sand or gravel, classification and grading likewise variable.

c) Characteristics:
plasticity, cohesion and hydraulic conductivity depend mainly on local particle size and cannot be generalized.

4.3.5 Lacustrine Sediments

a) Deposition: in fresh or variably saline water but often with seasonal change in rate of sedimentation giving rise to varves. An example in the alluvial and pyroclastice sediments in lakes of the African Rift Valley.

b) Mineralogical and mechanical composition:
Sand grades mostly quartz and other detritals, sometimes organic or inorganic clacite present; gypsum in saline lakes and volcanic glass in active areas. Clay minerals also are very variable according to source area; unusual clays such as palygorskite may form in very saline water. Particles mostly fine; textures silty clay, silty clay loam etc, often with large organic component; classification Ml, CL, OL, MH, OH.

c) Characteristics:
plasticity, cohesion and swelling variable but not usually very low; poor bearing capacity in locally moist areas.

4.3.6 Pyroclastic Deposits

a) Deposition: volcanic ash and cinders setting on land or in water bodies. Examples: in vicinity of most volcanoes e.g. African Rift Valley, pozzolanas.

b) Mineralogical and mechanical composition:
Volvanic glass, sometimes with recognisable minerals e.g. augite. Large fragments (cinders) or fine dust (ash) gives textures initially gravel or sand to silt respectively, but volcanic material weathers quickly in humid to loam or clay. Classification: ML to MH weathering to CH.

c) Characteristics:
Plasticity, cohesion and swelling negligible until material weathered when all may greatly increase. Hydraulic conductivity likewise high or very high initially, tending to fall with weathering.

4.4 Properties of Soil Components

4.4.1 Gravel and Sand

These are the coarse grained components of soil, which have the same engineering properties, differing mainly in degree. Well graded, compacted gravels or sands are stable materials. When they are not mixed with fines, they are pervious, easy to compact and little affected by moisture. Although grain shape and gradation, as well as size, affect these properties, gravels are generally more pervious, more stable and less affected by water than are sand, for the same amount of fines.

As sand becomes finer and more uniform, it approaches the characteristics of silt with corresponding decrease in permeability and reduction in stability in the presence of water. Very fine, uniform sands are difficult to distinguish visually from silt. However, dried sand exhibits no cohesion (i.e. does not hold together) and feels gritty in contrast to the very slight cohesion and smooth feel of dried silt. Dry silt feels like flour.

4.4.2 Silt and Clay

Even small amounts of fines may have important effects on engineering properties of the soils in which they are found. As little as 10 percent of particles smaller than 75 μm (BS No. 200 sieve) size in sand and gravel may make the soil virtually impervious, especially when the coarse grains are well graded.

Soils containing large quantities of silt and clay are the most troublesome to the engineer. These materials exhibit marked changes in physical properties with change of water content.

A hard, dry clay , for example, may be suitable as a foundation for heavy loads so long as it remains dry, but may turn into a quagmire when wet. Many of the fine soils shrink on drying and expand on wetting, which may adversely affect structures founded upon them or constructed of them. Even when the water content does not change, the properties of fine soils may vary considerably between their natural condition in the ground and their state after being disturbed. When the soil is excavated for use, as a construction material or when the natural deposit is disturbed, for example by driving piles, the soil structure is destroyed and the properties of the soil are changed radically.

Silts are different from clays in many important respects but because of similarity in appearance, they often have been mistaken one for the other, sometimes with unfortunate results. Dry, powdered silt and clay are indistinguishable, but they are easily indentified by their behaviour in the presence of water.

Silts are the non-plastic fines. They are inherently unstable in the presence of water and have a tendency to become "quick" when saturated. Silts are fairly impervious and difficult to compact. Silt masses undergo change of volume with change of shape (i.e. the property of dilatancy), in contrast with clay which retain their volume with change of shape (i.e. the proerty of plasticity). The dilatancy property, together with the "quick" reaction to vibration affords a means of identifying typical silt in the loose, wet state. When dry silt can be pulverized easily under finger pressure (indicative of very slight dry strength), and will have a smooth feel between the fingers in contrast to the grittiness of fine sand.

Silts differ among themselves in size and shape of grains, which are reflected mainly in the property of compressibility. The liquid limit of a typical bulky grained, inorganic silt is about 30 percent, while highly miceceous or diatomaceous silts, consisting mainly of flaky grains, may have liquid limits as high as 100 percent. The difference in quicking and dilatancy properties afford a means of distinguishing in the field between silts of low liquid limits (L) and those of high liquid limits (H).

Clays are the plastic fines. They have low resistance to deformation when wet, but they dry to hard, cohesive masses. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristics of clays. The small size, flat shape and type of mineral composition of clay particles combine to produce a material that is both compressible and plastic. The higher the liquid limit of a clay, the more compressible it will be when compared at equal conditions of pervious geological loading. Hence in the Unified Soil Classification System, the liquid limit is used to distinguish between clays of high compressibility (H) and those of low compressibility (L). Differences in plasticity of clay are reflected by their plasticity indexes. At the same liquid limit, the higher the plasticity index, the more cohesive is the clay.

Field differentation among clays is accomplished by the toughness test in which the moist soil is molded and rolled into threads until crumbling occurs, and by the dry strength test which measures the resistance of the clay to breaking and pulverizing.

Plasticity the most characteristic property of clay, is the ability of a material to change shape continuously under a stress and to retain the new shape when that stress is removed. It depends on the nature and particle size of the clay micelles and on the thickness of the water films between them. At low water contents only rigidly fixed water is present, surface attractive forces predominate and the clay is strongly cohesive and non-plastic. With increasing moisture, layers develop and at the plastic limit cohesion is sufficiently reduced for the plate shaped micelles to slide past each other, lubricated by the water films; at the liquid limit films are so thick that the clay becomes slurry with virtually no shear strength. The difference in moisture contents between the plastic and liquid limits (PL and LL) is the plasticity index (PI). The presence of organic matter can markedly raise the liquid limit.

4.4.3 Soil Shear Strength

Shear strength is a complex function, not yet fully understood, of friction between soil particles of all sizes which is dependent on the overburden, and cohesion between clay particles which is independent of loading. Shear failure may occur by rupture along a shear plane, or by flow, which in cohesive soils is plastic over the range of the plasticity index (figure 16) and which approximates to viscous above the liquid limit.

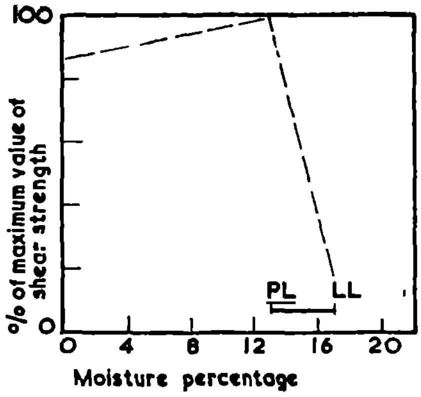


Figure 16. Shear strength of clay soil in relation to moisture content (after BAVER et al 1972).

The apparent viscosity of a slurry is the ratio of the applied shear stress to the rate of shear. Shear strength is related to clay mineral type and to exchangeable cations and is strongly influenced by the degree of compaction or bulk density as well as moisture content.

Soil shear properties can be markedly altered by remoulding. In wert granular soils and some flocculated clays working may increase the voids ratio, moisture films are reduced in thickness and strength rises. Such soils are termed dilatant, e.g. moist beach sands. More usually shear strength is reduced by working or shock although it may subsequently recover.

Sensitivity is the ratio of the shear strength of the soil in its original condition to that after remoulding at the same moisture content. Values range from less than 1 in "insensitive" soils to over 16 in "quick" clays, such as in some recent marine clays. Very sensitive soils may be subject to failure due to earth tremor or excessive vibration during construction.

Shear strength can be measured in the laboratory by shear box and other tests, but practical measurements can be made on site by empirical methods using shear vanes or con penetrometres.

Soil compressibility

Compressibility is the relationship between reduction of voids ratio (or increase in bulk density) and applied load. Compression of granular soils depends on the compressibility of the mineral grains, pore water pressures and the extent to which rearrangement of particles is possible. Adjustment to stress takes place relatively rapidly. In cohesive soils compression is slow and is often termed consolidation. The latter is importantly related to shear strength (fig 17).

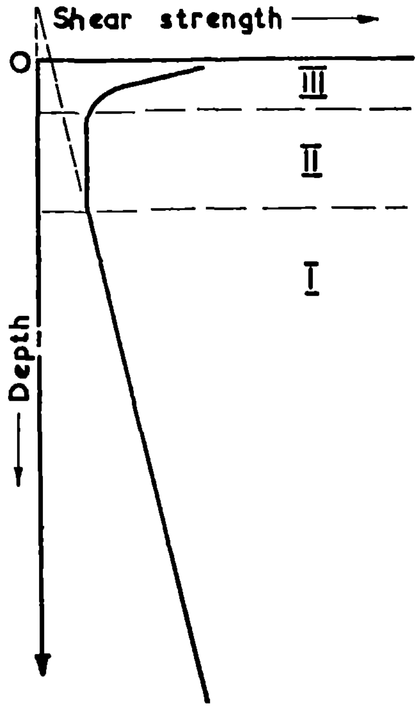


Figure 17. Changes in shear strength with drying and overburden of London clay soil (after Skempton and Northey 1952).

In saturated soils the stress is initially carried by pore water which builds up excess pressure. This is slowly dissipated by migration into adjacent soil, the rate depending on the hydraulic conductivity. Compressibility is also affected by the initial arrangement of the clay minerals. With increasing stress preferred orientation tends to increase and in the last stages of compression (secondary compression) the load is probably mainly carried by the swelling pressure of the clay micelles. Compressive behaviour is strongly dependent on clay mineralogy and exchangeable cations.

4.5 Soil Classification

There are various methods of soil classification. The method in use is the Unified Soil Classification System.

4.5.1 Unified Soil Classification System (UCS)

In this system soils are divided into two main categories, coarse grained and fine grained soils. These are defined as follows:

a) coarse grained soils are soils in which more than 50 percent by weight of a representative sample of the soil is retained on the 75 µm (BS No. 200) sieve

b) fine grained soils and soils in which 50 percent or more by weight of a representative sample of the soil passes the 75 µm sieve.

The results of a sieve analysis are needed to place the soil in one or other of these categories. The procedure for classification is then carried out under the appropriate category as described below.

Classification of coarse grained soils

The procedure for classifying coarse grained soils is outlined in diagramatic form in figure 18.

By definition the coarse grained soil particles stopped by the 5 mm sieve are gravels, and those passed are sand. The results obtained from a sieve analysis show whether the sample contains more gravel or sand, and this determines the first letter G (for gravel) or S (for sand) for the sample.

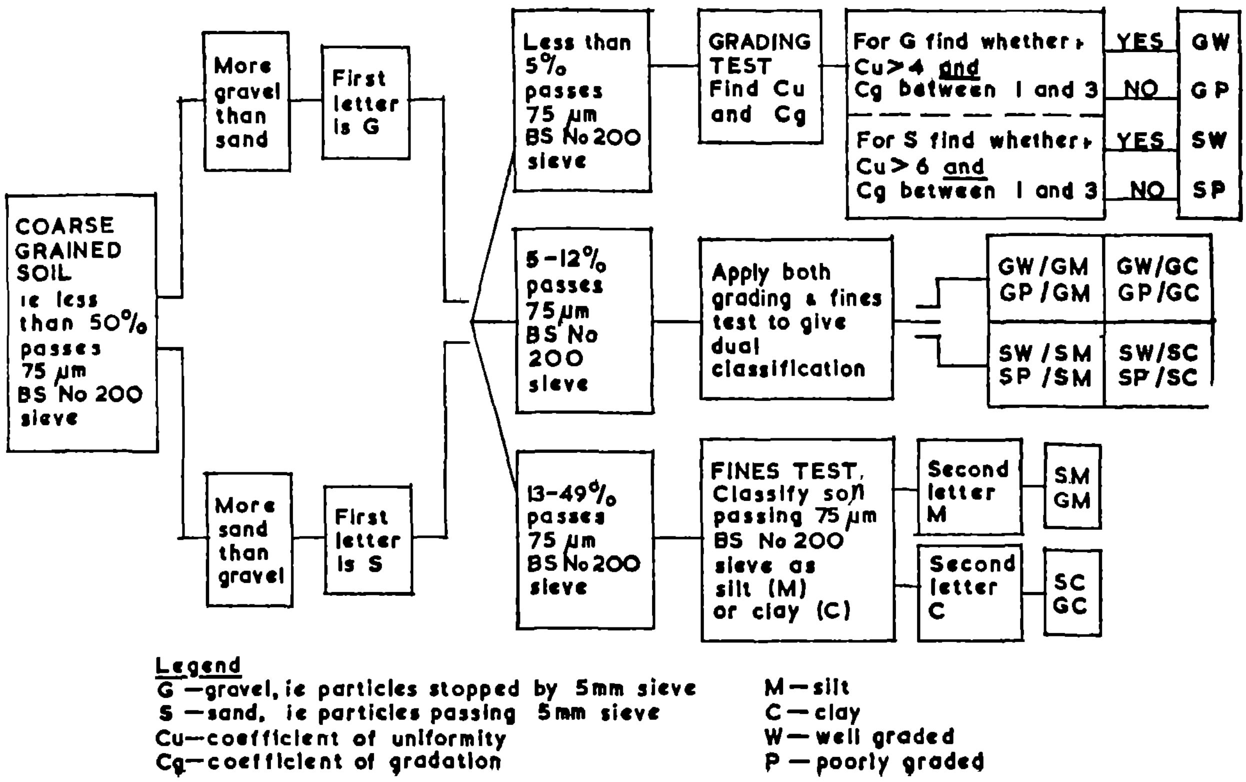


Figure 18. Unified classification system - coarse grained soils.

The sample is then placed in one of three cases depending on the percentage of the sample passing the 75 µm (BS No. 200) sieve as follows:

a) Case 1. Less than 5 percent of the sample passes the sieve.
b) Case 2. Between 5 and 12 percent passes the sieve.
c) Case 3. Between 13 and 49 percent passes the sieve.

The results of the sieve analysis enable the grading coefficients for the soil sample to be calculated. These are the coefficient of uniformity (Cu) and the coefficient of gradation (Gg) and are obtained from the formulae:

$$Cu = \frac{D_{60}}{D_{10}} \quad \text{and} \quad Gg = \frac{D_{30}^2}{D_{60}D_{10}}$$

where D_{60}, D_{30} and D_{10} are the particle size diameters at which 60, 30 and 10 percent of the soil passes.

The second letter of the classification is obtained as follows:

a) Case 1. Determine the values of Cu and Gg for the sample as described above and using figure 18 determine the second letter.

b) Case 2. Soils in this category are given dual classification as for case 1, and then as for case 3. The soil sample is then classified using both pairs of letter e.g. GW/GM.

c) Case 3. Classify the soil passing the 75 µm sieve by means of a fines test which will determine whether the fines are clay or silt and this will decide whether M (silt) or C (clay) is to be used.

Soils can also be placed into high (H) or low (L) liquid limit groups depending on whether they have a liquid limit of more or less than 50, and this can be shown by the addition of H or L to the classification group (see fig. 19).

The UCS allows the addition of a third letter to soils of types GM and SM to indicate the construction quality of the soil, as desirable or undesirable. If the liquid limit is 28 or less and the plastic limit 6 or less the letter d is added, i.e. GMd, whereas if the liquid limite exceeds 28 or the plastic limit exceeds 6, the sample is undesirable and the letter u is added, i.e. GMu.

Classification of fine grained soils (fines test)

The classification of fine soils is determined by comparison of the plasticity index and the liquid limit of the sample. The liquid limit, the plastic limit, and the plasticity index are determined by tests. The plasticity index is plotted against the liquid limit on the plasticity chart (fig 19) and the classification of the fine grained soil read off.

If the fine grained soil contains appreciable organic material, the soil is given the first letter O. The percentage of organic material should be quoted and the other constituents identified /2/.

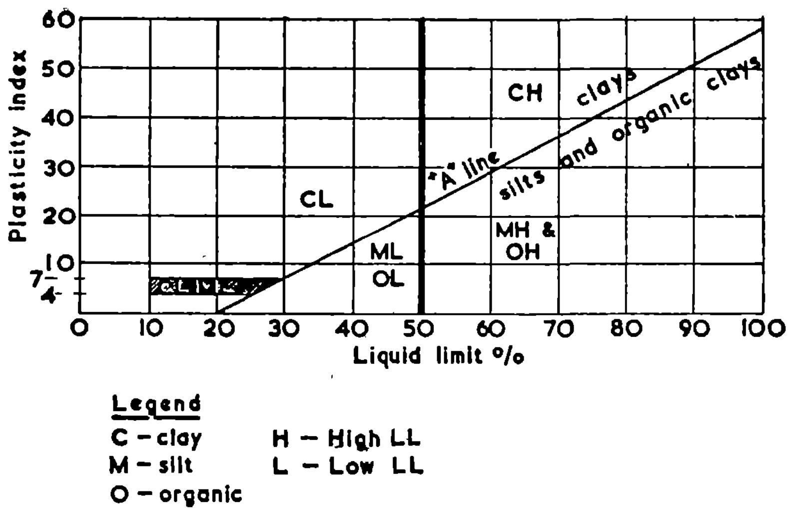


Figure 19. Plasticity chart (UCS).

4.6 Tropical Soils

In this section the discussion will be on the soils which are frequently encountered in the tropical regions, and which present special problems.

4.6.1 Laterite Soils

Laterite soils are tropically weathered materials rich in secondary oxides of iron, aluminium or both. They are diverse in their properties and exist as:

- red friable soils
- red clay
- hard or soft coarse angular red granula concretions
- laterite rock.

Thus laterite soils are more of a soil group rather than a well defined material. This is not the case with other soil types. They are among the major soil types of the tropics. They can be found in large areas of Malaysia, India, Africa and Central and South America.

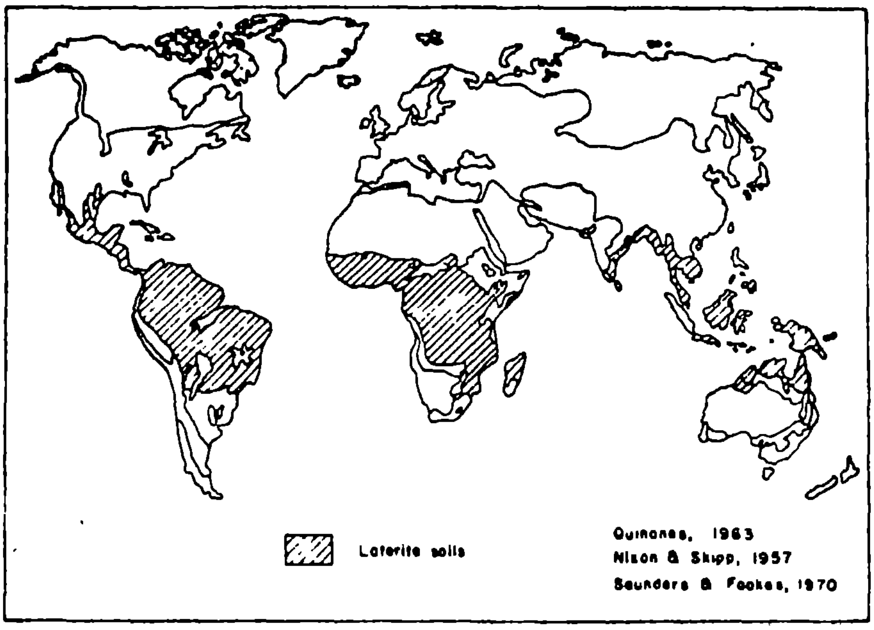


Figure 20. Generalized world map showing the distribution of laterite soils.

Laterite soils are formed from parent material rich in ferro-alumino silicates by a weathering process known as laterization. This is a process of alternate leaching and drying, whereby ferro-alumino silicate minerals are decomposed and sesquioxides are deposited in an acidic media. Silica is then leached away by alkaline soil solutions part of which may form complexes with the sesquioxides to accentuate the formation of concretionaries. Under non-tropical climates leaching of silica does not take place. Laterization is encouraged by:

- amount and kind of vegetation cover and organic matter and humic residues formed
- permeable profiles
- hot climates with coolish nights
- heavy rainfall and high humidity
- a combination of alternating leaching by acidic solutions and alkaline solutions
- fluctuating water table
- parent material containing ferro-alumino silicates.

During the process of laterization silica (SiO_2) is leached away and sesquioxides of iron and aluminium (Fe_2O_3 , Al_2O_3) are deposited. This implies that the more advanced the laterization is, the greater the amount of sesquioxides which will be deposited. Thus the degree of laterization may be assessed by the silica-sesquioxide ratio. The lower the ratio the more advanced is the laterization.

Identification of laterite soils

A typical laterite soil profile will consist of the following horizons:

Horizon	Description
A	zone of leaching
B	zone of accumulation - laterite horizon
C	zone of weathering and removal of soluble constituents
D	sound parent material

A great variety of soils may overlie the laterite horizon or underlie it.

The laterite horizon may be found on the surface due to erosion of the overlying soil. In dry areas or regions of good aeration the underlying soil is bright red in colour. In low altitudes the underlying soil is mottled or bright coloured.

Characteristics of laterite soils

Laterite soils have quite unique characteristics which distinguish them from other soils. This is due to their genesis. The dominant effects are:

- leaching which develops high void ratios
- cementing and coating effect of sesquioxides
- high specific gravity of the iron oxide minerals
- red colour of the iron oxide.

Colour

Due to the presence of iron oxides, laterite soils are red in colour ranging from light, through bright to brown shades.

Hardening

Laterite soils may be soft at the time of excavation but harden upon exposure. This is because the sesquioxide minerals which coat and cement together soil grains harden by dehydration and recrystallization.

Low insitu density

Laterite soils are characterized by high void ratios because of the genetical leaching effect. This situation gives rise to low insitu densities. However, at high degrees of laterization the accumulated sesquioxides fill up the voids to raise the insitu densities again (e.g. laterite rock).

High insitu permeability

Because insitu laterite soils are characterized by high porosity they are of relatively high permeabilities. This also is due to leaching.

Insitu high strength

Due to cementing effect of sesquioxides laterite soils have relatively high shear strengths.

High specific gravity

The specific gravity of iron oxides is higher than those of common soil grain minerals. Thus their presence in the soil matrices raises the specific gravity of the latter. The more advanced the laterization, the higher will be the specific gravity of the soil grains above the parent material. Specific gravity for laterite soils varies between 2,5 and 4,6.

Classification of laterite soils

To facilitate engineering analyses in practice, soils are classified. There exists several classification systems, but the most widely used is the Unified Soil Classification System (UCS). The soil classification is based on the grain size distribution and the plasticity of the fines of a soil sample. In order to determine these quantities the insitu structure of the soil has to be disturbed.

Some of the unique characteristics of laterite soils mentioned above like low insitu density, high insitu permeability and high insitu shear strength are for undisturbed state of soil. For this reason they would not be reflected by the normal classification tests. In fact the plasticity and grain size gradation suggest that the insitu laterite soils would be more impermeable, more compressibe and of lower strength than is actually the case. Thus to characterize the engineering potential of insitu laterite soils emphasis is put on tests on undisturbed samples and field tests.

For use of laterite soils in a disturbed form, e.g. compaction, classification tests stil need to be supplemented by tests on durability of coarse particles (gravel and sand) and plasticity characteristics of the fine grained fractions of gravels and sands.

Variability of the properties of laterite soils

Properties of laterite soils are highly variable and often can be erratic. Texturally they may contain: boulders, cobbles, gravels, sands, silts or clays. This situation results from the differences in:

- nature of parent material
- mode of formation (genesis)
- degree of weathering:
  - -- decomposition
  - -- desiccation
  - -- leaching
- position of sample in topographic site
- sample depth in profile.

Problem laterite soils

Particle size analysis and plastic limit tests can be carried out satisfactorily on most laterite soils by standard laboratory procedures. These soils are referred to as normal laterite soils. With some laterite soils the standard tests do not yield reproducible results. These soils are referred to as problem laterite soils. This situation is due to large influences of pretest preparations and test procedures.

Pretest preparation is affected by the following:

- removal of cementing free iron oxides (e.g. micro clusters)
- sensitivity to:
  - -- degree of remoulding
  - -- degree and type of drying (e.g. dehydration of iron oxides)
  - -- time of mixing
- dehydration of weak modular materials.

For normal stable laterite soils the natural moisture content is generally less than the plastic limit. With the problem laterite soils natural moisture content may be as high as the liquid limit in high rainfall forests. Problem laterite soils have been indentified as soils on:

- recent volcanic areas
- areas of continuous wet climate > 1500 mm/a.

They are characterized by:

- high natural water content
- high liquid limit
- low natural density 0.3 - 1.2 g/m^3
- low compressibility.

They can be evaluated on the basis of

- inconsistent particle size distribution
- inconsistent plasticity characteristics
- potential smell
- self stabilization.

Engineering properties of laterite soils

Shear strength

This is the major property of soils as it provides supporting ability and stability of slopes. The higher the degree of laterization the more favourable are shear strength parameters.

Bearing and penetration characteristics

(The difference between the tow is a matter of scale.) Because of erratic structure field penetrometer tests are most useful.

Compression strengths for concretionary laterite rocks and pisoliths:

- It may be satisfactory with hard concretionary particles.
- It may be very questionable with friable and weak concretionary particles.
- More reliable tests are:
  - -- aggregate impact value test
  - -- aggregate abrassion test
  - -- aggregate crushing value test
- specific gravity is a reliable index for strength.

Compressibility and permeability

- with fine-grained residual laterite soils the rate of consolidation is high
- permeability is generally higher for undisturbed porous, residual soils and lower for compacted soils

Compaction characteristics

The object of soil compaction is to reduce the sensitivity of strength and volume changes to environmental changes, especially those affected by moisture. The compaction process increases the strength and bearing capacity, and reduces the compressibility and the permeability. Compaction characteristics are influenced by processes of laterite soil formation and pretest preparations.

Stabilization of laterite soils

Laterite soils may be stabilized if need be. This can be effected by:

- cementing agents - cement, lime
- modifier agents - cement, lime, bitumen.

Due to high content of sesquioxides laterite soils use only small amounts of cement and lime (4 - 7 %) due to reaction of lime with iron and aluminium oxides. Certain types of laterite clays are unsuitable for stabilization with either lime or cement.

Field performance of laterite soils

Field performance depends on the content of sesquioxides. The higher the iron oxide content the higher the strength.

Embankments and earthdams

- Residual compacted soils are resistent to piping by seepage.
- A major design problem is to avoid piping by cracking of some compacted laterite soils.
- Laterite soils may be used for both homogenous as well as zoned cross-sections.
- In spite of their low permeability after compaction, their consolidation coefficients are usually fairly high.

Stability of slopes

- The depth of wetting is a controlling factor for stability of slopes in residual laterite soils.
- The most practical method of ensuring stability of slopes is to limit the depth of wetting by preventive infiltration.
- Landslides are normally confined to 3 - 6 m below the surface.
- Analytical design of cuttings and natural slopes in laterite soils are often impossible.
- Empirical design methods based on the systematic collection of slope performance data provide a logical alternative.
- Erosion resistance is influenced by soil properties and may be improved by spraying resistance agents - cement, lime, bitumen etc.

4.6.2 Black Cotton Soil

Black cotton soils are highly expansive clays found in the tropics. Potentially expansive soils can be found in the semi-arid regions of the tropic and temperate climate zones. They are abundant where evaporation exceeds precipitation and lack of leaching aides formation of montmorillonite clay mineral.

Expansive soils have been found in North and South America, Africa, Middle East, India, Australia etc. Expansive soils can cause considerable damage to engineering structures. For instance, in the USA (1973) the expansive soils damages exceeded the combined average annual damages from floods, hurricanes, earthquakes and tornadoes.

Formation

The expansive property of the black cotton soils (and other expansive soils) is attributed to the high content of montmorillonite clay mineral which causes large volume changes on wetting and drying. Montmorillonite results from decomposition of feldspar and pyroxene minerals of parent materials under alkaline conditions due to poor drainage. Expansive soils have been found over both igneous and sedimentary rocks.

- Igneous rock:
  - -- basalts Deccan plateau in India
  - -- dolerite sills and dykes in South Africa
  - -- gabbros and norities in South Africa

- Sedimentary rocks:
  - -- shale in North America
  - -- marls and limestones in Israel
  - -- shale in South Africa

Black cotton soils are black in colour because of the presence of notronite, a montmorillonite mineral in which ferric ions predominate. The soil is formed under "reducing" condition whereas under "oxidation" conditions ferrous oxide predominates and makes the soil red.

Characteristics of expansive soils

Swelling

Montmorillonite clay mineral expands as it absorbs moisture. It can expand upto 400 %. Thus clays with a high proportion of montmorillonite clay mineral will swell when moistened.

Moisture content

The degree of expansion or volume change will depend on the range between the initial moisture content and final moisture content. Thus no volume change for no moisture variation. Maximum change takes place between dry state and saturated state.

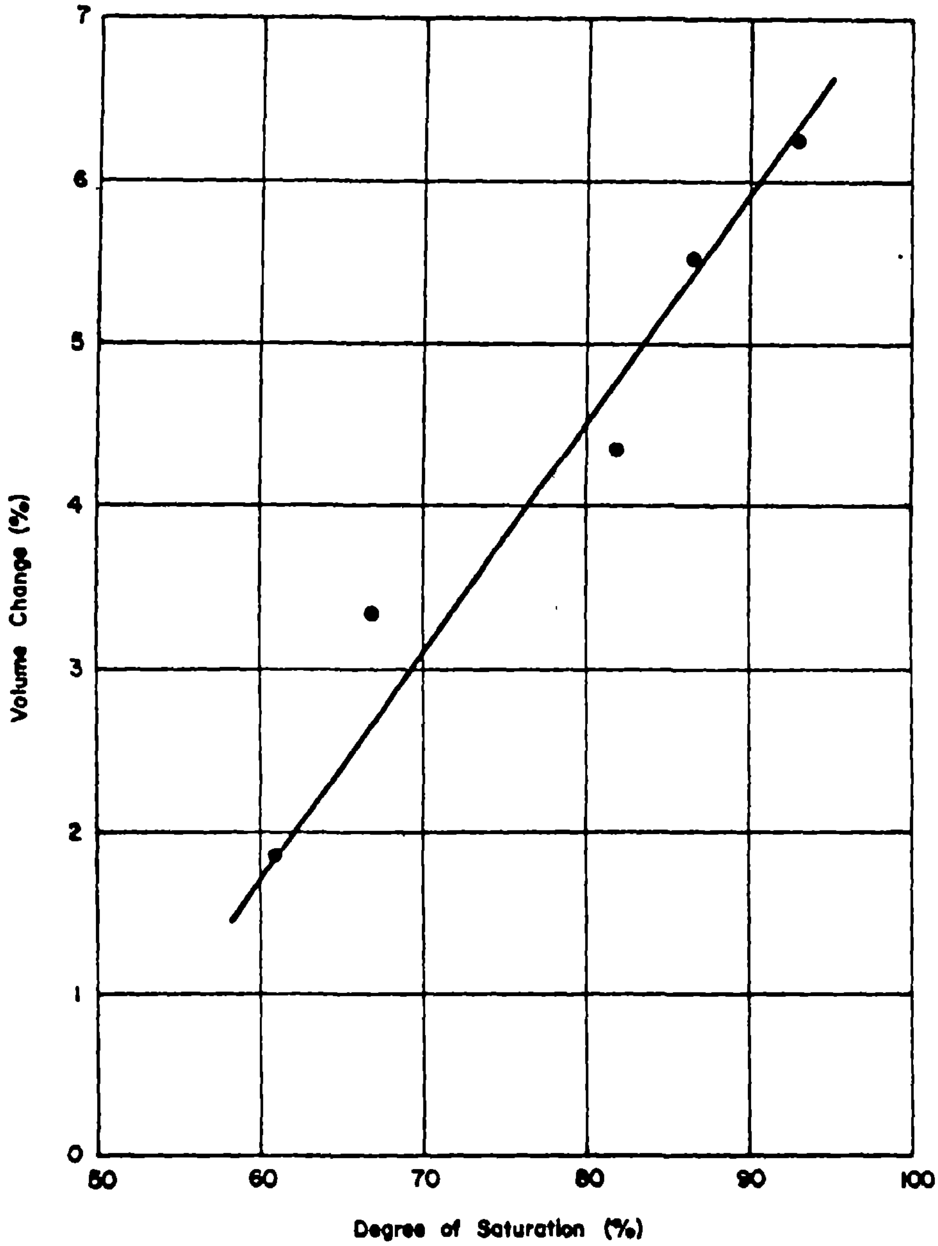


Figure 21. Effect of varying degree of saturation on volume change for constant density and moisture content sample.

Shrinkage

Shrinkage is a mirror image of swelling.

Thickness of stratum

The amount of swelling is also influenced by the thickness of the stratum of the expansive soil. The greater the thickness the bigger the volume change.

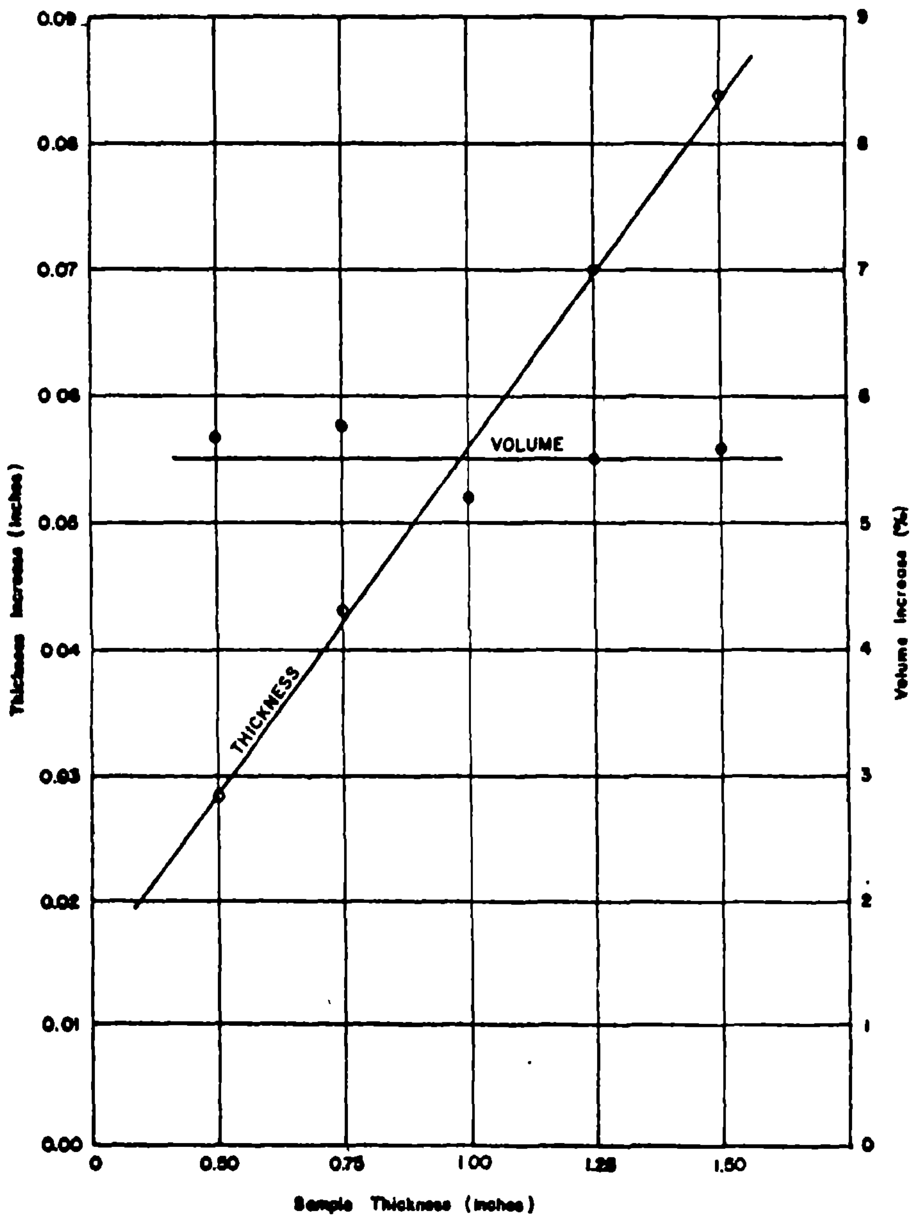


Figure 22. Effect of varying sample thickness on volume change for constant density and moisture content samples.

Dry density

Amount of swell varies with the dry density. Thus the higher the dry density the greater will be the swell potential. Clays with dry densities in excess of 1,75 g/cm^3 are of high swelling potential.

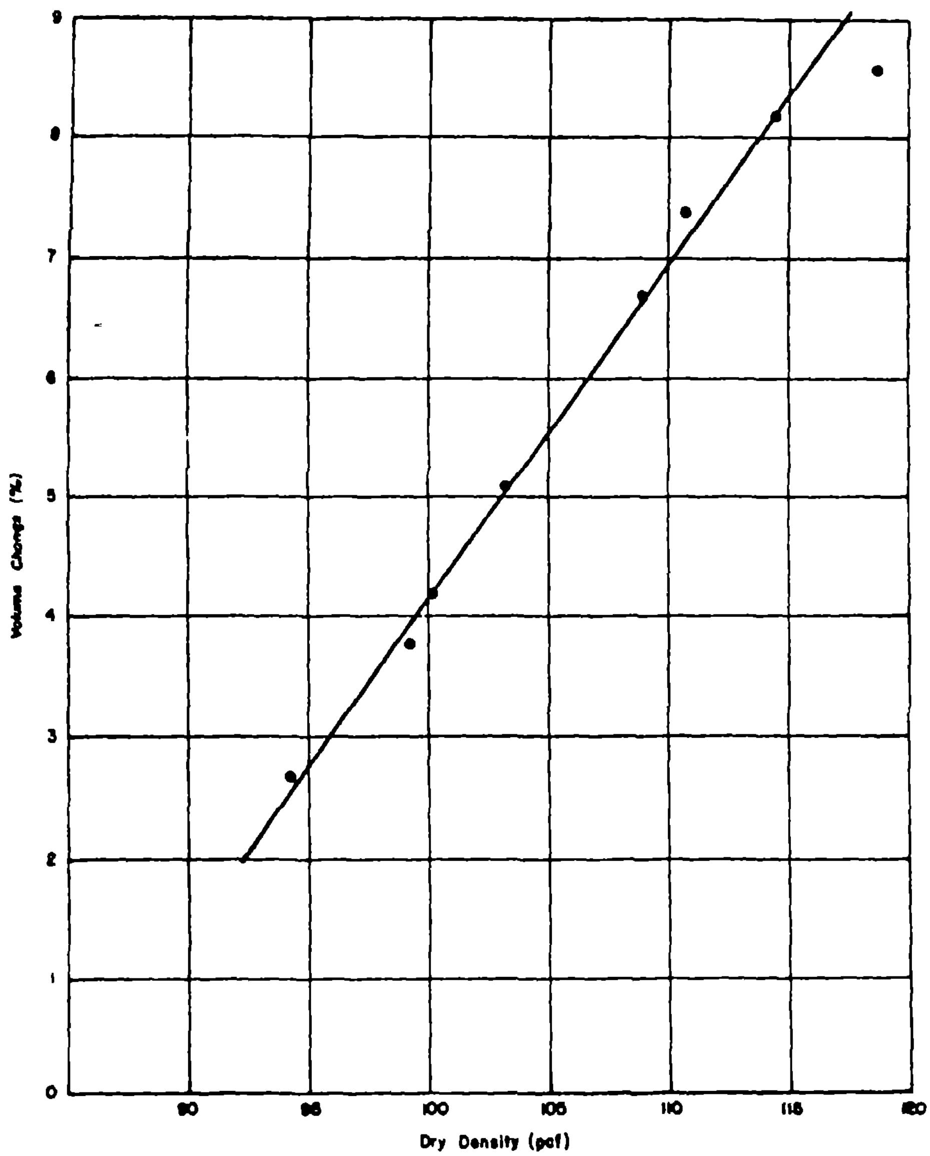


Figure 23. Effect of varying density on volume change for constant moisture content samples.

Swell fatigue

When an expansive clay is subjected to several cycles of wetting and drying from an initial moisture content, the amount of expansion will get less with each cycle asymtotically. This phenomenon is known as swell fatigue.

Swell pressure

When an expansive clay swells it exerts a pressure known as swell pressure. This is defined as the pressure required to keep the volume of a soil at its dry density constant i.e. at zero volume change.

Swell pressure is built in property constant for a given dry density. It increases with the increase of dry density. It is not affected by placing condition such as:

- surcharge pressure
- initial moisture content
- degree of saturation
- thickness of stratum.

Identification of black cotton soils

Black cotton soils may be identified by their characteristic black colour and by the following means:

- mineralogical identification
- soil index properties
  - -- potential volume change (PVC)
  - -- activity
  - -- Alterberg limits
  - -- free swell
  - -- colloid content
- direct measurement - most useful

Generally these methods should be used in combination. A guide for estimating swell potential is presented in table 7.

Table 7.

Swell potential	Probable expansion %	Liquid limit %	Plasticity index	< 2m fines %	Standard penetration test (SPT) blows/ft
Very high	> 10	> 60	35 - ≤	> 95	> 30
High	5 - 10	40 - 60	20 - 55	60-95	20-30
Medium	1 - 5	30 - 40	10 - 35	30-60	10-20
Low	< 1	< 30	0 - 15	< 30	< 10

Engineering properties of black cotton soils

In general, black cotton soils possess poor engineering properties. They are difficult soils to deal with. They should be avoided if possible.

The major engineering properties are that they are:

- impervious
- of low shear strength
- of high compressibility when compacted and saturated
- poor workability
- expansive property accompanied by swell pressure cause damages to engineering structures.

4.7 Procedure for Soil Investigation

The usual practice is that the team from the soils laboratory goes to the site right after the selection of the site. In the current practice, preliminary designs are carried out without any soil investigation, and often the final designs also. Then the soil investigation is done after the plans have been approved. This is quite contrary to standard practice, as this leads to many assumptions in the designs. Further, construction material might not be available at an economical distance, but since at this stage the plans are already approved, the construction must be carried out. Also some regions dig test pits at the site and send samples to the soils laboratory for testing. This is also contrary to standard practice, as only the soils engineer knows where to dig these pits. It is recommended that the field identification and laboratory testing be carried out by staff from the soil mechanics laboratory /15/. This will greatly assist the designer and save costs.

4.8 Selection and Operation of Borrow Areas

The borderline between borrowing and quarrying is not well defined, and quarries may also be opened for local usage. The term "borrow pit" is normally restricted to local excavations where blasting is not required.

Borrow pits are local sources of supply for soils and naturally occuring aggregates required for construction works. They may range in size from the small pits a few metres square, dug by hand along dirt roads to supply materials for repairing potholes, upto the very large pits worked by bulldozers and scrapers used to obtain the vast supply of materials needed for earth fill dams. For example at Kainji Dam, Nigeria, about 20 million tons of potential suitable borrow material was proved during the site investigations /2/. The design of these large structures depends upon adequacy of local supplies of suitable, easily won, borrow material. On the other hand, transport routes are usually designed to ensure a balance between cut and fill sections in order to minimise needs for handling imported fill and surplus excavation of suitable material. The maximum economic depth of excavation in borrow pits is related to the scale of the operation and to surface slopes. Generally the depth below access should not exceed about 6 metres.

Borrow pits should be selected on the basis of their content of suitable material, distance from the worksite, drainage problems and interference with other usage. They may be located in the reservoir area, and this is good for environmental reasons. This may also increase the reservoir volume.

Suitable deposits are often covered with surface soil which must first be stripped. The site investigations should include an estimate of the stripping quantities and the availability of a stocking area for the overburden which may be used later to restore the ground level so far as practicable, to landscape the area as to assist stability.

5. CONSTRUCTION OF EARTHDAMS

5.1 Introduction

In the preceding chapters poor construction control has been cited as one of the major causes of earthdam failures in Tanzania. The majority of dam failures which occured in Arusha region during 1977-1978 have been attributed to poor construction control, although hydrometeorological factors are also to blame.

VBB /25/ also report that the investigations on the failure of Wiyenzele Dam in Dodoma region and Itamuka Dam in Singida region revealed some construction deficiencies although these were not the main cause for the said failure.

The construction process usually starts with preliminary works. These are the construction of access roads, labour camp with all necessary accessories, diversion works etc. After the preliminary works then follows processing of construction materials, foundation preparation, embankment filling and finally slope protection. The spillway excavation is scheduled so that material usable on the embankment can be used directly without too much stockpiling. Incase the material is not usable then it has to be dumped away according to the specifications of the works.

5.2 Preliminary Works

The site has to be accessible from the main road and as such the preparation of the access road is the first operation. Care should be exercised in selecting the route so that excessive slopes and excavation or filling are avoided. Other than clearing and levelling there is usually no need of extra material on these temporary roads. However, some temporary culverts capable of carrying heavy equipment may be necessary and this has to be designed accordingly. In Tanzania, most access roads to damsites use drift culverts on which the floods pass as well as trucks. These provide a cheap alternative to any other type of culvert.

Then the labour camp has to be constructed. These are usually temporary structures which can be pulled down after the construction period. In Tanzania, the labour camp usually consists of single units of prefabricated huts or uniports. These are easy to erect and also to pull down.

Corrugated iron sheets are also used in a timber frame to make site camps, although these can be reused very few times after which they have too many holes. Relatively comfortable accomodation should be provided to the workers as a incentive to work in surroundings which they are little used to. The camp site has to be well selected so that water can be obtained either by pumping from the river or a shallow well or from a water bowzer. Sanitation of the site is of paramount importance, hence some type of communal pit latrines have to be constructed as well as workshops for servicing equipment.

For the small to medium size dams under discussion, economy dictates that expensive diversion works be avoided. In the case of seasonal rivers, usually the construction of a cofferdam is sufficient to keep the construction site dry. However, in case of a large flood, it may be necessary to make a temporary channel through the embankment to pass the same. In the case os perennial rivers it may be necessary to start construction on the abutments and complete the middle portion during one dry season.

As this may not be feasible and there may be other users downstream with water rights, some way of maintaining a certain flow has to be adopted. In such circumstances, the outlet works may be designed to pass the dry weather flows or that sufficient to cover the abstraction rights downstream whichever is greater. This implies that the outlet cum diversion works have to be completed before any impoundment is envisaged.

5.3 Foundation Preparation

The purpose of this is to ensure that the foundation will bond properly with the embankment material, and to reduce or eliminate subsurface seepage.

5.3.1 Foundation Excavation

Based on preliminary investigations a planned programme of foundation excavation is initiated after setting out the embankment spread. It is necessary to see to it that the volume of excavated material and the configuration of the excavation will reasonably approximate estimates in the plans. It is necessary precaution to establish slopes for excavations that will be permanently stable or that will not fail during construction.

In foundations in unconsolidated natural deposits, excavation may reveal inadequate localized or widespread foundation materials that require special treatment or total removal. Such material may be unconsolidated materials rich in organic substances such as topsoil. Swamp muck or peat, loose deposits of sand or silt and plastic, active, sensitive or swelling clays. Poor foundation conditions in rocks are associated with fracturing, weathering or hydrothermal alteration, or poorly indurated sedimentary rocks /27/.

Excavations in bedrock should to the extent possible extend into firm, fresh rock. The main idea is to prepare a clean surface that will provide an optimum contact with the dam materials that will be placed on it.

5.3.2 Foundation Treatment

Ideally, excavations in unconsolidated deposits for a dam should extend to solid bedrock for the full width of the dam. However, there are many locations where the depth of the valley fill is so great that dams must be constructed in part or entirely on unconsolidated deposits, and as required appropriate steps are taken to reduce subsurface seepage to premissible levels.

Figure 24 shows cross-sections of several earth and/or rockfill dams constructed at least in part on unconsolidated subsurface deposits. The sections show various measures that are taken to eliminate or greatly reduce potential seepage beneath the dam under a variety of circumstances /27/.

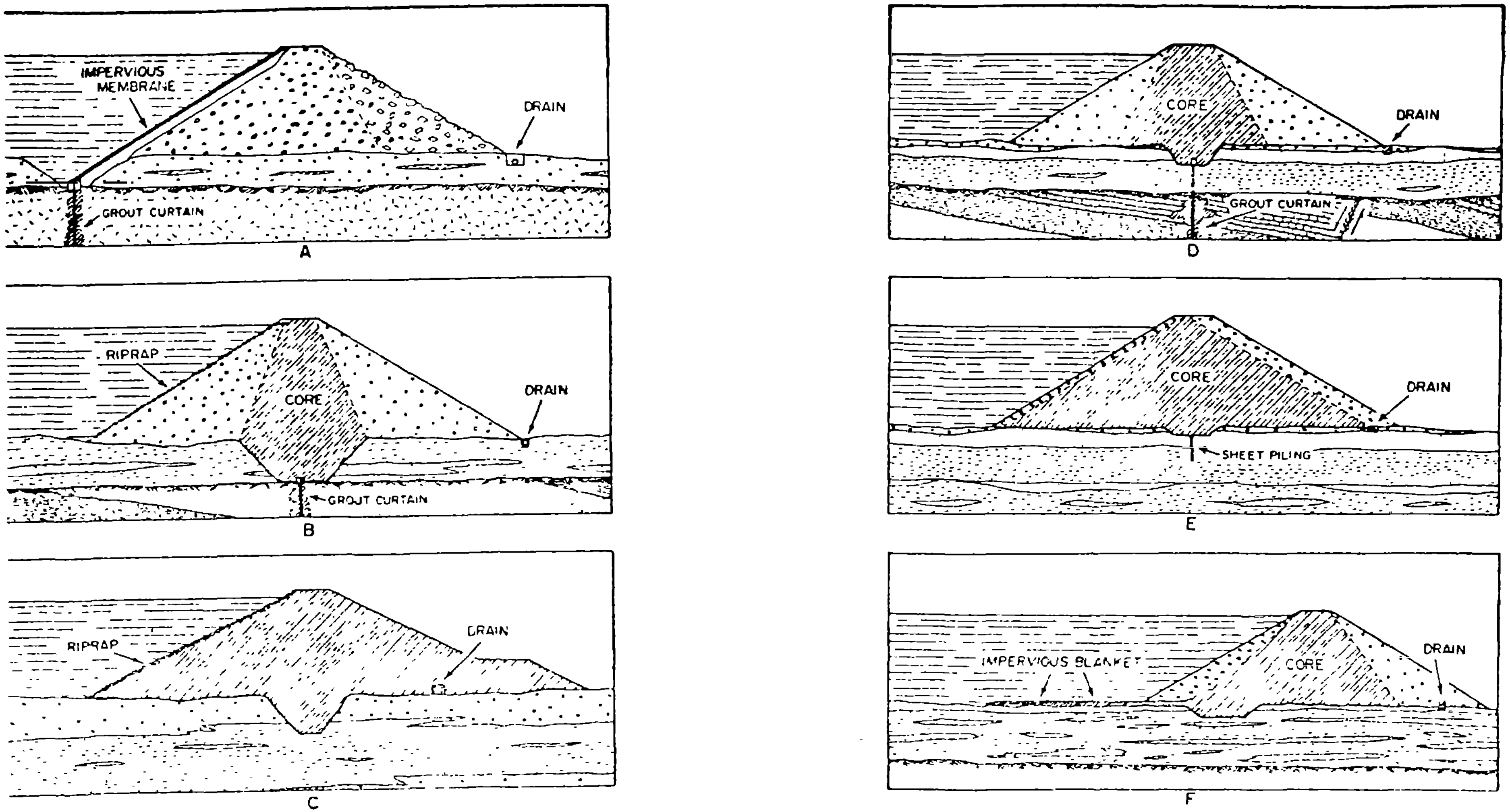
In a fractured foundation rock, the general method employed for sealing the channels is grouting. The usual technique utilizes drilling and pressure grouting with water-cement mixtures or with other types of sealants. In dam foundations three kinds of grouting programmes are identified /9/27/:

1. Comparatively shallow systematic "blanket" or "consolidation" grouting over critical portions of the foundation.

2. "Curtain" grouting from a gallery or concrete "grout cap" along a specified location to provide a deep impermeable barrier to subsurface ground water seepage.

3. "Off-pattern" special purpose grouting to improve strength and/or to overcome problems created by ground water circulation in zones identified by geotechnical studies.

Figures 25 and 26 illustrate the use of grouting in earth dam construction.



a

b

Figure 24. Foundation treatment methods.

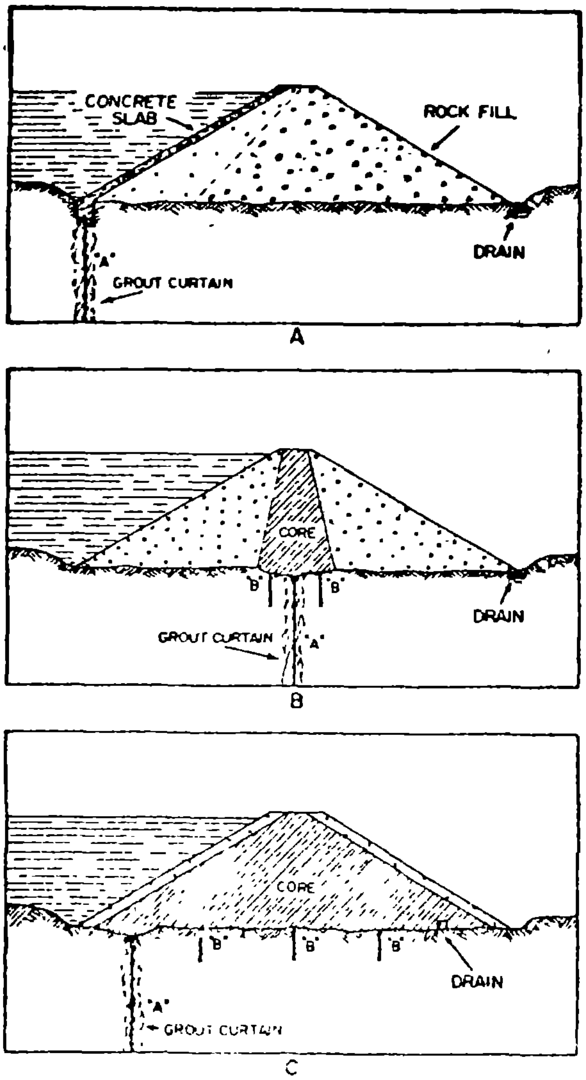


Figure 25.

Some cross-section of dams with rock foundations showing locations of drilled holes for foundation treatment.

a) rock-fill dam with impermeable concrete face
b) zoned earth and rock-fill dam
c) zoned earth and rock-fill dam.

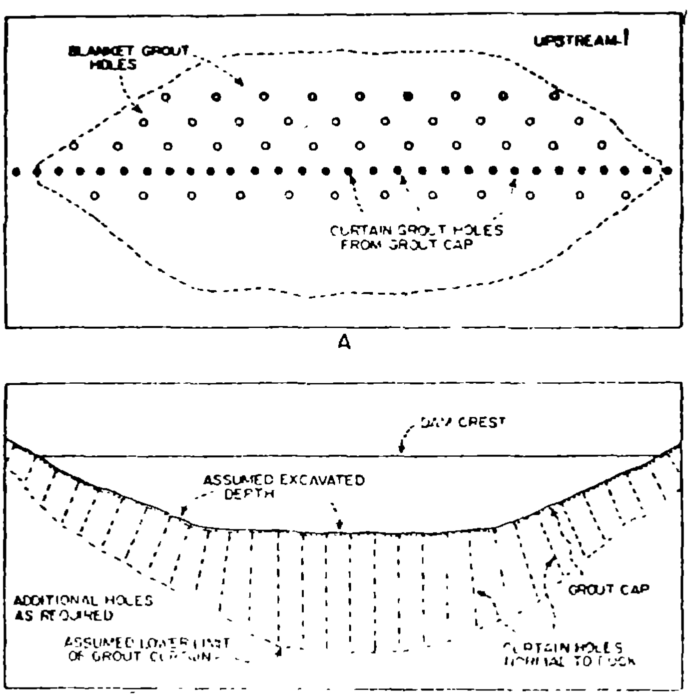


Figure 26.

Schematic locations of pattern blanket and curtain grout holes in bedrock of an earthdam of moderate size.

a) plan
b) section showing formula depths for curtain grout holes.

The above mentioned methods are rather complicated and require special equipment which the circumstances in a developing economy may not permit. The use of a upstream clay blanket should be investigated as this presents a cheap way of reducing the seepage without the need for extra equipment and expertise. The main constraint might be the construction material, but since this is almost always used as core material, it should present less problems and costs compared to grouting./13/ However, the feasibility of these methods should be worked out during the design stage so that comparable costs are known before selection of one of the methods.

5.3.3 Foundation Dewatering

When excavating at the riverbed for removal of silt and other undesirable material, presence of water may present a problem. The water will have to be removed so that the work can progress without hindrance. There are various dewatering techniques:

a) Gravity drainage. This involves the excavation of an open sump where the water collects and is pumped out. This is the simplest and most widely used method.

b) Wells and wellpoint systems. The drawdown caused by the pumping of a well creates conditions which are favourable for excavation. Figure 27 illustrates well layout for dewatering /28/. When the required ground water lowering at a site is not large, a wellpoint dewatering system is generally used. A wellpoint dewatering system consists of a series of closely-spaced small diameter wells driven to shallow depths. These wells are connected to a pipe or header that surrounds the excavation and which is attached to a vacuum pump. Figure 28 illustrates this /28/.

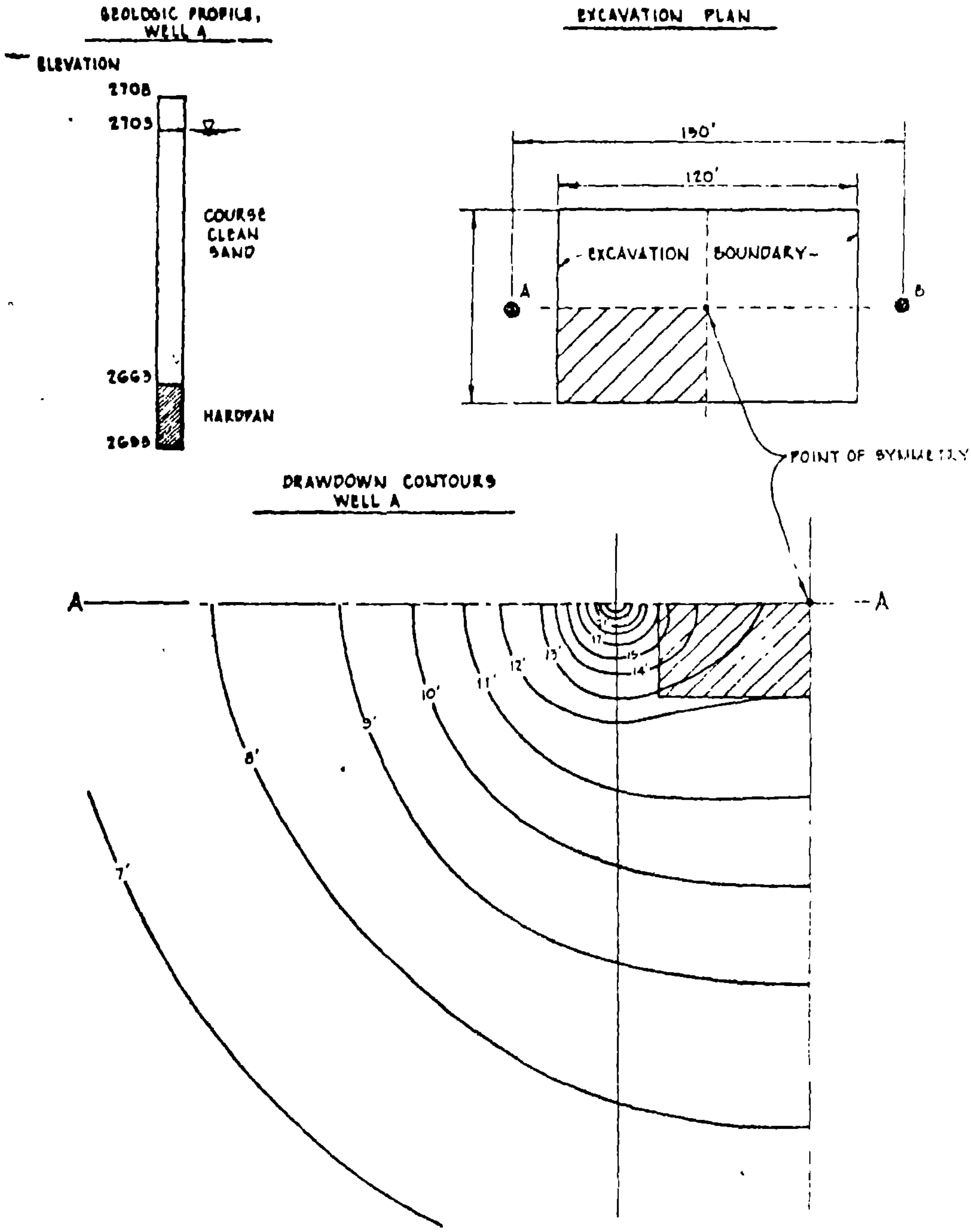


Figure 27. Plan view showing excavation boundary and well layout for dewatering. Drawdown water surface contours are shown for one quarter of the excavation area.

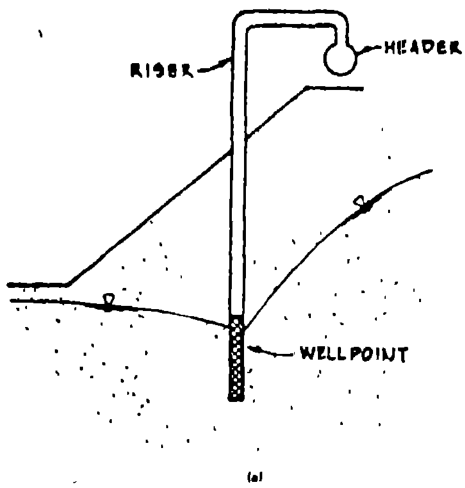


Figure 28. A single stage wellpoint system, showing the header and riser pipes.

c) Special methods. These are expensive modern methods which are employed when all the other methods have failed or have otherwise proved unfeasible. Some of the special methods are: electro-osmosis, freezing and grouting. Figure 29 gives a schematic arrangement of and electro-osmosis dewatering system /28/.

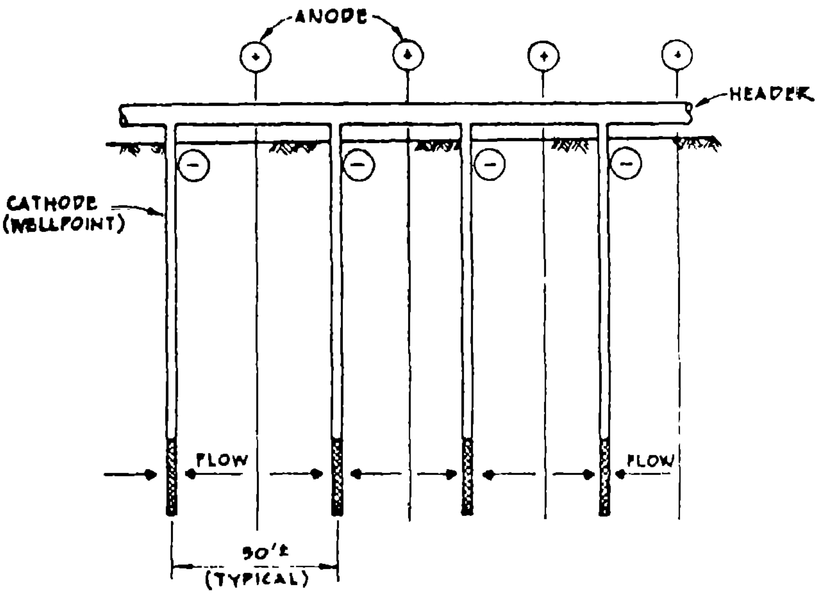


Figure 29. Electro-osmotic dewatering system (schematic).

During the foundation preparation of Mindu Dam in Morogoro, a wellpoint dewatering system was used without success. According to the contractor, dewatering was achieved as a matter of luck due to a prolonged dry season which lowered the water table.

The dewatering system depends much on the properties of the soil and in particular the permeability. In view of this it is important to study these properties before designing a dewatering system. Figure 30 shows the range of applicability of dewatering methods /28/.

5.4 Embankment Filling

5.4.1 Construction Methods

There are basically two methods of constructing the embankment. These are the hydraulic fill method and the rolled fill method. In the hydraulic fill the material is transported to the embankment construction surface in the form of a slurry through a pipeline, and deposited. This is done when the material is of medium to coarse sand, containing silty fines from below water table. The water deposited sand becomes adequately dense without any further compaction /22/.

When pervious material is contaminated with silt and clay, it becomes rather difficult for hauling equipment to travel on top. To proceed with construction, either the material is left to drain on the construction site for several days before compaction or the hydraulic fill method is used.

Nowadays this method is used mainly in the construction of ports and rarely in dam construction. In Tanzania, the rolled fill method is used on all dam projects. Hence, this discussion shall be limited to this method.

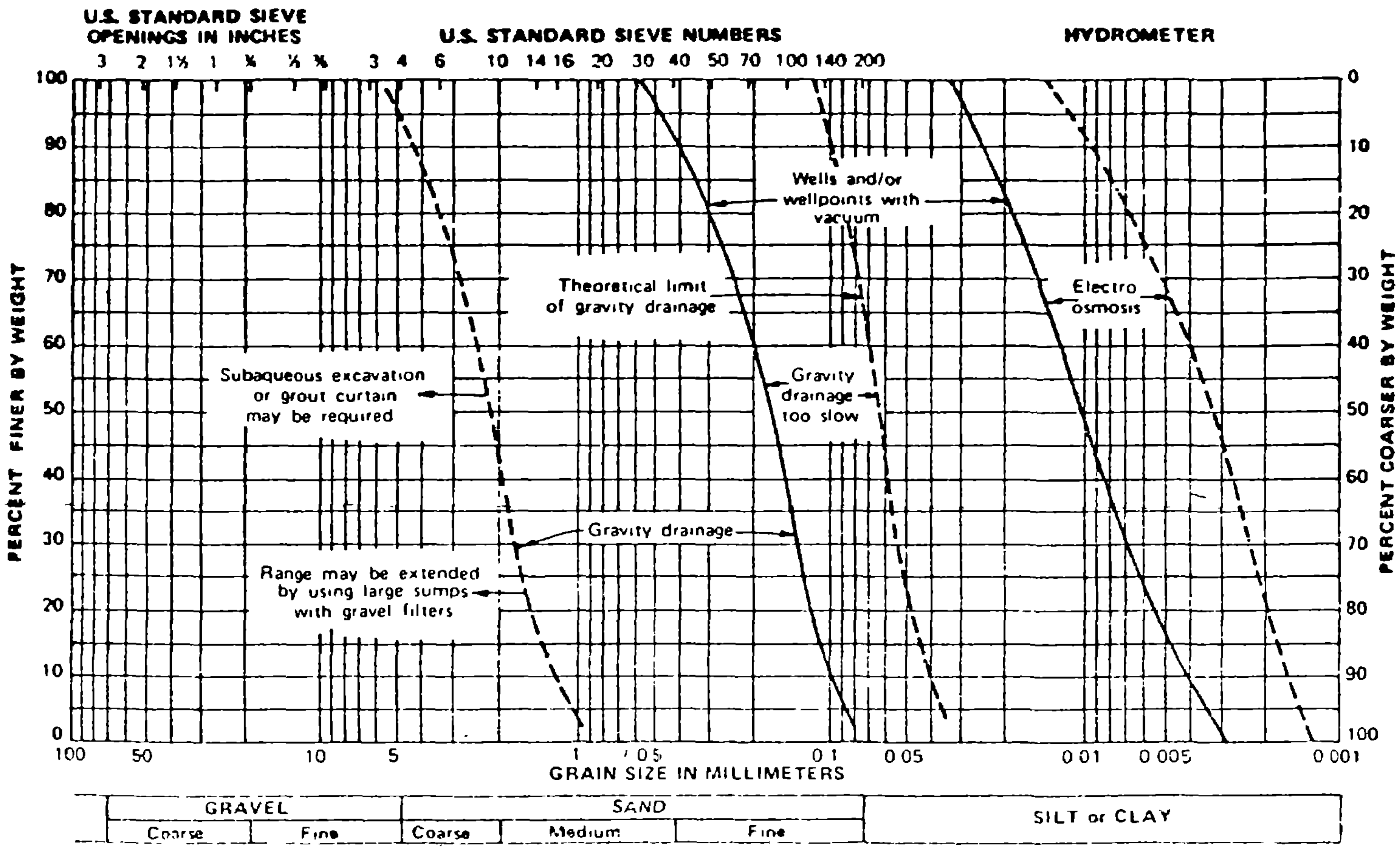


Figure 30. Applicability of dewatering methods. (Moretrench America Corporation)

The construction procedure in the case of rolled fill consists of excavating the material, hauling it to the construction site, mixing it (this can be done either in the borrow pit or on the embankment surface) to predetermined water contents and uniformity of properties, spreading it in layers and compacting it to the desired density. Usually a layer thickness of 15 - 20 cm is specified. The rate of embankment construction depends on the size of the project and the amount and types of equipment used, but is limited to a maximum practicable value by the area of the construction surface.

During the construction of Monduli Juu Dam which is 15 m high with a crest length of 165 m, an average fill of 1 200 m^3/d was achieved, using bulldozers, scrapers and sheepsfoot roller /19/. Larger quantities have been reported on larger dams /6/22/.

Depending on the size of the dam, the equipment employed ranges from manual or animal hauling to scrapers, trucks or belt conveyers. Due to compaction specifications and time scheduling, the compaction equipment is almost invariably the same. The same pieces used on earthdams are:

- excavating equipment, e.g. power shovels, draglines, bulldozers, scrapers etc.
- hauling equipment, e.g. carts, trucks, scrapers etc.
- spreading equipment, e.g. bulldozers, graders etc.
- compacting equipment, e.g. sheepsfoot rollers etc.
- rock separation equipment, e.g. screens etc.
- discs, harrows and plows
- watering equipment, e.g. pumps, water bowzers, compressors etc.

In addition to these, special equipment is frequently devised for individual construction problems.

The selection of the equipment for compaction depends on the nature of the soil to be compacted, the degree of compaction required, and the space available to do it. Sands are most efficiently compacted by vibration while cohesive soils are better compacted by pressure that is maintained for sometime /28/.

Among the various types of compactors available there are many variations of size and shape of the compacting element, ranging from the smooth drum of the vibrating roller to the small protrusions of the sheepsfoot roller. Figure 31 gives a guidance on equipment selection /28/.

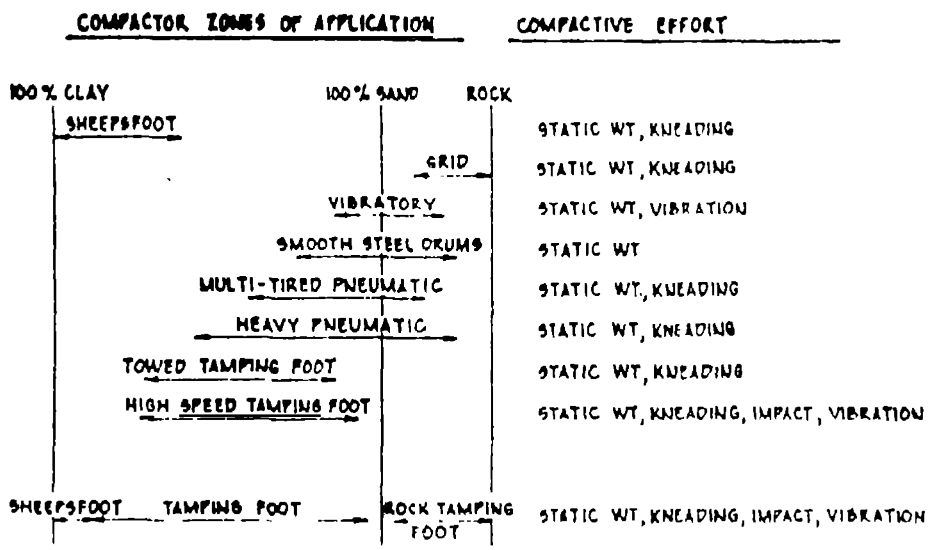


Figure 31. Compaction equipment selection guide. This chart contains a range of material mixtures from 100 percent clay to 100 percent sand, plus a rock zone. Each roller type has been positioned in what is considered to be its most effective and economical zone of application. However, it is not uncommon to find them working out of their zones. Exact positioning of the zones can vary with differing material conditions.

5.4.2 Mechanics of Compaction

After the material has been dumped and spread into a layer it is then compacted. For a given method and effort of compaction, a soil is compacted to a unique density at each water content. As can be seen in figure 32 the dry density of the soil increases with increasing water content until a "maximum dry density" is obtained at an optimum water content. After this point the dry density begins to decrease as the water content continues rising./22/28/

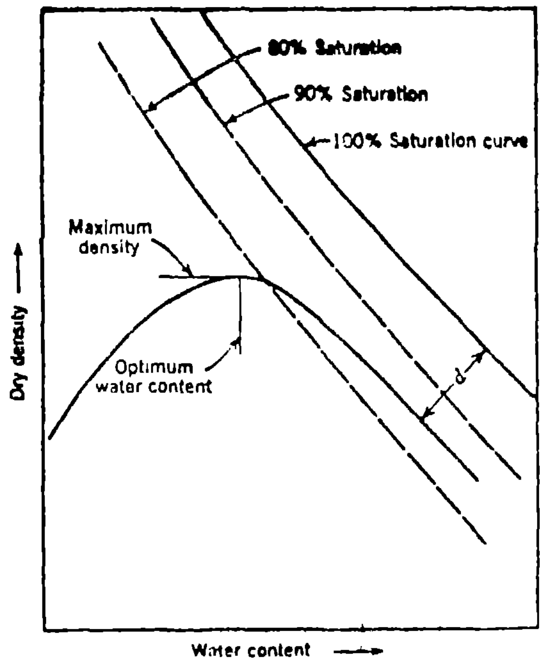


Figure 32. Typical relationship between dry density and water content for one compactive effort.

For any soil, the optimum water content, the maximum density, and the shape of the curve vary considerably with different methods of compaction. If a soil is compacted by the same method but with different compactive efforts, families of compaction curves with similar shape are obtained, figure 33.

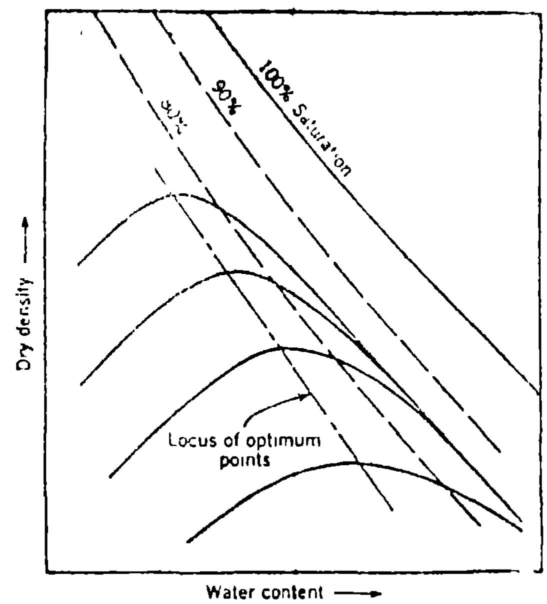


Figure 33. Typical family of density water content curves for one soil obtained using the same compaction method and equipment but different compactive efforts.

The higher the compactive effort the higher is the maximum density and the lower is the optimum water content. The locus of optimum points on the curves approximates a straight line which is roughly parallel to the curves of constant percent saturation. On the wet side of optimum water content, the curves obtained for all compactive efforts merge into a single line indicating that, at a given water content, there is a maximum degree of saturation which can be obtained with a given type of compaction equipment regardless of compaction effort expended.

On the dry side of optimum water content, the curves are approcimately parallel. It may be concluded from the curves, therefore, than an increase in compactive effort is more effective in increasing the density of the soil when its water content is on the dry side of optimum than when on the wet side.

The difference in the influence of the increased compactive effort on the density is due to the fact that the wetter material is soft and therefore weak in shear, so that the compaction energy is dissipated in shearing the compacted material without much additional densification. On the dry side of optimum moisture content, the material is stiffer and more of the energy of the compaction equipment goes into compressing the soil to a denser state /22/.

From the aforegoing discussion it may appear that there is no necessity of compacting a soil at its optimum water content i.e. the maximum density can be achieved by just increasing the compactive effort. However, the properties of the compacted fill may not conform to requirements of stability.

Placing of the material at optimum water content instead of dry of optimum increases the plasticity of the material and allows it to conform more readily to the shape of the foundation and abutments during post-construction settlement and helps to reduce the probability of tension cracks in the embankment. For small confining loads, placing material dry of optimum is undesirable because it increases the possibility of:

a) low density for the same compactive effort
b) greater permeability in the embankment core
c) excessive softening and settlement after saturation by the reservoir, resulting in possible cracking of the fill.

On the other hand, the water content should not be appreciably greater than optimum. Difficulties have been experienced with unstable fills when very wet soils are used, even in dams of low height /6/.

It is therefore the recommended practice to compact cohesive soils in small dams close to the optimum water content at Proctor maximum dry density.

In the laboratory compaction curves can be performed under controlled conditions and the curves for a given soil plotted. On the other hand, curves for compaction of embankments in the field for various types of rollers are more difficult to obtain because of the difficulties in controlling the variables which influence the test results.

5.4.3 Laboratory Compaction

During the construction stage, laboratory compaction tests are carried out on the proposed construction materials. These tests are then used as a control standard of the compaction during construction. On medium to large dams it may be necessary to set up a field laboratory with the requisite equipment to monitor the quality of the material and also the compaction. The laboratory test most commonly used for earthdam studies is the standard Proctor test /22/28/.

After establishing water contents and densities corresponding to the engineering properties desired in the fill, a compaction specification is prepared. The essence of the specification is that a certain percentage of relative density or relative compaction has to be attained for specific soils. In Tanzania, it is usual practice to limit the compaction to a minimum of 85 % of the Proctor maximum dry density on small to medium size dams. Table 8 shows USBR compaction requirements for dams less than 15 m high /28/.

Table 8. Example earthwork compaction requirements for earthdams.

Compaction Requirements	Water Control
Cohesive Soils:	Optimum ± 2 percent
0–25 percent retained on No. 4; RC=95 percent	
26–50 percent retained on No. 4, RC=92.5 percent	
More than 50 percent retained on No 4; RC=90 percent	
Cohesionless Soils	Very wet
Fine sand	
0–25 percent retained on No. 4: D_r=75	
Medium sand	
0–25 percent retained on No 4 D_r=70	
Coarse sand and gravel	
0–100 percent retained on No 4, D_r=65	

5.4.4 Compaction Control Tests

During the construction of an earthdam continuous control of compaction is necessary to ensure that the completed structure will serve its purpose. Tests are carried out periodically to ensure compliance with specifications. This may be done after a certain number of cubic metres has been placed, after a new lift has been placed, or sometimes after a day or two. The construction can sometimes be carried out more efficiently by careful interpretation of these results.

During the construction of Monduli Juu Dam, Arusha Region, tests were carried out on every layer for the first month and thereafter only once a day. It had been established that for the type of soil used, 8 passes of the sheepsfoot roller in use were enough to give the desired density within the required limits of water content. This eliminated the need to stop the work every now and then and may have influenced the time schedule /19/.

Compaction control tests involve in-place density and water content of the fill. The most common field density test is the simple sand cone method. In this method a small hole of standard dimensions is excavated and the material is weighed. The volume of the hole is then found by filling it with sand of known density, a balloon full of water or oil.

The weigth of the soil to volume ratio is the wet density or bulk density. The water content of the soil removed is also determined and the dry density is calculated as:

$$\gamma_d = \frac{\gamma_{bulk}}{(1 + w)}$$

where γ_{bulk} = wet or bulk density

w = water content

γ_d = dry density

For determining the water content, one method uses calcium carbide. This is mixed with the soil sample in a closed container and the pressure developed by the gas generated is measured. The gas pressure (acetylene gas) is directly related to the quantity of water mixed with the carbide and hence is a direct measure of the water content.

5.4.5 Control of Water Content

When the material in the borrow area is too dry its water content must be raised. In case the average water content is only a few percent below the desired value for compaction, the extra water can be applied by sprinkling the soil after it is spread on the contruction surface before rolling. The additional water is then blended into the soil with harrows or ploughs, the degree depending on the fineness and plasticity of the soil. Proper blending will eliminate pockets of wet soil and therefore poor compaction in the fill. The quantity of water which can practically be added is also depended on the fineness and plasticity (or permeability) of the soil. The coarser and less plastic the soil, the greater the quantity of water which can be worked uniformly into it.

Little or no water can be added to clays of medium and high plasticity unless they contain a large amount of sand or gravel. If the deficient water in the soil in the borrow pit is too high (more than 10 %), burrow pit irrigation is usually practiced /22/.

It is reported that this method is more economical than adding water on the construction surface. When this is done, earthfilling can proceed more rapidly because of reduced activity on the construction surface. Borrow pits are irrigated by ponding water on the surface with low dykes or with a pressure sprinkler system. When the material in the borrow pit is too wet, it should be drained before filling. It is usually easier to add water to a dry soil than to reduce the water content of a wet soil.

The fineness and plasticity of the soil greatly influences the ease with which the water content can be lowered. Furrows are usually made on the borrow area to act as drainage channels.

Another method is to dry the material by ripping or ploughing and aerating. This is relatively easy for silty and sandy soils but difficult for clays /22/. During the construction of Sasuma Dam, which supplies water to the capital of Kenya, Nairobi, wet clays in the borrow area were aerated in 10 cm layers with 6 passes of a rotary cultivator. Figure 34 shows the cross-section of Sasuma Dam /14/.

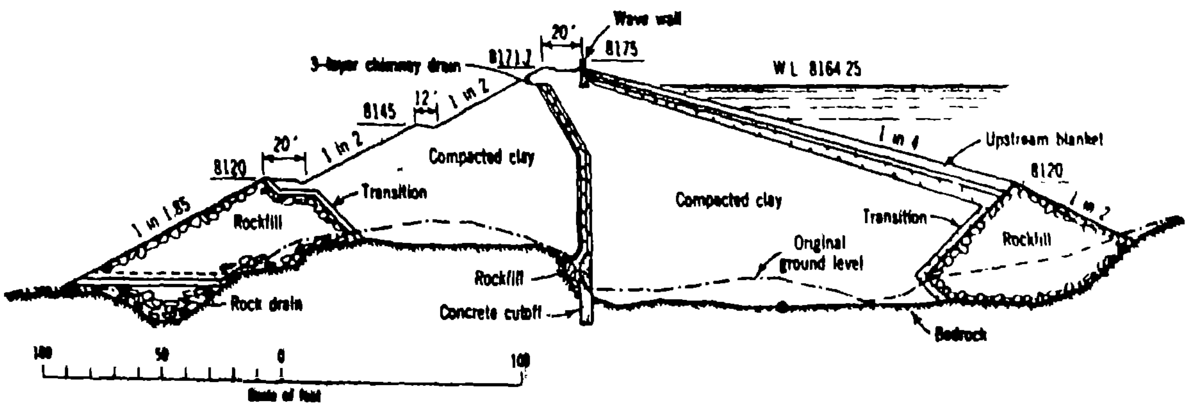


Figure 34. Cross-section of Sasuma Dam, Kenya.

5.4.6 Compaction Control for Pervious Sections

In the case of pervious embankment sections consisting of sand, gravel, or a mixture less laboratory work is usually carried out to supplement the primary visual control than for impervious sections /22/. Good compaction can generally be obtained without the close water content control necessary for impervious soils. In addition to this, the strength of compacted sands and gravel does not vary greatly with small changes in the density.

5.4.7 Control of Embankment Permeability

The control of permeability in the pervious zone is as important as the control of density. The problem of the permeability of upstream embankment zones which are designed to be free draining frequently arises during construction, because the sand or gravel borrow materials intended for these zones often are found to be dirtier and less pervious than anticipated. The stability of the upstream slope then comes into question and changes in design may be necessary.

The common practice of control is by visual inspection of the fine content and the ability of the construction surface to absorb the sluicing water added during compaction. At dams where permeability is critical, laboratory gradation tests are performed as a guide to visual control. In addition it is necessary to carry out frequent permeability test in the field laboratory.

In critical cases, large scale permeability tests have been performed by ponding water in holes or between bunds on the construction surface of the pervious section, measuring the rate of downward seepage, and computing the coefficient of permeability from flow nets /22/.

5.5 Slope Protection

It is general practice to protect the upstream slope against wave erosion. Commonly one-man stones are used in the case of manually placed rip-rap or a certain minimum thickness is specified in the case of dumped rip-rap /22/.

The downstream slope must be protected from erosion due to rain and wind. Except for very low dams, erosion has not endangered the stability of the structure but has only required maintenance, which can be difficult and expensive.

Grass planting is the most commonly practiced method of protecting the downstream slope. Only in very dry areas where there is not enough water to keep the grass cover alive, it is necessary to use other types of protection. In that case layers of rock rip-rap and coarse gravel also provide good slope protection.

The Mindu Dam which will supply water to Morogoro town is being protected on the downstream by crushed gneiss which is a byproduct in the site quarry, and therefore cheaper than vegetable soil.

Horizontal berms have been used on the downstream slope of many dams to control the runoff water and erosion /22/. The contact between the downstream slope and the abutments is almost always an area of concentrated surface runoff. The strip of embankment slope directly adjacent to the abutment is normally protected by the construction of a toe drain or placing rip-rap.

5.6 Maintenance of Dams

The maintenance of earthdams through regular inspection and repair is as important as the construction itself. In Tanzania, all the medium to large dams which are generally used for hydropower production are frequently and expertly inspected and often are equipped with monitoring devices. On the other hand, small dams which are many, get very little attention. The increased hazard potential due to downstream development and increased risk due to possible deterioration or inadequate spillway should arouse public concern as regard safety. All water works structures require constant inspection and maintenance but since dams are situated almost always in remote areas, this is rarely done. Even when this is done, there are no funds for repair work, nor is there equipment. An equal challenge is posed by the problem of effectively comminicating with the non-technical people who provide funds for the inspection or repair programmes, i.e., for demonstrating to them the benefits of such programmes.

All the dams inspected under Water Master Plan in Tabora region were found to require some maintenance /23/. This situation epitomises the current state of our dams, some of which are rather old.

However, with all the good intentions of regular inspections, there are limitations. Since the purpose of an inspection is to assist in preventing dam failures, it is interesting to ask whether it is possible that a dam could be properly inspected one day and found to have no visible discrepancies and then undergo a catastrophic failure a short time thereafter.

The answer to this question is affirmative. This is due to the fact that, unfortunately, there are some mechanisms of dam failures which develop suddenly with very little advance visible warning. One of these would be overtopping as the result of heavy rainfall or inflow into the reservoir, combined with an inadequately designed spillway. In limestone terrain the development of a large sinkhole could cause a sudden and complete breach of the embankment with little advance warning. Liquifaction or sudden loss of strength can occur in saturated fine sands subjected to vibration or shock, as would result during an earthquake. In short, a safety inspection should be considered an essential part of the safe operation of a dam, but by itself, is not an absolute preventive of dam failure /21/.

5.6.1 Checklist of Conditions to be Noted on Safety Inspection of Small Earthdams /21/

1. Vegetation on dike and within 15 m beyond toe of dike

    a) Overgrowth
    - requiring cutting for dike surveillance
    - requiring weed control for dike surveillance
    - indicating seepage or excessive capillarity

    b) Wet terrain vegetation
    - watch for boils
    - watch for sand cones, deltas etc.
    - changes with the season, pond level change

    c) Incomplete: requiring repair
    - poor growth
    - destroyed by erosion

2. Drainage ditches

    a) Clogged with vegetation

    b) Damp

    c) Flowing water: quantity

    d) Boils

    e) Silt accumulation, deltas, cones

3. Embankment

    a) Freeboard - pond level

    b) Crest:
    - cracking
    - subsidence

    c) Upstream face
    - cracking
    - surface erosion, gullying
    - wave erosion

d) Downstream face
   - cracking
   - subsidence
   - bulding
   - erosion, gullies: depth
     moisture on dry days
   - damp areas
   - boils, seeps

e) Berm and within 15 m beyond toe of dike
   - erosion, gullies
   - damp areas
   - boils, seeps

4. Spillways

   a) Intake level, boards
   b) Intake structure
   c) Discharge conduit condition
   d) Seepage or damp areas around conduit
   e) Erosion below conduit
   f) Boils in vicinity of conduit
   g) Spillway slab for uplift, subsidence, cracking

5. Areas of previous repair

   a) Effectiveness of repair
   b) Progression of trouble into new area.

6. CONCLUSION AND RECOMMENDATIONS

In the semi-arid regions with large concentrations of people and livestock, earthdam construction is a continuous process. The failure of these structures in Tanzania is attributed to insufficient investigations, poor construction and lack of maintenance. The investigation report on the failure of Monduli Juu Dam remarks: "The particular properties of volcanic soils in general and the experienced failure in particular require a careful and proper design of the reconstruction of the dam." This implies that the properties of some of the soils in the country are not properly understood.

The hydrometeorological network has been greatly increased by the Water Master Plans, but these will take sometime before the data can be used. In the meantime, however, existing hydrological methods must be used and these would require proper judgement. In view of the foregoin, the following is recommended:

1. The hydrological network should be expanded as much as is possible. The readers should get appropriate training, and if possible, regular refresher training so that the data is reliable. Whatever the size of the project, hydrological problems must be referred to qualified and experienced hydrologists for guidance.

2. Studies be conducted to evaluate the properties of various soils in the country. This would enable a proper understanding of the various soils and their use in engineering works.

3. All dams should be provided with upstreams and downstream protection of some type, irrespective of the size. Provision of toe drains should also be considered as necessary.

4. Proper records of dams should be kept so that information can be easily obtained about the size, amount of fill and costs. Also the type of material used as well as structures and soils revealed during construction would be needed incase of failure of the structure.

5. Regular inspection and maintenance should be carried out for the safety of the structure, public and property.

6. For the reservoir to be of service for a longer time it is essential to experiment and adopt soil conservation practice to reduce soil erosion.

7. All earthmoving teams should be provided with compaction equipment as well as testing equipment. The present practice is that only the national earthmoving teams have these facilities.

8. Proper specifications should be provided for construction to be used by both contractors and direct labour projects. Proper guidelines should be formulated to replace old "Head Office Notes" which are out of date and not easily obtainable.

9. As the construction of dams involves various subsections in the ministry e.g. hydrometeorological, design, construction, soils laboratory, proper coordination should be formulated so that all of them are involved in all phases of construction.

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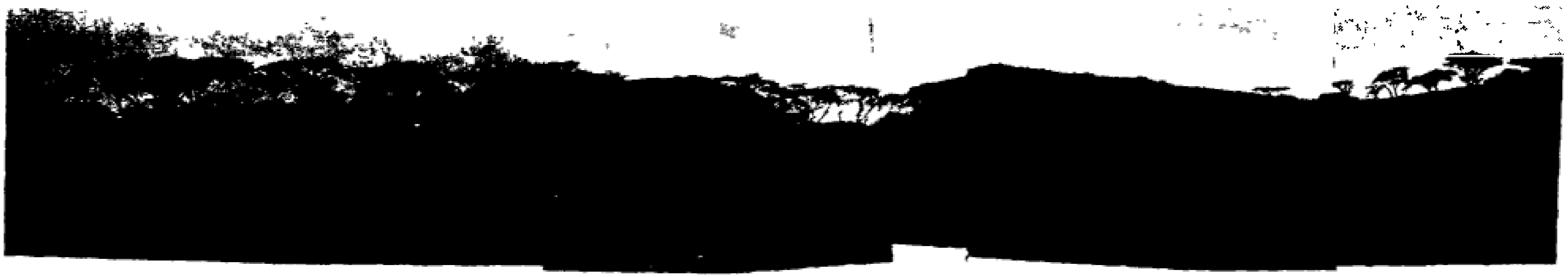
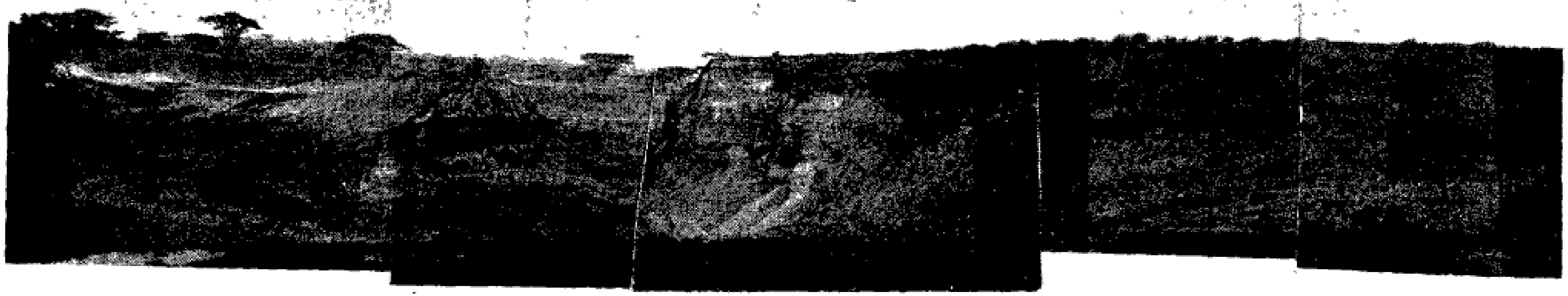


Monduli Juu Dam.   Downstream face after breach.
25.5.1977

APPENDIX



Monduli Juu Dam.   Upstream face from North End.
25.5.1977

APPENDIX

Monduli Juu Dam. Profile through breach south face.
25.5.1977

APPENDIX

Monduli Juu Dam. Profile through breached North Face.
Black cotton soil foundation right.
25.5.1977

APPENDIX