SEDIMENT MANAGEMENT IN MASINGA RESERVOIR, KENYA

DISSERTATION

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DEDICATION

Dedicated to my

Parents
Mr. Joseph Saenyi Muloboti and Mrs. Esther Nang’unda Saenyi
My roots

Wife
Caroline Nekesa Wanyonyi
For her understanding and patience

Children
Ivy Nelima Wanyonyi, Cynthia Nasimiyu Wanyonyi and Eddy Wekesa Wanyonyi
My future
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Saenyi Wycliffe Wanyonyi
ZUSAMMENFASSUNG

(Abstract in German)
Saenyi, Wycliffe W.
ABSTRACT

In Kenya, power generated from hydro-energy currently forms 70% of the total electricity output. The Seven Forks hydro-stations provides about 65% of the country's electricity requirement. Water has been cascaded from one station to the next, taking advantage of the head created by each dam to produce power. To provide adequate flow during the dry periods, water is stored at Masinga Reservoir and released during the dry season. However, this reservoir is threatened with serious siltation resulting from the accelerated erosion in Masinga catchment. The storage capacity of the reservoir is declining due to high rates of sedimentation. Sediment management strategies are therefore required to reduce the risk of siltation in Masinga reservoir and hence prolong its useful life. However, remedial measures which should be devised to curb siltation, in turn relies on the knowledge of the amount of sediment input into the reservoir and its spatial distribution.

In a nutshell, this thesis is an attempt to model both soil erosion from the catchment and sedimentation process in the reservoir in a rational integrated approach using limited available data. The catchment covers an area of 7335 km$^2$ and the surface area of the reservoir is about 120 km$^2$. The study is based on 20-year erosion and sedimentation model simulations.

A Water Erosion Prediction Project (WEPP) model has been applied to Masinga catchment in Kenya for estimation of soil loss due to surface runoff resulting from intense tropical rainfall. WEPP model is a distributed parameter continuous simulation model for predicting daily soil loss and deposition due to rainfall, snowmelt and irrigation. It was used to compute sediment yield from Masinga catchment and subsequently the amount trapped in the reservoir. Results from erosion modelling show that 6.4% of reservoir was silted up between 1981 and 1988. This figure agrees closely with the estimate of 6% computed from the 1988 hydrographic survey data. As such, WEPP model can be used to estimate the rate of siltation in the reservoir. Between 1981 and 2000, WEPP estimated a 10.1% reduction in storage capacity of Masinga reservoir.

Most computer models for the simulation and prediction of sediment transport in rivers and reservoirs are one-dimensional. Although truly two- or three-dimensional models are available, they require extensive field data for calibration and may be difficult to apply. A semi-two-dimensional model for water and sediment routing is an effective tool for solving river engineering problems. This thesis provides a brief description of the systematic and integrated approach based on well established sediment transport equations and the Bureau of Reclamation’s Generalised Stream Tube model for Alluvial River Simulation (GSTARS 2.1). Changes in elevations of various cross-sections within Masinga reservoir were modelled using GSTARS 2.1 and the simulation results employed in computing reduction in storage capacity of the reservoir. The model predicted a reduction in reservoir storage capacity of 13.7% for the period between 1981 and 2000.

Apart from volumetric analysis, GSTARS 2.1 simulation results for 2000 were also used to determine sediment distribution in the reservoir. The processing and visualisation of these results were performed using a two-dimensional BOSS SMS model to yield the sedimentation pattern. The 2000 reservoir bathymetry showed that high rates of sedimentation had occurred in the old river channels and little on the reservoir terraces. The pictures also revealed that there is more siltation at the two mouths of the reservoir, at the confluence, and near the dam wall.
Based on results of erosion and sedimentation modelling, sediment management strategies are proposed to reduce the risk that siltation in Masinga reservoir poses. Construction of sedimentation basins at the mouths of the reservoir is seen as the most feasible solution to sedimentation problems in Masinga reservoir. This should be reinforced by use of watershed management option to reduce the yield of sediment and its entry into the reservoir. Periodic sluicing of sediments through operation of bottom outlet gates would also help to ease the problem. Dredging of highly silted areas is plausible if the procedure is cost-effective.
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Notation

Dimensions

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Meaning</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Length</td>
<td>[L]</td>
</tr>
<tr>
<td>M</td>
<td>Mass</td>
<td></td>
</tr>
<tr>
<td>T</td>
<td>Time</td>
<td></td>
</tr>
</tbody>
</table>

Variable | Physical Meaning                                                                 | Dimension       |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>B</td>
<td>Width of channel, length of the interaction zone between the main channel and the flood plain</td>
<td>[L]</td>
</tr>
<tr>
<td>C</td>
<td>Depth-averaged local suspended sediment concentration</td>
<td>[ML^{-3}]</td>
</tr>
<tr>
<td>C_e</td>
<td>Courant number</td>
<td></td>
</tr>
<tr>
<td>C_a</td>
<td>Equilibrium concentration</td>
<td>[ML^{-3}]</td>
</tr>
<tr>
<td>E</td>
<td>Erosion rate</td>
<td>[ML^{-2}T^{-1}]</td>
</tr>
<tr>
<td>d</td>
<td>Grain size of sediment</td>
<td>[L]</td>
</tr>
<tr>
<td>d_{50}</td>
<td>Median grain size</td>
<td>[L]</td>
</tr>
<tr>
<td>D*</td>
<td>Dimensionless sediment parameter</td>
<td></td>
</tr>
<tr>
<td>g</td>
<td>Gravitational acceleration</td>
<td>[LT^{-2}]</td>
</tr>
<tr>
<td>h</td>
<td>Local average water depth</td>
<td>[L]</td>
</tr>
<tr>
<td>k</td>
<td>Constant of proportionity</td>
<td></td>
</tr>
<tr>
<td>k_s</td>
<td>Characteristic bed roughness</td>
<td>[L]</td>
</tr>
<tr>
<td>M</td>
<td>Erosion parameter</td>
<td>[ML^{-2}T^{-1}]</td>
</tr>
<tr>
<td>p_{bn}</td>
<td>Percentage of material of grain size n in the bed surface</td>
<td>[L]</td>
</tr>
<tr>
<td>q</td>
<td>Lateral discharge</td>
<td>[L]</td>
</tr>
<tr>
<td>S</td>
<td>Sedimentation rate</td>
<td>[ML^{-2}T^{-1}]</td>
</tr>
<tr>
<td>T</td>
<td>Sedimentation coefficient</td>
<td>[L]</td>
</tr>
<tr>
<td>T_a</td>
<td>Thickness of active layer</td>
<td></td>
</tr>
<tr>
<td>u</td>
<td>Local depth-averaged velocity in the longitudinal (streamline) direction</td>
<td>[LT^{-1}]</td>
</tr>
<tr>
<td>u_m</td>
<td>Flow velocity in the main channel</td>
<td>[LT^{-1}]</td>
</tr>
<tr>
<td>u_p</td>
<td>Flow velocity in the flood plain</td>
<td>[LT^{-1}]</td>
</tr>
<tr>
<td>u_s</td>
<td>Shear velocity</td>
<td>[LT^{-1}]</td>
</tr>
<tr>
<td>v</td>
<td>Kinematic viscosity</td>
<td>[MLT]</td>
</tr>
<tr>
<td>w_s</td>
<td>Fall velocity of sediment</td>
<td>[LT^{-1}]</td>
</tr>
<tr>
<td>x, y, z</td>
<td>Spatial co-ordinates</td>
<td>[L]</td>
</tr>
<tr>
<td>\rho</td>
<td>Water density</td>
<td>[ML^{-3}]</td>
</tr>
<tr>
<td>\rho_p</td>
<td>Bulk density of sediment</td>
<td>[ML^{-3}]</td>
</tr>
<tr>
<td>\rho_s</td>
<td>Sediment density</td>
<td>[ML^{-3}]</td>
</tr>
<tr>
<td>\tau</td>
<td>Bottom shear stress</td>
<td>[ML^{-1}T^{-1}]</td>
</tr>
<tr>
<td>\tau_{c,E}</td>
<td>Critical shear stress for erosion</td>
<td>[ML^{-1}T^{-1}]</td>
</tr>
<tr>
<td>\tau_{cro}</td>
<td>Critical shear stress for deposition</td>
<td>[ML^{-1}T^{-1}]</td>
</tr>
<tr>
<td>\Delta</td>
<td>Bed-form height</td>
<td>[L]</td>
</tr>
<tr>
<td>\Delta x</td>
<td>Distance along the x axes</td>
<td>[L]</td>
</tr>
<tr>
<td>\Delta y</td>
<td>Change in bed elevation</td>
<td>[L]</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Units</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>-------</td>
</tr>
<tr>
<td>Δz</td>
<td>Distance along the z axes</td>
<td>[L]</td>
</tr>
<tr>
<td>σ</td>
<td>Standard deviation of a set of data</td>
<td></td>
</tr>
<tr>
<td>ε</td>
<td>Convergence criterion</td>
<td></td>
</tr>
<tr>
<td>εₘ</td>
<td>Molecular diffusion coefficient</td>
<td>[L²T⁻¹]</td>
</tr>
<tr>
<td>κ</td>
<td>Coefficient or von Karman constant, κ = 0.4 for clear water</td>
<td></td>
</tr>
</tbody>
</table>

**Superscripts/Subscripts**

- b: river bed, dead water zone
- m: main channel
- n: grain size fraction
- p: flood plain
- s: sediment

**Abbreviations**

- FDM: Finite difference method
- FVM: Finite volume method
- FEM: Finite element method
- CM: Characteristic Method
- MOC: Method of Characteristics
- D: Dimension
- STARS: Sediment transport and river simulation
- GSTARS: Generalized stream tube model for alluvial river simulation
- HEC: Hydrologic engineering centre
- RESSED: Reservoir sedimentation model
- WEPP: Water Erosion Prediction Project
- SMS: Surface water Modelling System
- MUSLE: Modified Universal Soil Loss Equation
- GPS: Geographical Positioning System
- UNESCO: United Nations Educational, Scientific and Cultural Organization
- USLE: Universal Soil Loss Equation
- RUSLE: Revised Universal Soil Loss Equation
- MOSES: Modular Soil Erosion System
- FOCS: Field Officers Computing System
- KINEROS: Kinematic Runoff and Erosion Model
- AGNPS: Agricultural Nonpoint Source Pollution Model
- EPIC: Erosion Productivity Impact Calculator
- ANSWERS: Areal Nonpoint Source Watershed Environmental Response Simulation
- HSPF: Hydrological Simulation Program – FORTRAN
- SWRRB: Simulator for Water Resources in Rural Basins
- CREAMS: Chemicals, Runoff, and Erosion from Agricultural Management Systems
- SWAT: Soil and Water Assessment Tool
- GRASS: Geographic Resources Analysis and Support System
- CLIGEN: Climate generator
- OFE: Overland Flow Element
- UTRC: Upper Tana River Catchment
- TARDA: Tana and Athi Rivers Development Authority
- MOA: Ministry of Agriculture
- MOWD: Ministry of Water Development
- KENGEN: Kenya Power Generating Company
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>EEC</td>
<td>European Economic Community</td>
</tr>
<tr>
<td>ODA</td>
<td>Overseas Development Agency</td>
</tr>
<tr>
<td>MOW</td>
<td>Ministry of Water</td>
</tr>
</tbody>
</table>
CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

One of the most effective ways of developing surface water resources is to construct reservoirs. When the reservoirs are built on sediment laden rivers, however, their useful lives are often curtailed unless special measures are taken in its planning and management to reduce reservoir sedimentation. The loss of reservoir capacity is often a grave matter. In addition to being an important economic loss, safety and welfare of many people downstream may be affected if storage is needed for flood protection and water supply.

Any reservoir on a sediment carrying river will gradually become silted up even though the process may take a long time. It is therefore very important to take care that the silting up of the reservoir does not occur before the benefits from it are fully achieved. It would be a gross error of the decision makers if the sedimentation of the reservoir was not foreseen and if no measures were taken in order to check it, or to reduce its effects to an acceptable level. Negligence in this respect would cost dearly in the future, since the available water resources are limited as are the economically suitable reservoir sites within a basin.

Therefore systematic plans and policies are needed to reduce adverse impacts of sedimentation and prolong the useful lives of reservoirs. The ability to estimate the rate of watershed surface erosion, sediment transport, scour and deposition in a river system, and sediment deposition and distribution in a reservoir is essential for the development of sound sediment management plans and policies.

As a result of runoff from rainfall, soil particles on the surface of a watershed can be eroded and transported through the processes of sheet, rill, and gully erosion. Once eroded, sediment particles are transported through a river system and are eventually deposited in a reservoir, lake or at sea. Therefore, an erosion model capable of predicting surface erosion and routing sediments through a channel system is desirable. Engineering techniques used for the determination of reservoir sedimentation processes rely mainly on field surveys. Field surveys can be used for the determination of what has happened but not for predictive purposes.

During the 1997 Congress of the International Commission on Large Dams (ICOLD), the Sedimentation Committee passed a resolution encouraging all member countries to (a) develop methods for prediction of the rate of surface erosion based on rainfall and soil properties, and (b) carry out the simulation and prediction of reservoir sedimentation processes using numerical computer models. This thesis reports on a research carried out on Masinga catchment and reservoir in compliance with the above two ICOLD resolutions. The results will be presented to demonstrate the feasibility of this systematic and rational approach for the determination of surface erosion rate and sediment transport in river channels. This notwithstanding, the determination of reservoir sedimentation pattern will also be addressed.
1.2 IMPACTS OF RESERVOIR SEDIMENTATION

Sedimentation in reservoirs adversely affects the reservoir operations. These impacts can be categorised as physical and biological impacts, economic and social impacts, and impact on the safety of the dam itself.

1.2.1 ECONOMIC AND SOCIAL IMPACTS

1.2.1.1 Consequences of Reduction of Storage

The impact of reservoir sedimentation on the social and economic objectives of the project depends upon the size and the characteristics of the deposits. The consequences are very complex, because the dams usually serve multiple objectives, which may evolve over the years. Sedimentation affects the storage capacity, which is the main asset of the reservoir. The loss of storage is particularly felt in connection with energy production, water supply for domestic use, industry and agriculture, and in flood control. Sedimentation also affects surface area of the reservoir, by reducing water depth and favouring development of aquatic growth. This affects, sometimes, adversely, the use of the reservoir for recreation, fire fighting, and public health.

1. Influence on Energy Production

Hydro-electric plants are used to provide the peak demands for energy because of the flexibility of their operation. The reservoir is, therefore, an energy accumulator and the larger its capacity the more efficient it is. Hence, any reduction of the capacity will adversely affect the energy output of the associated power plant and the maximum power available during the most critical period of consumption, when the water input is insufficient.

2. Influence on Agriculture and Industry

If there are alternatives to the production of water power, there is no substitute for water for irrigation. Although the most efficient techniques of irrigation may save water, they cannot do without it. Therefore, sedimentation of a reservoir serving agricultural purposes is far more critical than that serving power production.

The effects of a well-designed irrigation project may often be spectacular with regards to agricultural production; in certain climates, irrigation is a necessary condition of life itself. The loss of storage may, thus, lead to shortages of water with catastrophic consequences, since this can cause the loss of crops which are as a result of the year’s labour.

The deepening of the riverbed downstream of the dam as a consequence of trapping sediments in the reservoir, may lay bare intakes of water supply systems which may need to be reconstructed in order to serve their objectives.

Industry is a large consumer of water, the water being used in the industrial process itself or for cooling. Water shortages may be very costly for industry, although the consequences are more transient than in agriculture.

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Sediments ejected into the downstream reach of the river by floods, flushing, or reservoir emptying, etc., may cause different nuisances: silt deposition in water supply conduits, obstruction of sprinklers in irrigation systems, clogging of heat exchangers in factories, disturbing the operation of wastewater treatment plants, producing spots on fruits, and thus reducing their commercial value, etc.

### 3. Influence on Discharge Regulation

Reduction of storage capacity also affects the capability to regulate discharges of supplemental water in time of low river discharges for environmental protection and for industrial or agricultural activities.

In the case of reservoirs for flood control, the loss of storage may have even more serious consequences because the flood plain dwellers downstream of the dam, believing that they are protected by the dam, usually neglect other measures of protection against possible flood damage.

#### 1.2.1.2 Influence of the Reduction of the Water Surface

The reduction of the water surface is caused by both the emergence of deposits and the growth of weeds in shallows.

#### 1. Influence of Boating, Sailing and other Outdoor Sports

Sediment deposition and growth of weeds can obstruct the access for marine vessels and hinder their movement.

Mud is a serious nuisance to beaches which are otherwise suitable for swimming and water sports.

On the other hand, aquatic vegetation is an excellent shelter for birds and other aquatic fauna, hence is welcome by fishermen and hunters.

#### 2. Influence on Public Health

The same vegetation on the banks of the reservoir, which provides shelter for animals, also favours the proliferation of insects, whose unpleasant bite alone can discourage human settlement in the area. More serious is the danger from insects which carry diseases such as malaria, yellow fever, sleeping sickness, etc.

#### 3. Influence on Fire Fighting

Large reservoirs are well suited for refilling of sea planes used in some countries for fire fighting. A substantial reduction of the free surface can impede the action of the aircraft.

#### 1.2.2 IMPACT ON THE SAFETY OF THE PROJECT

Sediment deposits may affect the safety of the dam in the following ways:

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1.2.2.1 Influence of Sediment Deposits on the Bottom Outlets

One of the most important hazards of sedimentation is the choking of the bottom outlets by accumulated masses of sediment, reinforced by entangled tree trunks and immersed debris. A complete occlusion of the outlet may occur. The reservoir must, however, be emptied occasionally. Therefore, at planning stage, facilities should be designed to allow floating objects to be evacuated through gates of appropriate dimensions at the right places. The conduits of the bottom outlets must be straight and of sufficient size, controlled by gates which allow free passage of the flow. It is recommended that trees in the reservoir area should be cleared before it is filled.

1.2.2.2 Influence on Gates and Valves

At high flow velocity through the bottom outlets, cavitation and flow-induced vibration may be accompanied by abrasive action of sediment particles, affecting the structures, linings, concrete surfaces, and the apron if the sediment is composed of hard minerals. The emanation of gas, particularly of hydrogen sulphide, discharged by the dislocated and decompressed mud, may corrode the metallic parts of the gates and the equipment located in the gate chamber.

1.2.2.3 Influence on the Dam

The pressure exercised by the sediment deposits poses a security risk to the dam as a structure. Hence, weight of sediment deposits should be considered in the structural design of the dam. Moreover, it is important to make sure that the designed characteristics of the structure are not altered by deterioration of the concrete. The chemical reactions within the deposits, particularly those pertaining to sulphur cycle, as well as the degree of corrosivity of the water, must be taken into account. The concrete and concreting techniques, as well as the lining of the surface of the dam, should be foreseen by the designer.

1.2.2.4 Influence on the Supervision of the Dam

Silt deposits may hinder the supervision of the dam. They can prevent the use of underwater means of supervision such as submersibles, divers, television, etc. They hamper visual observation after having emptied the reservoir, because the surface of the reservoir is soiled by mud and access from the sides is made impossible.

1.2.3 PHYSICAL AND BIOLOGICAL IMPACTS

The construction of a dam in the river valley causes a modification of the regime of the river discharges, a transformation of river channel morphology in the zone of the reservoir and a significant change of the conditions of the sediment transport. Downstream discharges may be reduced if a part of the flow is diverted by the project. The unity of the river basin, however, does not allow modification of one reach of the river without disturbing the basin as a whole to some degree, particularly with regard to sediment transport and its effects.
1.2.3.1 Influence on the Reservoir

By impounding a river, the cross sections available for the flow increase and the flow velocity decreases; which leads to a decrease of the sediment transport capacity. This causes the deposition of sediments, the consequences of which depend on the size, shape and location of the deposits. The coarser particles are deposited first, building up an underwater delta near the upstream end of the reservoir, the configuration and progression of which depend on the regime of flow and variation of water levels in the reservoir. This delta causes a rise of the original river bed and, in addition to reduction of the active storage, it may cause a reduction of the clearance below bridges which may represent a danger to navigation. It also causes an increase of water levels during floods, which may lead to the submergence of the riverside land, particularly when the emerging sand banks are recultivated or overgrown by shrubbery. A further consequence of delta formation is the raising of the water table, which results in the appearance of marshes.

The finer sediments are carried into the reservoir and settle on the reservoir bottom depending on the flow velocity and wind-induced or thermal currents. These fine materials will be spread over the littoral and sublittoral zone from which the silt and mud will be carried progressively to the dam where it accumulates and, in some cases, creates a real mud lake. The deposited fine materials may affect both the active and the dead storage. In the littoral areas there will be a marked rise of the bottom creeks and shallows, which may lead to the development of aquatic growths. These may, in turn, accelerate the consolidation of the deposited silt and mud. If these emerge over prolonged periods of time, they may dry up and become eroded by wind. In deep sublittoral zones, the deposited silt will progressively consolidate, gradually filling the dead storage, except when sediment flushing is practised.

The consequences of the deposition depend on the position of the river inflow in relation to the structure. It is obvious that the inflow of a tributary close to a water intake must be carefully controlled. The effects of bank erosion caused by wave action and reservoir level fluctuations must be taken into account, in addition to the transport of sediments.

1.2.3.2 Influence of Sediments on the Aquatic Life

Sediment is one of the factors of the aquatic biocenosis, which is governed, particularly by the energy flux transmitted by solar radiation which in turn determines the photosynthetic activity and recycling of organic matter. The penetration of solar radiation into the water depends on the clarity of the water and therefore, also on the concentration of sediment suspension.

1.3 RESEARCH QUESTIONS

It can be envisaged from the foregoing section that the problem of reservoir sedimentation need to be addressed right at the design stage of the dam or during reservoir operation. In view of this, the study is designed to address the following research questions which will be explored throughout the thesis:
- Can sedimentation be detected in Masinga reservoir?
- What are the likely future sedimentation scenario in this reservoir?

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What will be the impact of sedimentation on this surface water resource and on hydropower generation?

What response strategies can be adopted to reduce the risk posed by sedimentation in this reservoir?

1.4 STATEMENT OF PROBLEM

Many parts of Kenya have environmental conditions that are conducive to accelerated soil erosion (Ongweny et al., 1993). Cultivation of steep slopes within the wet highlands, resulting from the population pressure on the land, and intense grazing in semi-arid lowlands enhance rapid soil erosion (Figures 1.1 & 1.5). Removal of soil degrades the soil profiles on hillslopes and when the soil is transported to the main streams of Kenya, it causes many other problems. The most urgent of these downstream problems is the transportation of sediment to main reservoirs. Masinga reservoir has experienced the problem sedimentation since its inception in 1981 (Hirji and Bobotti, 1998).

To ensure that Masinga reservoir is effectively used, appropriate measures have to be taken to prevent rapid loss of its storage capacity. Although the dam may still provide the full utilisable head, the number of peak load hours will be reduced substantially, due to reduced reservoir capacity. Because of sedimentation, there may also be an increasing risk of intakes being blocked by sediments (Morris and Fan, 1998). Additionally, sediment load associated with major flood events create high loads on the dam wall threatening the dam’s safety (Chanson and James, 1999). Masinga reservoir is a regulating scheme for the lower dams and any loss of storage capacity increases the risk of failure to meet the design objectives in a dry period. A case in point is the more recent drought (La Nina) of 1999/2000 which led to the closure of Masinga power station when the reservoir level fell below minimum operating level of 1033 m above sea level (a.s.l). It is believed that the reduced reservoir storage capacity due to sedimentation caused a reduction of utilisable head which in turn contributed to the closure of the dam. This is evidenced from the photographs taken in August 2000 (Figures 1.3 and 1.4 & 1.6). Since the dam traps a substantial amount of sediments, the modification of discharges downstream of Masinga dam with reduced sediment content has caused scouring of the river bed (Fig. 1.2).

The rate of deposition in any reservoir depends on the rate of sediment supply. Without the knowledge of the rate of sediment supply, it would be difficult to compute the trap efficiency of the reservoir. There is no doubt that reservoir failures have occurred because of insufficient knowledge of the long term supply of sediment at the inception of such projects (Unesco, 1985).

Since any sediment brought into the reservoir occupies space, the storage capability of the reservoir is reduced. At times the storage is so depleted that the services given by the stored water are directly affected. Therefore it is important to update the storage capacity of the reservoir with time so as not to fail in its objectives. This would be possible if the Engineer knew the probable volume of sediment anticipated to accumulate within the life span of the reservoir. This research is designed to address pertinent issues of sediment accumulation in Masinga reservoir.

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Fig. 1.1: Overstocked goats clearing the remaining vegetation from Masinga catchment, accelerating soil erosion (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)

Fig. 1.2: Foundation of bridge downstream of Masinga dam eroded as the channel degrades. Glaring evidence of sedimentation in reservoir (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)

Fig. 1.3: Sediment dominated Masinga reservoir with original river channel reduced to merely 2 m. dry cohesive sediment deposits seen (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)

Fig. 1.4: Remnants of once expansive Masinga reservoir. At background, a really mud lake can be seen (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)

Fig. 1.5 (left): shows gully erosion taking place in Masinga catchment (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)

Fig. 1.6 (below): shows the extend of sedimentation in Masinga reservoir. Cracks can be seen in dry cohesive sediment deposits (photograph taken on 31/8/2000, during the La Nina drought of 1999/2000)
Another important point is to study the manner in which sediment deposition is distributed throughout the reservoir. Such information would give the Engineer an idea of where to expect most sediment accumulation and therefore give remedial measures to be applied.

Although there have been a few consultancy reports, sedimentation problems in Kenya seem to have been underestimated so far, as was reported by Dunne and Ongwenyi (1976). There is need therefore, for a more comprehensive assessment of the amount of soil erosion that has occurred so far from Masinga catchment and also predict the probable future rates of sedimentation in the reservoir. Such assessment should be based upon a detailed investigation of the hydrologic and geomorphic processes as they are controlled by the environmental characteristics of the region.

This thesis reports on the research conducted on Masinga catchment and reservoir with simulations performed over a period of 20 years. The research was not only aimed at assessing the soil erosion but also its effect on reservoir sedimentation. This is useful for planning of future surface water resources development in Kenya and other developing countries.

1.5 AIMS OF THE STUDY

The specific aims and objectives of the research included the following:

(1) Definition of the rates of sediment production for various sub-catchments within Masinga drainage basin.
(2) Predict total event based sediment yield from Masinga catchment for the period of 20 years.
(3) Simulation of the time series of suspended load for rivers draining into Masinga reservoir.
(4) Computation of the rates of sedimentation in Masinga reservoir.
(5) To predict the present sedimentation patterns in Masinga reservoir and in backwater areas affected by reservoir operation.
(6) Consideration of the most appropriate and useful techniques for soil and water conservation in the light of information on rates of sediment production.
(7) Recommendation on techniques for sediment management to prolong the useful life of Masinga reservoir.

1.6 ORGANISATION OF THE THESIS

This thesis is divided into 7 chapters.

Chapter one outlines the statement of the problem, specific aims and objectives of the present study. The chapter also provides some background information on the problems caused by sediment accumulation in reservoirs.

Chapter two briefly reviews the theory of sediment transport and numerical modelling of sedimentation in rivers and reservoirs. The same chapter gives a synopsis of some work done by other researchers on soil erosion and sediment yield studies in Masinga catchment and elsewhere. Furthermore, case studies of sediment management in reservoirs are reviewed here.

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Chapter three outlines the research methodology employed in this study. An overview of some of the erosion and sedimentation models is given. The chapter also describes the selected erosion and sedimentation models, highlighting pertinent issues on data requirements for each model.

Chapter four concentrates on the suspended sediment generator for the rivers draining into Masinga reservoir. The application of the selected erosion model (WEPP) to Masinga catchment is dealt with here. Emphasis is laid on the description of study catchment as well as its database. WEPP model simulation results and discussion are also presented in this chapter.

Chapter five deals with the application of GSTARS 2.1 to Masinga reservoir. Within the same chapter, basic information about the study reservoir and its database are described. Applicability of GSTARS 2.1, the simulation results and discussion are presented in this chapter. Calibration and application of the model are also addressed here.

Chapter six consolidates the findings from chapter four and five in conjunction with the first three chapters and discusses how policy formulation and implementation on soil erosion can be integrated in sediment management in reservoirs. Sediment management strategies in the catchment as well as in the reservoir are assessed and appropriate techniques that may be used in order to remedy the sedimentation problem are proposed in this chapter.

Chapter seven summarises the entire study by way of outlining the main conclusions, recommendations and areas for further research.
STATE OF THE ART

2.1 REVIEW ON SOIL EROSION AND SEDIMENT YIELDS

A lot of measurements on soil erosion and sediment yields have been carried out throughout the world, but relatively few in Africa. The results show the overwhelming influence of land use on sediment yields, but there have been relatively few attempts to quantify the effect of this factor under tropical conditions (Ongwenyi, 1999).

The present review seeks to briefly evaluate some of the erosion and sediment yields as well as reservoir sedimentation work that has been done on the world wide basis and in the tropics. Particular attention will be focussed on the work already done in Africa as a whole with special reference to Kenya and to some extent the work that has been carried out in Masinga catchment, the area of the present study.

Soil erosion is not a new problem in Kenya. It has been recorded since the nineteenth century and during the early decades of last century, it was already regarded as a serious problem in many parts of the country (Atkins, 1984). The erosion problem has come about through increasing pressure on land in terms of people, livestock and crop production exacerbated to some extent by vagaries of climate. The land and water resources are being increasingly threatened by these rising demands. Farmers with minimal resources are moving into marginal areas (in terms of both climate and slope) with high erosion hazards, and low production potential hence worsening the problem of soil erosion.

Simanton et al. (1993), estimating sediment yield in a semiarid basin, showed that sediment yield estimates and erosion prediction models depend on the experimental data collected from the field and small drainage basin areas. It was concluded that in all field studies, the design should ensure, as practically and economically as possible, that complete measurement of data is made. Suspended and bed load, both need to be sampled to get accurate information about sediment yield of a given basin or sub-basin.

Judson (1974), discussing erosion on a world wide basis, has shown that the Congo with a drainage area of $2.5 \times 10^6$ Km$^2$ has sediment production rate of 78 t/Km$^2$/year with a denudation rate of 2 cm per 1000 years. Stoddart (1969) has indicated that Africa with an area of $29.81 \times 10^6$ Km$^2$ has a mechanical denudation rate of 25 t/Km$^2$/year. However, he did not take into account the fact that land use types have changed tremendously in many parts of the continent which is also characterised by great climatic variability. These figures may thus represent very approximate magnitudes of the erosion and sediment yields. Holman (1968) has included the sediment yields of some of the major African river basins in his assessment of sediment yields of the major rivers of the world.

Onodera et al. (1993) conducted a research on a gentle slope in inland area of Tanzania for two years, in order to clarify the seasonal variation of sediment yield on inter-rill area in semiarid zone. The sediment yield and runoff indicated the clear relationship, except for the values at the beginning of the rainy season. It was found that soil erosion rate from a complete bare land was 3.1 mm/year, on the land covered with acacia trees was 1.0 mm/year and on the grassland was 1.2 mm/year. He inferred that the increasing of the sediment concentration in the beginning of the rainy season was controlled by the existence of erodible soil on the ground, and the decrease in late rainy season is controlled by the developing vegetation.
Several attempts have been made to relate in broad terms the sediment yields of catchments to individual climatic parameters. Langbein and Schumm (1958) have related sediment yields to mean annual effective precipitation within a range of climatic zones. They have shown, for example, that as effective rainfall and runoff in arid regions increase, the mean rate of erosion also increases. This is because there is more rainfall, and hence runoff, to cause soil removal. As rainfall continues to increase, however, vegetation becomes more dense shielding the soil from raindrop energy and sheet wash. The tendency of vegetation to increase with rainfall therefore eventually counteracts the effect of increasing rainfall causing erosion rates to decline with increasing runoff beyond the margins of the desert. Dendy and Bolton (1976) reached essentially the same conclusion.

Fournier (1960) has related an index of seasonality of rainfall to sediment yields while Douglas (1967) has related sediment yields to annual runoff. Wilson (1973) has related mean annual rainfall to sediment yield in catchments.

The quantitative description of sediment transport across areas of deposition is an essential part of assessing the off-site effects of soil erosion on hillslopes. Beuselinck et al. (1998) did experiments on sediment deposition by overland flow and found out that up to a certain unit discharge, a simple settling equation without a transport term, gives a good prediction of the sediment delivery ratio and the grain-size distribution of the deposited and exported material. Only when the critical unit discharge is exceeded do properties of the overland flow influence the sediment delivery amount.

Douglas (1967) evaluated the Langbein-Schumm relationship and postulated that man’s use of the landscape has increased the rates of erosion, that they by far exceed those before man became an important geologic agent.

Effectiveness of land management decisions aimed towards preventing negative impacts of soil erosion in complex landscapes can be significantly improved by detailed predictions of erosion and deposition patterns for proposed land use alternatives. Mitas and Mitasova (1998) presented a new generation of simulation tools for modelling soil erosion, sediment transport and deposition by overland water flow in complex landscapes. The simulations were based on the solution of bivariate continuity equations describing water and sediment transport over three-dimensional terrain with variable climatic, soil and land-cover conditions.

Wilson (1973) reviewed all of these relationships and concluded that difference in climatic regimes and land use make it impossible to define a single rule relating sediment yields to rainfall or runoff and even within a relatively uniform area. The most important factor is land use. Jansen and Painter (1974) incorporated an index of natural vegetation into their multiple regression analysis of sediment yields but this did not take into account the fact that their natural vegetation has been altered by various land use types.

With increased computer power, soil erosion modelling has become the state of the art. Green et al. (1998) developed a framework for modelling erosion and sediment transport in a large drainage basin which is sufficiently management-oriented to be able to provide useful indications of the on-site and off-site effects of climate and land use on soil erosion. The framework is not so empirical as to unduly limit its ability to evaluate management options. This modelling approach has the following advantages: it is temporally continuous and the time step can be selected short enough for the model to be event responsive; it attempts to separate climate, landscape and land use effects; it deals with gully and stream-bank erosion as the predominant sources of sediment, but can be combined with sheet and rill erosion.
models; assesses on-site (erosion) and off-site (water quality) effects; it is based on an intermediate scale (on the order of 100 km$^2$) for validation, but allows sub-scale variations in land cover/use to affect parameters at the intermediate scale, and uses routing to obtain discharge and suspended sediment at larger scales (avoiding the scaling up problem of a plot-scale approach); and is anchored to field measurements.

Stefano Ferraris et al. (1998) validated a quasi three-dimensional model of overland flow and soil erosion phenomena by comparing numerical investigations with the results of an ad hoc designed field experiment. The model employed couples a shock capturing finite-volume approximate solution of a kinematic wave and soil transport rill model with a two-dimensional finite element solution of a variably saturated Richards’ equation on the vertical plane perpendicular to the rill. The results showed that the approach allows an effective numerical simulation of rill soil erosion phenomena. Nevertheless, Stefano proposed that some improvements in the numerical model are required in order to treat low discharges and drier initial conditions.

David Favis-Mortlock et al. (1998), on the other hand argues that no current model is capable of explicitly forecasting the location and subsequent evolution of hillslope rill erosion systems in real landscapes. He attempted to develop and validate a self-organising dynamic systems approach to hillslope rill initiation and growth. He arrived at a conclusion that his approach, when applied to runoff routing at the scale of microtopography, can explain and indeed predict the spatial patterning of erosional features on a hillslope. However, the model has some limitations, notably: many process descriptions (infiltration, deposition, etc.) are omitted completely; the model demands a great deal of data regarding microtopography; and the approach is very demanding in terms of computational resources.

Gerlinger and Scherer (1998) simulated soil erosion and phosphorous transport on loess soils using advanced hydrological and erosion models. It was demonstrated that the temporal variability of the erosion resistance is less important than the spatial variability. Soil properties such as clay content, amount of organic matter and moisture content, which influence aggregate stability and crusting, seemed to be suitable in revealing the relative spatial differences of erosion resistance. The simulation results were compared with measurements and showed that the model was suitable for simulating soil erosion and phosphorus transport in loess soils.

Hrissanthou (1998) compared two mathematical models for computation of sediment yield from Kossynthos basin and found out that the degree of conformity between the annual values of sediment yield at the basin outlet according to both models was satisfactory. The lack of sediment data was the main reason for applying two different mathematical models to the study basin. The small deviation between the results of both models was an indication for the size order of the computed sediment yield. However, drawbacks for this modelling chain were reported as: (1) the temporal development of the physical processes over the considered time period was not followed. The models computed only total values of runoff, surface erosion and sediment transport, (2) the equations used for surface erosion and sediment transport were not adapted to local conditions; especially the equations for surface erosion were developed for small experimental fields, (3) gully and bank erosion were neglected.

Sharma (1998) developed a sediment delivery model for estimating the sediment delivery rates in an arid upland basin. The model uses a steady-state sediment continuity equation and first-order reaction model for deposition, since the initial potential sediment load is greater than the overland flow transport capacity in arid regions (as calculated by the Yalin method).
The results indicated that the model can simulate the spatial sediment delivery. Identification of vulnerable areas to erosion with a drainage basin is plausible through interfacing GIS technology and a distributed parameter sediment delivery model.

Environmental and hydrological implications of developing multipurpose reservoirs in some Kenyan catchments was assessed by Ongwenyi et al. (1993). His focus was on Tana and Athi river drainage basins. He stressed the fact that the development of multipurpose reservoirs would be the best measure of meeting Kenya’s water demands by the year 2020. However, environmental problems like soil erosion and silting of dams could curtail these efforts.

A study carried out on Masinga catchment by Pacini (1994) showed that soil erosion in the catchment is on the increase because of the rapid deforestation in the area. Most of the sediments coming from the catchment end up in Masinga reservoir. Pacini postulated that increased soil erosion would continue for years and thus higher sediment loads will be carried into the reservoir, if proper soil conservation measures are not put in place. Considerable siltation is expected, especially below Tana Bridge and in the upper Thika branch of the reservoir (Fig. 5.2). This research is aimed at predicting the daily amount of sediment yield from Masinga catchment based on climatic, topographic, and soil properties of the area. The information obtained from erosion modelling is then used to model sedimentation processes in the reservoir.

### 2.2 SUSPENDED SEDIMENT TRANSPORT

#### 2.2.1 Suspended Sediment Load

A reservoir is body of water, either natural or man-made, used for storage, regulation and control of water resources. When a river enters the reservoir, flow velocity decreases and the sediment load begins to deposit. Consequently, the sediment transport capacity of flow also decreases and sediment deposition increases in the flow direction. The grain size of deposited particles is reduced as the flow transport capacity decreases, and as a result the median diameter of bed material decreases in the flow direction (Müller, 1985). Coarser grains, transported as bed-load, are deposited close to the beginning of the backwater effect, in the far upstream region from the dam. Finer grains, transported as suspended load, are carried far downstream to the dam (Figure 2.1). Deposition takes place if the suspended sediment concentration is larger than the equilibrium concentration, $C > C_e$. This equilibrium concentration varies with flow conditions. As this concentration decreases along the flow direction deposition rate increases whereas erosion rate decreases. The maximum deposition occurs where the bottom shear stress is smaller than the critical shear stress for erosion, $\tau < \tau_{c,E}$. In the middle region, both sedimentation and erosion occur.

Sediment transport can be classified as bed-load, suspended load or wash load based on the type of transport. Particles transported as bed-load or as suspended load is dependent not only on particle size but also on the local flow strength. This means that a particle may be carried as bed-load in one reach and as suspended load in another section, where the flow conditions are different (Figure 2.2).

In most rivers the suspended sediment load is the most significant in terms of quantity. This may be due to the fact that non-cohesive fine sediment, such as silt and fine sand, has lower critical shear stress of entrainment (erosion). Even at low flow, suspended load almost always exists. During typical flow conditions, such as during long period between two floods, suspended sediment concentrations greater than 0.1 kg/m$^3$ or more often occur (Reid and
Frostick, 1994). For instance, average annual sediment concentrations are 0.05 kg/m$^3$ in most rivers in Germany; 1.14 kg/m$^3$ in the Yangtze River and 36.9 kg/m$^3$ in the Yellow River in China. Their maximum sediment concentration are 4.5 kg/m$^3$, 10.5 kg/m$^3$ and 590 kg/m$^3$, respectively. In contrast, bed-load transport may contribute less than 1% of total sediment transport in some streams.

2.2.2 Suspended Sediment Transport for Uniform Sediments

2.2.2.1 Critical Shear Stress for Erosion (Resuspension)

The criterion for initial instability of uniform bed grain was experimentally determined by Shields (1936), which is represented (Figure 2.3) by a relationship between the critical mobility parameter

$$F_c = \frac{u_{cr}^2}{\rho} = \frac{\tau_c}{(\rho_s - \rho)gd_{s0}}$$

and the particle parameter

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\[ D_s = d_{50} \left[ \frac{\rho_s - \rho}{\rho} \cdot \frac{g}{\nu^2} \right]^{1/3} \]  

(2.2)

**Figure 2.3: Critical shear stress for erosion and suspension. After van Rijn (1984)**

The critical shear stress in Shield’s curve is defined as the flow stage where 50% of the bed material starts moving. From Figure 2.3, one can see that the critical mobility parameter strongly depends on the particle parameter. Sediments with very small values of \( D_s \) usually start moving in suspension from rest over the bed surface. Sediments with larger values of \( D_s \) are initially suspended from moving bed material, which is transported as bed-load near the bed surface.

A lot of effort has been put into investigating the initiation of suspension based on the different assumptions. Xu (1998) shows part of these research results (Figure 2.3).

### 2.2.2.2 Erosion Rate

Various formulas are available for calculating erosion rates. Generally the erosion rate is represented as a function of flow and river bed material conditions. For uniform sediment a total erosion rate is determined by using an equivalent diameter, usually the median diameter.

Parchure and Mehta (1985) suggested a formula for determining the erosion rate which is related to the effective shear stress for erosion of fine and cohesive particles. The effective shear stress is given by the amount of bottom shear stress, \( \tau \), that exceeds the critical shear stress for erosion, \( \tau_{c,E} \). In other words, erosion occurs only when the bottom shear stress is greater than the critical shear stress. The critical shear stress is given as

\[ \tau_{c,E} = \rho \mu_{cr}^2 \]  

(2.3)
The expression of particle erosion rate is then given by

\[ E = k(\tau_{c,E} - \tau), \quad \tau > \tau_{c,E} \]  

(A.4)

A power-law expression is used to consider the effect of consolidation of cohesive sediment on erosion rate (Ariathurai and Arulanandan, 1978; Cormault, 1971; Kuijper et al., 1989) as

\[ E = \begin{cases} \frac{M(\frac{\tau}{\tau_{C,E}} - 1)^n}{1} & \tau > \tau_{C,E} \\ 0 & \tau \leq \tau_{C,E} \end{cases} \]  

(M.5)

\( M \) is a parameter to be calibrated. \( M \) has the same unit as erosion rate and varies with the critical shear stress (Ariathurai and Arulanandan, 1978). It has a range of \( 1.6x10^{-5} \) kg/m².s to \( 1.38x10^{-3} \) kg/m².s (Mehta, 1989). The value of \( n \) in this equation varies between 1 and 4. This means that erosion rate is not linear with the effective shear stress, \( \tau - \tau_{c,E} \).

Equation (M.5) is available for calculating the erosion rate of fine cohesive sediments. It can also be used for determining the erosion rate of non-cohesive sediments since this expression includes the main factor, effective shear stress which dominates erosion.

2.2.2.3 Sedimentation Rate

The sedimentation rate, \( S \), is related to the reference concentration of suspended sediment at a distance, \( a \), above the bed surface, \( C_a \), the particle fall velocity, \( \omega_s \), and the bottom shear parameter:

\[ S = \begin{cases} \frac{\omega_s C_a}{(1 - \frac{\tau}{\tau_{C,S}})} & \tau < \tau_{C,S} \\ 0 & \tau \geq \tau_{C,S} \end{cases} \]  

(F.6)

where, \( \tau_{c,S} \) is the critical shear stress required for initial deposition of suspended sediment particles. Deposition occurs only when the bottom shear stress is smaller than the critical shear stress, \( \tau_{c,S} \). Considering that the flow capacity for suspended sediment transport is closely related to the energy of turbulent motion, the expression for the critical shear stress, \( \tau_{c,S} \), is experimentally determined (Westrich and Juraschek, 1985):

\[ \tau_{c,S} = \frac{\rho_s - \rho g hw_s C}{\rho_s T_K U} \]  

(F.7)

where \( T_K \) is an efficiency parameter related to the boundary conditions, movable or non-erodible boundaries, permeable and non-permeable walls. For non-erodible bed conditions, a value of 0.0018 for \( T_K \) has been suggested by Westrich and Juraschek (1985).

Many researchers have suggested that the reference concentration, \( C_a \), is a function of hydraulic parameters and sediment properties. Some researchers have proposed that the reference concentration is proportional to the bed-load rate within the near-bed layer, using the assumption that there is a continuous exchange between the bed-load and suspended-load (Bridge and Dominic, 1984; Einstein, 1950; Van Rijn, 1984). The proportional coefficient can

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be determined experimentally. Others believe that the reference concentration is related to the excess bed shear stress or shear velocity (Celik and Rodi, 1984; Celik and Rodi, 1985; Smith and McLean, 1977; Yang, 1982).

There are also several hypothesis about the concentration reference height, \( a \). Some researchers believe that \( a \) is related to water depth, \( a = 0.05h \), the characteristic bed roughness, \( a = k_s \) (van Rijn, 1984), half the bed-form height, \( a = \Delta/2 \) (Holly et al., 1985), or two grain diameters, \( a = 2d \) (Einstein, 1950). Others supposed that \( a \) may be related to the effective shear stress with a grain diameter and characteristic bed roughness (Smith and McLean, 1977).

### 2.2.3 Suspended Sediment Transport for Non-uniform Sediment Mixtures

Sediments in a natural river are non-uniform with the sediment gradation varying continuously. For most river systems a pattern within downstream reaches containing more fine sediments is often found due to selective transport. For example, this pattern was found at the Lauffen Reservoir on the River Neckar (Mayer, 1988; Müller, 1986), even though the sediments included cohesive clay, the entrainment of which may be limited by consolidation.

Selective transport plays an important role in non-uniform sediment transport processes. It affects the changes in river morphology by varying the compositions of bed material and transported sediment. For several decades, considerable effort has been given to the study of size selectivity in the entrainment and transport of non-uniform sediments for gravel bed load (Gessler, 1967; Gessler, 1970; Hsu and Holly Jr, 1992; Little and Mayer, 1976; Proffitt and Sutherland, 1983; Sutherland, 1987; Wang et al., 1993) and fine suspended sediments (Colby et al, 1956; Maper, 1969; Müller, 1985; Xu and Westrich, 1997; Yalin and Krishnappan, 1973).

#### 2.2.3.1 Methods for Calculation of Fractional Entrainment

Many approaches have been suggested for calculating the entrainment rate for sediment mixtures. To calculate the entrainment of non-uniform sediment, both the amount and the grain size distribution of entrained particles need to be determined. Generally, there are two methods to obtain these values: probabilistic method and the deterministic method.

Einstein (1950) was the first to introduce the probability concept, in sediment transport research and to use the concept to develop the bed-load and suspended-load formulas for non-uniform sediment. He found that the Hydrodynamic lift caused by turbulent flow is attributed to the particle movement and entrainment. In a study on the armouring effects developed at the bed surface, Gessler (1970) developed an incipient motion model in which the shear fluctuations and critical shear stress were correlated. In this model, Gessler assumed that fluctuating bottom shear stress can be described by a Gaussian distribution and erosion occurs only when the instantaneous bottom shear stress exceeds a critical shear stress. He defined the probability of the local shear stress, \( \tau \), not exceeding the critical shear stress, \( \tau_c \), \( q_c \), as:

\[
q_c (\frac{\tau}{\tau_c} < 1) = \frac{1}{\sqrt{2\Pi}\sigma} \int_{-\infty}^{\tau/\tau_c} \exp\left(\frac{-x^2}{2\sigma^2}\right)dx
\]  

(2.8)
in which $\sigma$ is standard deviation of the shear stress fluctuation. For coarse material, $\sigma = 0.57$. Considering a given sediment grain size fraction $n$, the probability that the instantaneous bottom shear stress does not exceed the critical shear stress for erosion of that size fraction of sediment mixture is represented by $q_{\tau_{cn}}$. Thus, $(1-q_{\tau_{cn}})$ represents the probability of local bottom shear stress exceeding this critical value, $\tau_{cn}$. If $p_n$ represents the percentage of size fraction $n$ in the sediment mixture, the percentage of eroded material for this size fraction $n$ at the cross section considered can be defined as

$$P_{n,E} = \frac{\int_{n_{min}}^{n_{max}} (1-q_{\tau_{cn}}) p_{bn} dn}{\int_{n_{min}}^{n_{max}} (1-q_{\tau_{cn}}) p_{bn} dn}.$$  \hspace{1cm} (2.9)

The grain size distribution of the armour coat can then be defined by

$$P_{n,A} = \frac{\int_{n_{min}}^{n_{max}} q_{\tau_{cn}} p_{bn} dn}{\int_{n_{min}}^{n_{max}} q_{\tau_{cn}} p_{bn} dn}.$$  \hspace{1cm} (2.10)

where $n_{max}$ and $n_{min}$ denote the maximum and minimum grain size of a mixture, respectively, and $p_{bn}$ is the percentage of grain size fraction $n$ on the bed surface.

Based on the sediment-mixture experiments of Yen (1988), Hsu and Holly (1992) suggested that the percentage of a given grain size fraction in the transported material is proportional to the joint probability of two factors: the mobility of this size fraction for the prevailing hydraulic conditions and the availability of this size fraction on the bed surface. The availability of a given size fraction $n$ on the bed surface is represented by $p_{bn}$. They used the same assumptions as Gessler. In their model, the grain size distribution of the eroded material is given by

$$P_{n,E} = \frac{q_{\tau_{cn}} p_{bn}}{\sum_{D_{min}}^{D_{max}} q_{\tau_{cn}} p_{bn} d(D)},$$  \hspace{1cm} (2.11)

Where $D_{max}$ and $D_{min}$ are the representative diameters of the largest and smallest size fractions, and $q_{\tau_{cn}}$ is the probability of the local shear stress exceeding $\tau_{cn}$, expressed by

$$q_{\tau_{cn}} (\frac{\tau_{cn}}{\tau} > 1) = \frac{1}{\sqrt{2\pi\sigma}} \int_{\frac{\tau_{cn}}{\tau}}^{\infty} \exp\left(-\frac{x^2}{2\sigma^2}\right) dx.$$  \hspace{1cm} (2.12)

Similarly, Li suggested a formula to describe the sedimentation and erosion for sediment mixtures considering fluctuating velocity of turbulent flow (cited by Zhou (1995)). In addition to the use of probability theory to take into account turbulent flow properties, many deterministic methods have also been developed to calculate the entrainment rates for non-uniform sediments. Based on field and flume data, Samaga et al. (1986) and Holtroff (1983)

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introduced some modifications to Einstein’s method. Swamee and Ojha (1991) studied the properties of grain-size distribution curves and extended the formulas for calculating transport rate for uniform sediments to the non-uniform sediment mixtures. Some researchers directly apply the formulas for uniform sediment to the calculation of non-uniform sediment transport (Rahuel et al., 1989).

A variety of sediment transport formulas are available. Vanoni (1975) used several formulas to predict sediment transport on the Colorado River and the Niobrara River. He found large differences between the estimated sediment discharges. A given formula predicted sediment discharge adequately for one river, but very poorly for the other river. Yang and Molinas (1982), Shen and Hung (1983), van Rijn (1984) and Nakato (1990) also compared predicted and measured flow and sediment data in natural rivers. They all concluded that a few formulas could predict the sediment discharge reasonably well for the flow conditions tested but that all the other formulas were found to over-predict or under-predict the sediment discharge. It is extremely difficult to find one formula that is widely applicable. The selection of formulas for predicting sediment discharge in movable-bed rivers must be related not only to the physical phenomena of interest but also to the range over which the selected formula is valid (Nakato, 1990).

The total sediment transport includes the bed-load and the suspended load. Bed-load transport is saturated sediment transport, whereas suspended sediment transport is unsaturated. The continuity equation of total-load transport is:

\[
\frac{\partial(G_b + G_s)}{\partial x} = \rho_p \frac{\partial z}{\partial t} + \frac{\partial hC}{\partial t}
\]

(2.13)

where \(G_b\) and \(G_s\) are transport rates for bed-load and suspended load, respectively. The difference between these two kinds of transport loads can be understood by comparing their continuity equations of sediment transport. The continuity equation for fractional bed-load transport is

\[
\frac{\partial G_{bn}}{\partial x} = -\rho_p \frac{\partial y_n}{\partial t}
\]

(2.14)

\[
G_{bn} = f(h, \tau, \tau_{cd}, d_n, \gamma_s, \gamma, g)
\]

(2.15)

The river bed changes, sedimentation and erosion are included in the value \(\Delta y_n\), which is determined by the difference in transport capacities between two adjacent sections. The sectional transport rate is related to the flow and bed conditions. A positive value of \(\Delta y_n\) indicates sedimentation while a negative value of \(\Delta y_n\) indicates erosion. The mass balance of the control volume between two adjacent cross sections is used for calculating the bed change within \(\Delta x\), by summing sedimentation and erosion. For fractional erosion the transport rate is limited by the mobility and availability of bed material of a given size fraction, which depends only on the local flow and bed conditions.

However, suspended sediment transport is unsaturated, fractional entrainment (erosion rate) is not only related to sediment inflow, outflow, the mobility and availability of bed material, but also to suspended concentration. Sediment deposition depends on the sediment fall velocity and the suspended sediment concentration near the bed surface. The continuity equation for suspended sediment transport is expressed by:

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The concentration and the sedimentation rate must be determined by convection-dispersion equation of suspended sediment transport. Obviously, the fractional entrainment cannot be determined using the method for bed-load transport. As pointed out by Rahuel et al (1989), in some earlier efforts, the lack of success in modelling the sediment transport processes could be related to over simplified treatment of sediment transport. Considering effect of turbulence on sediment transport, Nezu and Nakagawa (1994) stated that ‘the classical stochastic model and its extension formula cannot be universally applied to rivers carrying suspended sediment. Their weakness is that they do not incorporate modern turbulence knowledge. The random character of the motions of sand particles is regarded as more significant than the flow in such stochastic models. As a result, they are not based on the mechanics of turbulence but use only the continuity equation for sediment transport and the parametric probability density functions. In this sense, these stochastic models are not hydrodynamic models, but are really kinematic models’.

2.2.3.2 Sediment Exchange Near the Bed Surface

In sediment transport, exchange between transported sediments and river bed material occurs when the bottom shear stress exceeds the critical shear stress. To describe the exchange one usually divides the bed into several horizons (Figure 2.4) according to the moving manner and position of sediment particles (Alonse et al., 1976; Bayazit, 1975; Bennett and Nordin, 1977; Lee and Odgaard, 1986; Park and Jain, 1987; Rahuel et al., 1989; Ribberink, 1987; Silvio, 1991; van Niekerk et al., 1992).

(1) Flow zone, in which suspended load is transported along with water flow.
(2) Bottom layer, in which bed-load is transported. In this region, the particles move mainly by rolling and sliding.
(3) Active layer, from which sediment particles may be entrained into the bottom layer or the flow zone as suspended sediments. Because of the existence of exchange between the bottom layer and this layer, it is also called the mixing layer in some models.
(4) Parent layer, beneath the active layer. The bed material in this layer is not involved in transport calculations in the current time step, but supplies the material to the active layer to which the calculation may refer.
(5) Immobile layer, in which bed material is not eroded in the flow transport process.

Moving particles in the bottom layer can enter the flow at the interface between the bottom and the suspension layers, i.e., bed-load and suspended load layer. Although in reality it is hard to position this interface, it is useful to define a boundary between the bed-load and suspended load layers for numerical calculation.

The parent layer may need to be further divided into layers if the bed material is mixed with partial cohesive clay. This is because the compaction and consolidation of cohesive sediments varies with depth under the bed surface and age of the deposited sediments. The compaction and consolidation influence the cohesive sediment entrainment (Mehta, 1989; Partheniades, 1965; Teisson, 1991; Tsai and Lick, 1988).
Rahuel et al. (1989) used the concept of the mixing layer in their fractional bed-load transport model to deal with the effects of sorting and bed armouring. They used a hiding factor to correct the overestimation of entrainment of fine bed material. The sediment exchange between the particles in the bottom layer and mixing layer is considered only in the current time step of calculation. The material exchange between the mixing layer and the underlying parent layer depends on the difference in thickness of the mixing layer, $T_a$, and the erosion thickness, $E_a$. When $T_a < E_a$, some of the underlying layer material will fill the mixing layer. Conversely, when $T_a > E_a$, some of the mixing layer material will appear in the underlying layer. Such a mixing layer has a quasi fixed upper boundary at the bed surface and a moving lower boundary. Its thickness is determined by flow conditions and the grain size distribution of bed material. This kind of model has been widely used (Bennett and Nordin, 1977; Boral et al., 1982; Borah et al., 1982; Karim and Kennedy, 1981).

Rahuel et al. (1989) stated that the mixing layer thickness is strongly dependent on the time scale considered. For short time scales, the mixing layer can be thought of as a thin surface layer, in which particles are susceptible to entrainment into the flow due to the fluctuation of the bed shear stress. For longer time scales, the mixing layer can be treated as a zone of bed-form movement (dunes and ripples), or a layer of eroded or deposited material.

Silvio (1991) proposed a four-layer model for total load transport calculation. He defined the four layers as: water stream, bottom layer, mixing layer and intrusion layer. The last two layers correspond to the active layer. He reasoned that since the surface of gravel rivers has coarser composition than the material underneath, the composition of transported material should be related to that of the material beneath, under non-equilibrium transport conditions.

Another alternative method is to consider a hiding and exposure effect. He concluded that this method may be more useful for the study of sediment transport of widely graded sediment mixtures.

The height of the bottom layer is usually defined as a proportion of the water depth or as a thickness of transported bed-load (van Rijn, 1984). The height of an active layer is generally related to the thickness of bed-forms (Lee and Odgaard, 1986), the representative diameter of bed material (Borah et al., 1982) or the critical shear stress for erosion (van Niekerk, 1992).
2.2.4 Flood Plain Sedimentation

2.2.4.1 Interaction Between the Flow in the Main Channel and the Flood Plains

When overbank flow occurs in a channel with compound cross-sections, the flow velocity in the deeper main channel will be significantly different from that on the shallow flood plains. This yields strong interactions between the flow in the main channel and the flood plains. Momentum is transferred from the main channel to the flood plain. It results in a decrease of the velocity in the main channel and an increase of that over the flood plains and generates turbulent shear at the interface. In the interaction zone there exist vortices with longitudinal axis, called secondary currents. In trace experiments Wood and Liang (1989) observed that the tracer moved either onto or off the flood plain. Imamoto and Ishigaki (1992) demonstrated this phenomenon by using the visualisation technique. These secondary currents are driven by the anisotropy and inhomogeneity of turbulence (Tominage and Nezu, 1991).

At the interface between the main channel and the flood plain, the turbulent shear stress, $\tau_{zx}$, generated by the difference in velocity between the main channel and the flood plain can be described by the Reynolds shear stress formula as follows

$$\tau_{zx} = -\rho \varepsilon \frac{\partial u}{\partial z}. \tag{2.18}$$

The turbulent diffusion coefficient is usually related to a horizontal length scale and the velocity which may vary across the shear layer (Alavian and Chu, 1985). It can be expressed by

$$\tau_{zx} = -\rho \varepsilon_{mp} \frac{u_m - u_p}{B} \tag{2.19}$$

An expression to determine $\varepsilon_{mp}$ was suggested by Evers (1983), in which the difference of velocities between the main channel and the flood plain, $u_m - u_p$, is taken as a velocity scale and the width of the interaction zone, $B_{mp}$, as length scale. Then, the turbulent diffusion coefficient $\varepsilon$ has the following form

$$\varepsilon = 2\kappa (u_m - u_p) \frac{\partial u}{\partial z} B_{mp} \tag{2.20}$$

where $\kappa$ is a proportional factor. Under the assumption that $\tau_{zx}$ has a linear distribution in the lateral direction, integrating the above equation on the x-z plane, the turbulent shear stress, $\tau_{zx}$, can be obtained

$$\tau_{zx} = 2\rho \kappa (u_m - u_p) |u_m - u_p| \tag{2.21}$$

For practical application, the coefficient $\kappa$ is experimentally suggested as 0.01 by Evers (1983) and proposed as 0.01 – 0.02 by Kohane (1991).
Yen et al. (1985) derived a quasi-one dimensional method for backwater computation in a compound channel. He used the continuity and momentum equations for the main channel and the flood plains separately and took into account the effect of the lateral discharge and interfacial shear stress between them. Based on his experimental results, he concluded that the effect of interfacial shear stress on the backwater profile is insignificant compared to the effect of lateral discharge. Kohane (1991) predicted the effect of the interfacial shear stress on the backwater profile for a 32 km reach upstream of the River Neckar using Yen’s method. Comparing the results with a one-dimensional model, he found that the water surface calculated by Yen’s method was higher than that calculated by the one-dimensional model. The maximum difference between them is at the sections with large changes in cross-sectional geometry or strong diverging flow. He concluded that in these cases the computation of a water surface by one-dimensional model was not accurate enough. A one-dimensional multiple strip model should be used, even though this model requires a great computational effort.

Pasche (1985) and Alavian (1985) developed $\kappa$-$\varepsilon$ models (two-dimensional) to describe the exchange of momentum and mass between the main channel and the flood plains. Pasche found that the non-dimensional lateral depth averaged diffusion coefficients are very different in the main channel and the flood plains (Figure 2.5).

Many experimental results have shown that the interaction between the main channel and the flood plain flow significantly affects the transverse dispersion, and consequently lateral transport resulting in the sediment deposition on the flood plain. Therefore an expression for lateral diffusion coefficient in the interaction zone of the main channel and the flood plain is desirable. Xu (1998) derived a method to evaluate such a diffusion coefficient.

![Figure 2.5: Bottom shear stress distribution in a compound section. After Pasche (1985)](image)

### 2.2.4.2 Sediment Deposition on Flood Plains

Over-bank flow results in a transfer of mass, momentum and energy between the faster flow in the deep main channel and the slow flow on the shallow flood plain. On the flood plains, transport capacity diminishes due to lower velocity and increasing flow resistance, and sediments rapidly settle (Pizutto, 1986).

The sediments deposited on the flood plains are generally finer than those occurring close to the active channel (Reid and Frostick, 1994). Flow patterns and deposition rates depend on the flood plain topography and vegetation (Kessel et al., 1974). Vegetation on the flood plains enhances deposition. The deposition rate and amount are of great environmental significance since sediment bound pollutants can be deposited there. During flood events sediments...
accumulated on the flood plain may be re-suspended and transported again (Macklin et al., 1992).

The sedimentation rates calculated by different methods range from 1 millimetre to a few centimetres per year. Measured sedimentation rates over 35-year period in Southern Norway, were between 0 – 10 mm per year for the Culm River, and between 3 and 40 mm per year for Severn River. For both cases, the greatest deposition occurred near the channel (Walling and Bradley, 1989). Obviously, there is considerable variation in the lateral sedimentation rate on the flood plain.

According to an analysis of river flow records, Leopld and Naddock (1953) suggested that bank-full discharge had a recurrence interval of about 1.5 years. Over-bank flow can therefore be expected to take place with about the same frequency. Because channel and flood plain sediments are distinctly different, this point on the magnitude-frequency plot is very significant. This means that for any river, the channel geometry is adjusted to cope with floods of moderate magnitude with the same frequency. It has been proved that the formation of channel geometry is controlled by bank-full flows. The bank-full flow can be frequent and powerful enough to erode banks and river beds. Floods have almost little effect on the channel form, as they rarely occur and over-banking flow greatly reduces the flow power to erode the river bed and the banks (Leopld, 1994).

In fact, a flood plain represents a balance between the erosion energy and the alluvial resistance (Nanson, 1986). Leopld and Naddock suggested a general theory of flood plain formation related to both deposition on the flood plain and lateral bank erosion (Leopld, 1994).

2.3 MODEL RESEARCH SITUATION

2.3.1 General Remarks

The numerical simulation of sediment transport processes in river channels and reservoirs is an important tool to see how the channel characteristics and morphology respond to changes caused by natural or human interference. The main objects of simulation of sediment transport processes are shown in Figure 2.6. They involve: simulation of changes in river/reservoir morphology, description of the effect of selective transport on the sediment transport process and assessment of the impact of river drainage works or reservoir flushing, etc.

In the past 35 years, modern computational techniques with enhanced computer capacity have rapidly advanced sediment transport modelling. Considerable effort has been given to develop numerical methods for simulating sediment transport processes in river channels and reservoirs. Different numerical models have been suggested. These models can be classified according to

(1) Numerical method: finite differential method (FDM), finite volume method (FVM), finite element method (FEM), characteristic method (CM);
(2) Dimension: one-dimensional (1D), two-dimensional (2D), three-dimensional (3D) models and quasi two-dimensional or three-dimensional models (multiple strip model and stream tube);
(3) Physical properties considered in the problems: two main classes in particle transport models: particle dispersion and sediment transport. The latter can be further subdivided according to different transport loads: bed-load, suspended load and total load models.
Based on different methods for treating bed material we have: uniform sediment and non-uniform sediment mixtures.

The selection of computational methods is usually connected with the selection of model dimensions, and model dimensions are closely relate to the problems.

One-dimensional numerical models have been extensively applied to simulate longitudinal river/reservoir morphological changes, such as aggregation and degradation upstream and downstream of reservoirs. Most of the simulation for sediment transport in rivers use one-dimensional models because of their shorter computational time, lower cost and small data requirements. 1D numerical models are usually sufficient, but they only give sectional averaged bed elevations. If more detailed information about river/reservoir morphological changes are needed, 2D or 3D models are required.

A frequently applied computational method for 1D problems is the FDM. FDM may be explicit or implicit. An implicit scheme is unconditionally stable, so only the accuracy should be considered. An explicit scheme is conditionally stable and must satisfy the Courant criterion (Section 3.3.4.3). Generally, using the implicit scheme, a longer time step is allowed, whereas using the explicit scheme the time step is limited by the Courant criterion (Schönung, 1990).

2D and 3D models are also known as horizontal and vertical models, respectively. 2D depth-averaged flow and sediment transport models have been widely developed for simulating river bed changes at river confluences and sediment deposition in reservoirs and lakes. 3D models are used for investigating the detailed changes at important positions. However, only a

---

Figure 2.6: Framework of sediment transport modelling

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few studies based on the 3D calculations have been reported because of the complicated
mathematics and expensive computer time requirement.

Frequently used computational methods include FDM, FVM and FEM. The basic
approximation principle of finite-difference and finite-volume methods is the same. In the
finite difference method, differential quotients of a differential equation are directly
approximated. In the finite volume method, differential quotients are integrated over a control
volume. In FVM, the calculation domain is divided into a number of non-overlapping control
volumes, surrounding each grid point. Through integrating the differential equation over a
control volume, a conservative discretization equation is obtained, which contains the value of
unknown variables for a group of grid points (Patankar, 1980).

In the FEM the unknowns are approximated on the basis of nodes which are connected to
each other by means of an element. Inside the elements the values of the unknown are
evaluated by interpolation. Since the governing differential equation will not be met by the
interpolated unknowns everywhere, a local residue occurs. By applying a weighting function
and integrating over the volume of the element, the residue inside the element is redistributed
to the nodes. The sum of the weighted and integrated residues of all elements solved by a
node must equal zero. Different choices for the functions of interpolation and weighting lead
to different sub-schemes of the FEM such as the Galerkin method or the Petrov-Galerkin
method (Schönung, 1990).

The FEM is very flexible with respect to combining different element types such as a triangle
and quadrilaterals to unstructured grids, so irregularly shaped domains can be approximated
by the method. In principle, this is also possible using the FVM. The most important
disadvantage of the FEM is that mass balance is not met locally, whereas the FVM is strictly
conservative. Furthermore, the derivation of equations is easier in the FVM than in the FEM.

Multiple-strip models and stream tube models are intermediary classes of models between the
1D and 2D models. In these models, a channel is divided into several parallel strips according
to different principles. Multiple-strip models are generally used for simulation of water
surface for overbank flow and deposition on the flood plains during flood events. Therefore
they usually include three strips, one for the main channel and two for the flood plains. In
stream tube model, a channel is divided into a certain number of stream tubes that have an
equal portion of discharge. No flow is allowed to cross into adjacent stream tubes. Hydraulic
parameters and particle movement are simulated for each stream tube.

Quasi 2D models consist of a stream tube flow model and a 2D suspended sediment transport
model. The stream tube flow model provides a quasi 2D flow field. Based on this flow field,
2D phenomena of sediment and pollutant transport can be described approximately.

2.3.2 Overview of the Present Models

In the past decades, many numerical models for the simulation of sediment transport in river
channels have been developed. Some well-known models are given in Table 2.1.

2.3.2.1 One-dimensional Models

The HEC-6 model was developed by the U.S. Army Corps of Engineers in 1977, and refined
in 1990 (Thomas and Prashum, 1977; HEC-6, 1977). It includes a steady-state hydrodynamic
model and a transport model for total sediment load. The finite difference method with a 6-
point iterative finite difference scheme is used to solve the sediment continuity equations. Non-uniform sediment properties and armouring effects in sediment transport processes are taken into account. A wide gradation of sediment grain sizes from clay to gravel are included. For sediment routing it provides three transport relations to be selected from.

<table>
<thead>
<tr>
<th>Author</th>
<th>Model</th>
<th>Year</th>
<th>Load</th>
<th>Problem</th>
<th>Dimension (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S Army corp of Engineers</td>
<td>HEC-6</td>
<td>1977-1990</td>
<td>Total (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>Chang &amp; Hill</td>
<td>FLUVIAL-11</td>
<td>1976-1982</td>
<td>Bed (nonu.)</td>
<td>√ √</td>
<td>1D</td>
</tr>
<tr>
<td>Holly et al.</td>
<td>CHARIMA</td>
<td>1985-1990</td>
<td>Bed (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>van Niekerk</td>
<td>MIDAS</td>
<td>1992</td>
<td>Bed &amp; Susp. (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>Yotsukura &amp; Sayre</td>
<td>SEDZL</td>
<td>1986</td>
<td>Bed &amp; susp. (nonu.)</td>
<td>√ √ √ √</td>
<td>2D Orthog. C.</td>
</tr>
<tr>
<td>Zegler et al.</td>
<td>SEDSL</td>
<td>1986</td>
<td>Total (nonu.)</td>
<td>√</td>
<td>Stream tube</td>
</tr>
<tr>
<td>Molinas &amp; Yang</td>
<td>GSTARS</td>
<td>1986-2000</td>
<td>Total (nonu.)</td>
<td>√ √ √ √</td>
<td>Quasi 2D Orthog. C.</td>
</tr>
<tr>
<td>Lee et al.</td>
<td>UGASTARS</td>
<td>1997</td>
<td>Bed &amp; susp. (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>Yung-Hai Chen</td>
<td>RESSED</td>
<td>1984</td>
<td>susp. (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>Yung-Hai Chen</td>
<td>TWODSR</td>
<td>1988</td>
<td>Bed (nonu.)</td>
<td>√ √</td>
<td>2D</td>
</tr>
<tr>
<td>Holly</td>
<td>SEDICOU-P</td>
<td>1986-1988</td>
<td>Bed &amp; susp. (nonu.)</td>
<td>√ √ √ √</td>
<td>1D</td>
</tr>
<tr>
<td>Lai</td>
<td>ONED3X</td>
<td>1987</td>
<td>Bed (u.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>Molinas</td>
<td>STARS</td>
<td>1983</td>
<td>Total (nonu.)</td>
<td>√ √</td>
<td>Stream tube</td>
</tr>
<tr>
<td>Karim</td>
<td>IALLUVIAL</td>
<td>1985</td>
<td>Bed (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
<tr>
<td>WES corp of Engineers</td>
<td>TABS2</td>
<td>1984</td>
<td>Total (u.)</td>
<td>√</td>
<td>2D</td>
</tr>
<tr>
<td>Ruh-Ming Li</td>
<td>HEC2SR</td>
<td>1980</td>
<td>Total (nonu.)</td>
<td>√</td>
<td>1D</td>
</tr>
</tbody>
</table>

**Susp.** Suspended  
**u.** Uniform sediments  
**nonu.** Non-uniform sediment mixtures  
**Orthog.c.** Using an orthogonal curvilinear co-ordinate system

HEC2SR was developed in 1980 by Dr. Ruh-Ming Li of Simons, Li and Associates, Inc. (SLA), is designed to simulate watershed sediment yield, aggradation, and degradation in river basin. It incorporates a sediment-routing program into the HEC2 program developed by Mr. William S. Eichert at the Corps of Engineers for calculating backwater profiles.

RESSSED was developed by Dr. Yung-Hai Chen. It is a simplified quasi-nonequilibrium model for reservoir and river erosion sedimentation related problems.
FLUVIAL model was developed for water and sediment routing by Chang and Hill (1976, 1977) and Chang (1982, 1987). The model includes river bed sedimentation and erosion, width variation, and changes caused by the curvature effect. The last two terms are closely connected with changes in river morphology because of an interrelation between changes in channel width and channel-bed profile. The concept of minimum stream power is used to determine river width changes. After the banks are adjusted, the remaining change in area is distributed to the river bed. For aggregating section, deposition is assumed to build up the bed in a horizontal layer. For a degrading section, the change is assigned in proportion to the effective tractive force, \( \tau - \tau_c \). The water routing is assumed to be uncoupled from the sediment process. The FDM is used as computational method.

The models IALLUVIAL (1982), CARICHAR (1989) and CHARIMA (1990) were developed by Karim, Rahuel, Holly et al. at the Institute of Hydraulic Engineering, Iowa in the last twenty years. These models describe the bed evolution of branched and looped channel systems. A Preismann’s finite difference approximation was used in the solution of the sediment transport equation. Armouring effects were considered in these models. In CARICHAR model, the concept of a mixing layer was used to treat non-uniform sediment mixtures and a hiding factor was introduced for bed-load transport. The flow and sediment transport equations are solved through a fully coupled, implicit finite difference scheme. It takes into account the interaction between flow hydrodynamics and sediment transport processes.

ONED3X is a coupled multi-mode method of characteristics (MOC) model developed in 1987 by Dr. Vincent Lai of the U.S. Geological Survey (Shou-Shan, 1988). Between 1986 and 1988, Dr. Forrest Holly, Jr. of the University of Iowa, developed SEDICOUP, a totally coupled program.

Another one-dimensional model was developed by van Niekerk, et al. (1992) to simulate river bed changes within a relative straight, non-bifurcating alluvial channel. The model includes density and size sorting of bed-load and suspended load for sediment mixtures. In the model the hydrodynamic part and the sediment transport part are uncoupled during each time step. Flow depth and velocities are determined from the gradually varied flow equation using the standard step method for backwater calculation. The transport of particles in suspension is modelled by a convection-dispersion sediment continuity equation, either explicitly solved by using the Rouse equation, or implicitly by using a finite-difference scheme. The vertical sediment distribution of the suspended load is computed in analogy to a parabolic distribution for momentum diffusion. A modified Bagnold formula (Bridge and Dominic, 1984; Engelund and Fredsoe, 1976) was used for the determination of the bed load transport rate for each size-density fraction. The percentage of transport rate for each size-density fraction is calculated according to the efficient bottom shear stress, the difference of instantaneous bottom shear stress and critical shear stress.

2.3.2.2 One-dimensional Multiple Strip Model

The 1D multiple strip model is an expansion of the 1D model. It is used for simulating water and sediment routing of a river channel with compound cross sections. Traditionally, the main channel and the flood plains are treated together as a single composite channel in computing backwater profiles of a flood flow. Yen (1984, 1985) developed a new quasi-1D backwater computation method that treated the main channel and the flood plain separately and considered lateral mass exchange, turbulent shear stress at the interface between the main

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channel and the flood plains and different bed roughness across the channel. His calculations showed that if the ratio of the flow depth in the flood plain and the main channel was smaller than one-half, the backwater profiles calculated by his new method differed from those generated using traditional methods. This difference was noticeable when the difference in bed roughness between the main channel and the flood plains was large and the width of the main channel was smaller than that of the flood plain.

Based on Yen’s method, Kohane (1991) developed a 1D multiple strip flow model, in which the main channel and flood plains were taken as different strips. The momentum exchange and turbulent shear stress on the interface between the main channel and the flood plain and the variety of bed roughness for each strip were taken into account. Based on Evers’ hypothesis (1983), Kohane developed a formula of the turbulent shear stress at the interface (Equation 2.21).

2.3.2.3 Stream Tube Models and Quasi Two-dimensional Models

Fischer (1967) was the first to use the stream tube model to simulate 2D dispersion problems in streams. It was subsequently refined by Chang (1971) and Sayre and Yeh (1973). Sayre and Yeh introduced a natural co-ordinate system in which a simplified equation was used to align the longitudinal co-ordinate surface perfectly in the direction of the depth-averaged local velocity vectors so that they form stream tube surfaces. They eliminated the lateral velocity in two-dimensional depth-averaged convection-dispersion equation to simplify the computation. Yotsukura and Sayre (1976) further improved the model by introducing cumulative discharge and metric coefficients. They proved that the surface corresponding to the constant cumulative discharge was tangential to the local velocity vector and thus acts as a stream tube surface. The significant feature of the cumulative discharge method is that it shifts the problem of calculating the transverse velocity from the convection-dispersion equation onto the co-ordinate system, whose configuration is constrained by the requirement that the water continuity equation is to be satisfied. This model was successfully used to simulate the solute dispersion in natural rivers.

STARS is a sediment transport and river simulation stream tube model. The model is an outgrowth of a model originally developed by Dr. Albert Molinas at Colorado State University and was submitted under contract to the Bureau of Reclamation in 1983. Bureau of Reclamation later modified the model to make it more efficient, flexible, and user-oriented (Shou-Shan, 1988).

GSTARS, Generalized Stream Tube for Alluvial River Simulation, first developed by Molinas and Yang (1986) for solving river engineering problems. It takes into account the bed-load transport of non-uniform sediment mixtures. In the calculation of fractional erosion the formulas of total load for uniform sediment (Ackers and White, 1973; Engelund and Hansen, 1972; Yang, 1973; Yang, 1984) were used first to determine the erosion rate for each size fraction. Then the erosion rate of size fraction n is determined by multiplying this determined erosion rate of the size fraction n and the availability of that size fraction in the bed material.

GSTARS can be used to simulate the complicated response of a river channel in terms of width and depth adjustment processes. It can also describe different types of channel development which could happen during morphological adaptation process of a river/reservoir channel.
GSTARS has been successful in simulating and predicting local scours as well as deposition across and along natural rivers with movable beds and fixed widths. For example, it was applied in the study on the Mississippi River Lock and the study on the local scour along the Willow Creek Dam unlined emergency spillway. The computation results demonstrated that GSTARS was able to simulate complicated morphological processes of natural rivers and might have the potential to simulate the process of river meandering.

GSTARS treated the total load as if it was an equilibrium transport conditions. In reality suspended sediment transport is not an equilibrium sediment transport process. Lee et al. (1997) modified the model of GSTARS by considering bed-load and suspended load separately, creating USTARS. For entrainment and deposition of suspended load, the expressions of van Rijn (1984), Rahuel and Holly (1989) were used. USTARS uses the same method as GSTARS to determine fractional entrainment. The convection-dispersion equation of suspended sediment transport was solved by FVM and the split operator approach. The equation is separated into four portions: convection-step, longitudinal dispersion-step, transverse dispersion-step and reaction-step. They are solved subsequently in one time step.

MABOCOST is a quasi-2D numerical model developed by Luk and Lau in 1987. It describes the mixing in natural sinuous streams with non-uniform cross sections and velocity distributions and simulates localized sources or sinks due to chemical and biological reactions.

In order to eliminate longitudinal numerical dispersion completely, variable-length elements developed by Beltaos (1978) were used in solving the sediment convection-dispersion equation. These elements were arranged in the parallel stream tubes. The length of each element is always equal to the product of the local velocity and the time step so as to satisfy the Courant criterion (Section 3.3.4.3).

### 2.3.2.4 Two- and Three-dimensional Models

To predict small scale river bed changes, such as river bed changes near hydraulic structures, local scours at bridge crossing, scour and fill in curved channel reaches, etc., 2D or even 3D models are required. With advancement in computer technology, many 2D models have been developed and have been widely used in the study of sediment transport processes, such as reservoir sedimentation, river morphological changes, the distribution of deposition and erosion (van Rijn, 1986; Zhou and Lin, 1995; Ziegler and Lick, 1986), and in estuaries, bays and river mouths (Larsen and Eisenhauer, 1992; Wang and Adeff, 1989; Wang et al., 1986).

The 2D sediment transport model, SEDZL, developed by Ziegler and Lick (1986) simulates reservoir deposition. To improve accuracy an orthogonal curvilinear co-ordinate system, transformed from Cartesian coordinates, is adopted in the model. SEDZL considers the effect of flocculation and consolidation for mixing sediment, cohesive fine sediment and non-cohesive coarser sediment, in sedimentation and erosion processes.

In order to consider the effect of grain size variation, particles are classified based on their size fraction as either cohesive sediments or non-cohesive sediments. The expressions of Gailani et al. (1991) and van Rijn (1984) are used to determine the entrainment for these two size fractions, respectively. The percentages of bed material and entrainment for each size fraction were assumed to be the same. The bed was vertically discretized into 7 layers to account for the fact that bed consolidation and compaction increases with depth (Parchure and Mehta, 1985; Patankar, 1979) within 7 days after deposition (Tsai and Lick, 1988). Each layer
is composed of the deposition in one day. When the time after deposition of the layer exceeds 7 days, it was treated as 7 days.

The model of SEDZL was used to simulate fine-grained, cohesive sediment transport in Watt Bar Reservoir, on the Fox and Pawtuxet Rivers (Ziegler and Lick, 1986; Ziegler and Nisbet, 1994 & 1995). The results proved that this model realistically predicts overall patterns of erosion and deposition throughout the reservoir.

In order to investigate the effect of the turbulent shear stress on the interface between the main channel and the flood plains, many 2D and 3D $\kappa-\epsilon$ models have been developed (Cokljat and Younis, 1995; Lin and Shiono, 1995; Nokes and Hughes, 1994; Naot et al., 1993; Wood and Liang, 1989). These models can simulate secondary flow and predict velocity fields and eddy diffusion.

A 3D numerical model of solute transport in compound channel flow has been developed by Lin and Shiono (1995). The numerical model used linear and non-linear $\kappa-\epsilon$ models to evaluate turbulent parameters and used the simpler method of Patankar and Spalding (1972) to solve the differential equations. According to the model, the transport behaviour near the interface between the main channel and the flood plains showed that the secondary flow was significant. Tracer concentrations simulated using $\kappa-\epsilon$ models agree with the experimental results better than those obtained by the linear model. The bed shear stress obtained by the two models agrees well with the experimental value by Wood and Liang (1989).

Naot et al. (1993) developed a 3D turbulent model to simulate the hydrodynamic behaviour of compound channels, which was originally advanced with reference to flow in a rectangular open channel. The approach of a second-order Reynolds stress closure is used to simulate secondary flow in this model. The computation results showed that the turbulent model can describe the main features of more complex flows by including the effects of asymmetry, flood-plain roughness, and sidewall inclination in compound channels. Without considering the vortices in the vertical direction, the close agreement obtained between the prediction and the measurement shows that this vertical eddy does not affect the process of momentum transport between the main channel and the flood plains (Xu, 1998).

A stream tube model, a quasi-2D model, and 2D model can be used for describing 2D suspended sediment transport processes and pollutant dispersion in rivers (Xu, 1998). In the stream tube model, the depth-averaged transverse velocity is equal to zero. The sediment transport is considered only in a single stream tube and the dispersion between the neighbouring stream tubes is ignored. In a straight channel with a uniform flow, the assumption and approximation are acceptable. However, for a river reach with secondary flow, caused by bend effects or local geometric changes, the application of stream tube model is limited. A quasi-2D model provides an improvement on the stream tube model by considering the transverse particle dispersion in the governing equations (Xu, 1998).

TABS2 is a 2D uncoupled model developed by a team of researchers at the Waterways Experiment Station of the Corps of Engineers over the period of 1977 through 1984 (Shou-Shan, 1988). It is used to study a variety of problems in river, lakes and estuaries. TWODSR was developed in 1988 by Dr. Yung Hai Chen. It is a 2D model based on an uncoupled, unsteady approach.
2.3.2.5 Summary of the Merits and Demerits of Selected Sedimentation Models

In this section, the capabilities and limitations of each of the 11 selected computer stream sedimentation models for sediment transport (Shou-Shan, 1988) are presented.

(I) HEC-6

Model Capabilities

1. HEC-6 can predict long-term trends of scour and deposition in a stream channel.
2. The model can also be used to predict reservoir sedimentation, degradation of the channel bed downstream from a dam, and the influence of dredging activities.
3. It can be run in the fixed-bed mode, similar to HEC-2, by removing all sediment-data records.
4. Sediment transport computations and scour/degradation are performed by grain size fractions. Therefore, Hydraulic sorting and bed armouring can be simulated.
5. Local inflows and outflows of water and sediment from tributaries and/or diversions can be included.
6. It can analyse channel contraction required to either maintain navigation depths or diminish the volume of dredging.
7. The model is also operational on microcomputer.

Model Limitations

1. HEC-6 is a 1D approximation of both flow and sediment routings. It is unable to directly simulate such problems as meandering phenomenon, local scouring, bank erosion, and width adjustment (not able to accurately predict channel response).
2. The flow is assumed to be quasi-steady, i.e., composed of a series of steady state flows. It is not suitable for rapidly changing flow conditions.
3. Sediment transport capacity is assumed to be equilibrium with the flow hydraulics for each computational cycle.
4. Inadequacies in current engineering knowledge reflected in the model include the computations of sediment transport rates, armouring, and hydraulic sorting.
5. Density currents, secondary currents and bed forms are not accounted for.

(II) TABS2

Model Capabilities

1. TABS2 is a 2D uncoupled model that can be used either as a stand-alone solution technique or as a step in the hybrid modelling approach.
2. It can calculate water surface elevations, current patterns, dispersive transport, sediment erosion, transport and deposition, and bed-surface elevations.
3. The model can be used to determine the impact of project designs on flows, sedimentation, and salinity.
4. It is useful to a limited extent for stream width studies and is applicable to clay and/or sand bed sediments. It (STUDH Program) can be used to compute sediment transport, deposition, and erosion in 2D open channel flows.
5. It (component program, RMA-2V) computes water surface elevation, current patterns, flow distribution around islands, flow at bridges and having one or more relief openings, flow in contracting and expanding reaches, flow into and out of off-channel storage for...
hydro or pumping plants, flow at river junctions, and general flow patterns in rivers, reservoirs, and estuaries. It is designed for far-field problems in which vertical accelerations are negligible and can analyse both steady and unsteady state flow problems.

Model Limitations

1. Long, continuous, simulation application is not feasible because of the computation cost.
2. Variations in velocity or constituent concentration with depth are not predicted.
3. RMA-2V is not designed for near field problems where flow structure interactions (vibrations or vertical accelerations) are important. Areas of vertically stratified flow are not predicted.
4. Armouring is not addressed; for sediment calculation, only a single, grain-size sediment can be analysed. In using the STUDH program, fall velocity must be prescribed along with the water surface elevations, x- and y-velocity, diffusion coefficients, bed density, and critical shear stress for deposition.
5. The model is not designed to study problems of local scouring, armouring effect, bank erosion, width adjustment, or meandering.

(III) IALLUVIAL

Model Capabilities

1. IALLUVIAL is designed to simulate 1D, long-term changes in alluvial streams
2. It can simulate problems of Vertical non-homogeneity of bed materials and can input tributary and bank erosion rates including the respective grain-size distributions of the incoming sediments.

Model Limitations

1. IALLUVIAL is incapable of simulating problems of local scour, width adjustment, and flow at a junction with a tributary.
2. The model is still undergoing development and modification

(IV) STARS

Model Capabilities

1. STARS can simulate stream-flow with a fixed or moveable bed.
2. It can predict different rates of scour and fill for each stream-tube.

Model Limitations

1. STARS is unable to make channel width adjustments.
2. It is not applicable to:
   - supercritical flow
   - reverse flow
   - fast-moving flood waves over long reaches
   - bank failure evaluation and
   - wash load problems.
(V) SEDICOUP

Model Capabilities

1. SEDICOUP can simulate the movement of unsteady water and sediment with separation of suspended load and bed load.
2. It can treat sediment mixtures, sorting, and armouring with a multi-class and mixed-layer approach.
3. Through the use of an advection-diffusion approach, wash load, suspended load, and bed load all may be represented.
4. An algorithm, incorporated in the model allows the distinction between the active bed, the flood plain, and the banks. The bank degradation and aggradation with corresponding changes of bank slope and width adjustment as well as effect of groynes can be simulated.

Model Limitations

1. The program's limitations are plan variation of the bed, reversal flows and effects of downstream tide and multi-connected stream networks.
2. It is not able to do simulation of local scouring, cohesive materials, bank erosion, width adjustment, meandering, and costs of running the program.

(VI) FLUVIAL-12

Model Capabilities

Dynamic river channel changes simulated by the model are:

1. Channel bed scour and fill (or aggradation and degradation),
2. Width variation, and
3. Changes in bed topography induced by the curvature effect.

While this model is for erodible channels, rigid physical constraints may also be specified. Applications of this model in this case include:

1. Evaluations of general scour at bridge crossings
2. Sediment delivery
3. Channel responses to sand and gravel mining, and
4. Channel design.

Model Limitations

The model is limited to quasi-two dimensional flow conditions. It should not be applied to highly two dimensional flows, such as that in a highly braided channel.

(VII) HEC2SR

Model Capabilities

With HEC2SR, users can input sediment inflows directly or internally to generate sediment loading data by considering sediment transport capacities of the upstream main stream and
tributaries. It is made of modular structures so users can easily modify each individual functional component without great difficulty.

Model Limitations

1. HEC2SR is 1D. Because of the high cost of backwater computation, it is not suitable for investigating long-term riverbed changes.
2. Because of the assumption of regional degradation and aggradation, it is not applicable to localized scour or deposition.
3. The model does not account for both lateral channel migration and secondary currents. It does not account for bank erosion and has intrinsic limitation on its ability to accurately predict channel response.

(VIII) TWODSR

Model Capabilities

1. TWODSR is integrated with a 1D water and sediment routing model to simulate a long reach of river system and has the potential to be applied to evaluate a variety of river development and response problems in detail. The mathematical model is generally suited to study both simple and complex problems of water and sediment flow in natural channels. It provides a practical and useful method to model a long-reach of river system and to study segments of special interest in detail.
2. The 2D modules provide a depth-averaged 2D simulation of high spatial resolution, making possible the quantitative description of the movement of sand bars, the filling of dredge cuts, the channel response to man-made structures, etc.
3. The model is capable of simulating:
   - Width adjustment,
   - Meandering,
   - Flows at a junction with a tributary,
   - The effects of dikes on stream-flow, and
   - The effects of upstream sediment reduction or dredging on downstream reaches.

Model Limitation

TWODSR is presently unable to simulate problems of fine sediments and armouring effect.

(IX) RESSED

Model Capabilities

RESSED is a useful tool to simulate sediment transport conditions in the river and reservoir systems where fine sediments are important in sedimentation processes.

Model Limitations

1. It is a 1D model, applicable only to narrow reservoir, or a run-of-river reservoir
2. It is unable to simulate problems of local scouring and bank erosion.
3. The sediment transport computation under non-equilibrium condition can be improved.
(X) ONED3X

Model Capabilities

1. ONED3X is flexible and by user's control or adjustment, can cover a range of computational flows and varying degrees of computational accuracy. All dependent variables simulated are for unsteady flow.

2. The model is a comprehensive; it contains several variants, including non-homogeneous terms, scaled terms, and special terms.

3. It can function in four computational modes and can automatically change to the appropriate computational modes.

4. It is a coupled model in which hydrodynamic and sediment transport equations are solved simultaneously.

5. The flow resistance coefficient and the sediment concentration coefficient can be calibrated independently in the scale zone for the hydrodynamic and the sediment transport simulations.

6. The hydrodynamic aspects of the model are as advanced as comparable fixed-bed models.

Model Limitations

1. ONED3X is a 1D.

2. In the model, the sediment concentration function, which must be differentiable (to at least one order), is used to represent sediment transport rate. Bed loads are not accounted for.

3. The bed sediment layer is assumed to be of one grade or several grades of uniform granular material.

4. The maximum duration of flow simulation with continuous bed degradation is limited by the thickness of the bed sediment layer.

5. The model is incapable of simulating problems of:
   - bank erosion
   - local scour
   - channel adjustment
   - armouring effect
   - wash load

(XI) CHARIMA

Model Capabilities

1. CHARIMA is 1D and applies to non-uniform sediment mixtures moving essentially as bed load

2. It can be applied to situations with strong inertial unsteady effects, for rapidly rising flood waves and reversing tidal flow in estuaries and for channel networks of any level of connectivity, such as deltas and distribution channels.

3. It can treat special features, such as weirs and rock outcrops.

4. Compound sections can be treated approximately

Model Limitations

1. CHARIMA does not consider lateral bank movement or geomorphic width changes and therefore is not useful for predicting plan-view changes.
2. It is incapable of simulating problems of local scour, fine sediment, plane variation of the bed, bank erosion, width adjustment, and meandering.

3. Because of the uncoupled solution, small time steps must be used in case of large variations of water or sediment discharge.

2.4 Review on Reservoir Sedimentation

In this section, case studies on sediment management in some reservoirs are reviewed. This is not in any way exhaustive but serves to explain the fact that deposition of sediments in reservoirs is a problem to reckon with. Shahin (1993) while reviewing reservoir sedimentation in some African river basins, concluded that the design sediment inflow to some storage reservoirs had been underestimated. As a result, those reservoirs have been filled with sediments faster than was expected. He stressed the need for more frequent and accurate bathymetric surveys of the existing reservoirs to improve on the accuracy of sediment deposition volume estimates. Reservoir operation rules securing the maximum benefit of hydro power production and/or land irrigation might be in conflict with the strategy aiming at reducing reservoir sedimentation. In this situation, a cost-benefit analysis might help settle the conflict of interests.

Banasik et al. (1993), investigating sediment deposition in a designed Carpathian reservoir showed that there is significant influence of reservoir water management on distribution of deposited sediment in the reservoir and backwater profile.

Several attempts have been made to model sedimentation processes in reservoirs. Okabe et al. (1993) developed a simulation model for sedimentation process in gorge-type reservoirs. The results showed that grain sorting process seemed to be simulated more exactly by a kind of multi-layer model for the change in the grain size composition in bed surface layers. On the other hand, the variation in the streamwise bed profile was simulated more accurately if the influence of the stream constriction on the re-erosion of deposited sediments was taken into account. Ito (1993) investigated reservoir sedimentation for different size particles through a laboratory flume experiment. It was shown that the sand particle size affected the formation of deposited sediment profiles and the rate with which the delta’s shoulder advanced forward in the reservoir. Especially, the latter varied much for different size particles. However, the sediment distribution in flow did not agree with the equilibrium distribution near the bed surface. It was thought that in this near bed region, less flow disturbance existed than near the water surface.

Bednarczyk and Madeyski (1998) assessed suspended load trapped in a small reservoir in relation to the erosion in a loess basin. Although proper determination of the intensity of erosive processes, such as those that occur in loess basins, presented great difficulties, they managed to determine a relationship between the suspended load trapping ability and the erosive processes. The measured value was 44.9 t/km²/year, whereas by means of MUSLE method it was 66.9 t/km²/year. Reasons for this discrepancy can be explained by the fact that precise determination of all the factors of MUSLE is very difficult. Especially difficult are those factors which vary periodically. Besides, not all the sediment load eroded from the basin enters the rivers. Considering the fact that turbidity measurements are also charged with a certain error, the MUSLE method, with a proper choice of its parameters, can also be used in loess basins. This is of significant importance in predicting the silting up of reservoirs.

Ongwenyi et al. (1993), while giving an overview of the soil erosion and sedimentation problems in Kenya, concluded that there is need for an evaluation of the economic impact of
sedimentation in all reservoirs built or planned for in any drainage basin taking into account environmental implications. He suggested that regular and uniform sediment monitoring should be started immediately. There is also a need for training and support of a large number of Kenyans in all aspects of water and soil management, and conservation. It was noted also that the future sedimentation problems cannot be adequately assessed merely by measuring sediment in the river channel at the site of the planned or existing reservoir. Realistic evaluation must be based upon a thorough understanding of the hydrologic and geomorphic processes as they relate to land use on the entire catchment.

From the literature studied so far, it is clear that an integrated approach to study sediment yield, transport, and reservoir sedimentation processes is needed. Yang (1996) stated that: “It is now possible to use the unit stream power theory to determine the total rate of sediment yield and transport from a watershed regardless of whether the sediment particles are transported by sheet, rill, or river flows. By doing so, the actual amount of sediment entering a reservoir can be determined using a consistent rational method.’’ GSTARS 2.1 has been enhanced to become a reservoir sedimentation management tool to determine reservoir sediment distribution and with appropriate operation rules can prolong the useful life of a reservoir (Yang, et al., 1998).

In view of Yang’s statements, the present work is an attempt to provide a systematic and rational integrated approach to determine the rate of surface erosion, sediment transport in river channels, and reservoir sedimentation pattern based on the above considerations.
METHODOLOGY

PART A: EROSION MODELLING

3.1 Introduction

This research entails sediment management in reservoirs. Sediment management involves measures taken to ensure that the reservoir is not fully silted up before realising its full benefits. The measures taken can be divided into two broad classes: those that apply to the catchment and those that are directly applied to the reservoir. The catchment approach involves using soil conservation techniques (both cultural and physical conservation measures) to curb soil erosion, hence cut off or minimise supply of sediments into the reservoir. Management of sediments already trapped in the reservoir involves short-term measures to alleviate the problem of sedimentation. Some of the measures employ permanent hydraulic structures, in which case the removal of sediments is through reservoir operations. Others involve regulation of inflow sediment into the reservoir. Flushing out of sediments by releasing flow (through opening of bottom outlets) and dredging are other remedial measures for recovering the active storage capacity of the reservoir. However, to be able to give meaningful suggestions on techniques for managing sediment in reservoirs, one needs to model sedimentation process beforehand.

Modelling the reservoir sedimentation process can be accomplished through use of analytical methods, physical model constructions, and numerical models. An analytical solution for Masinga reservoir with such a complex shoreline is not plausible. Numerical models have several important advantages over physical models: lower cost, ease of rerunning to simulate a variety of different conditions, ability to simulate some types of problems numerically that are unsuitable for physical modelling because of the scaling laws involved (e.g. sediment cohesion), portability, and reproducibility. Because of the above considerations, a numerical model was employed to model the sedimentation process in Masinga reservoir.

Through sedimentation process modelling, reservoir bed profile changes with time are simulated. The models compute the amount of scour or deposition for each reach and the cross-section geometry adjusted accordingly. Sedimentation models however, require suspended load as one of the input data. For Masinga reservoir, no reliable suspended sediment data is available. Only scanty information on suspended solids is available. For instance, the measurements by Pacini (1994) of suspended solids for Thika and Tana rivers is the longest continuous data series available and covered only a short period from March 1991 to June 1993. Isolated short-period measurements are also available, but not sufficient enough to help in devising sediment management strategies for the reservoir.

Hydro-surveys using an echo-sounder and Geographical Positioning System (GPS) are useful in determining the rate of sedimentation in the reservoir. However, for Masinga reservoir, only two surveys data are available, namely; the original rangeline of the reservoir basin measured before impoundment in 1981 and the 1988 hydro-survey, which was the last survey on this lake. Hence, from 1988 to date, we cannot say for certain, how much sediments have been deposited in the reservoir. Some little information on climate, soils, slope and land use of Masinga catchment is available. Because of this data scarcity, a model is needed which utilises the little available information about the catchment and the reservoir in an integrated and efficient way to model sediment input into the reservoir. One such model is the WEPP which makes use of climate, soils, slope and plant/management information to predict sediment yield from the catchment.

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3.2 Suspended Sediment Generator

Because of the lack of reliable data on suspended load for rivers draining into Masinga, it was deemed necessary to model sediment input into the reservoir. That notwithstanding, suspended load is the basic input parameter for GSTARS 2.1 (Yang and Simoes, 2000) model, a selected sedimentation model for simulating sedimentation patterns in Masinga reservoir. An erosion prediction model was used for modelling of sediment input into the reservoir.

3.2.1 Review of Erosion Prediction Models

(a) USLE

The Universal Soil Loss Equation (USLE) was developed and first published in 1958 by Walter H. Wischmeier and D.D. Smith. Over the next 20 years they refined and improved the USLE and published the results in Agriculture Handbook 537 in 1978, which is still a standard reference. The USLE is widely used for land management planning world wide and is regarded as a primary tool of conservationists for planning purposes (according to the International Soil and Water Conservation Society). The equation provides techniques for numerically evaluating effects of climate, soil properties, topography, crop-productivity level, time and method of seeding, crop sequence, residue management, special conservation practices, and other pertinent variables that effect soil erosion. It is a required element in farm and ranch plans used to qualify for soil conservation programs; it is an invaluable tool for natural resource inventories carried out throughout the globe; it has been the basis of economic analyses related to agriculture; and it has been an important element in analyses dealing with assessment and control of surface water quality. The USLE has provided an invaluable tool for natural resource inventorying in most parts of the world. The technology has been used to analyse the erosion effects on crop productivity and has been the basis of economic analyses conducted by soil conservationists.

The USLE, has been the most widely accepted and utilised soil loss equation for over 30 years. Designed as a method to predict average annual soil loss caused by sheet and rill erosion, the USLE is often criticised for its lack of applications. While it can estimate long - term annual soil loss and guide conservationists on proper cropping, management, and conservation practices, it can not be applied to a specific year or a specific storm. The USLE is a mature technology and enhancements to it are limited by the simple equation structure.

The USLE for estimating average annual soil erosion is:

\[ A = RKLSCP \]  \hspace{1cm} (3.1)

where

- \( A \) = average annual soil loss in t/a (tons per acre)
- \( R \) = rainfall erosivity index
- \( K \) = soil erodibility factor
- \( LS \) = topographic factor - L is for slope length & S is for slope
- \( C \) = cropping factor
- \( P \) = conservation practice factor

Evaluating the factors in USLE:
R - the rainfall erosivity index

Most appropriately called the erosivity index, it is a statistic calculated from the annual summation of rainfall energy in every storm (correlated with raindrop size) times its maximum 30-minute intensity. As expected, it varies geographically.

K - the soil erodibility factor

This factor quantifies the cohesive, or bonding character of a soil type and its resistance to dislodging and transport due to raindrop impact and overland flow.

LS - the topographic factor

Steeper slopes produce higher overland flow velocities. Longer slopes accumulate runoff from larger areas and also result in higher flow velocities. Thus, both result in increased erosion potential, but in a non-linear manner. For convenience L and S are frequently lumped into a single term.

C - the crop management factor

This factor is the ratio of soil loss from land cropped under specified conditions to corresponding loss under tilled, continuous fallow conditions. The most computationally complicated of USLE factors, it incorporates effects of: tillage management (dates and types), crops, seasonal erosivity index distribution, cropping history (rotation), and crop yield level (organic matter production potential).

P - the conservation practice factor

Practices included in this term are contouring, strip cropping (alternate crops on a given slope established on the contour), and terracing.

(b) RUSLE

The Universal Soil Loss Equation (USLE) was revised and released as the Revised Universal Soil Loss Equation (RUSLE) by the USDA- Agricultural Research Service (ARS), Natural Resource Conservation Service (NRCS), and co-operators through the Soil and Water Conservation Society. RUSLE is a significant advancement and improvement over the widely used USLE.

Revisions to the universal soil loss equation were implemented in the mid 1990s to more accurately predict soil erosion caused by water. It includes the same factors as the earlier formula; climate, soils, topographic conditions, and the degree to which the use and management of the soil reduces erosion. But it takes advantage of new knowledge about these relationships and the capabilities of computer technology. The comparison between predicted erosion and true values is important in making and carrying out conservation plans and achieving conservation compliance. The forms of RUSLE are:

(i) RUSLE 1.05

RUSLE is an erosion prediction model that predicts long time average annual soil loss resulting from raindrop splash and runoff from specific field slopes in specified cropping and
management systems and from range land. RUSLE is a replacement for the Universal Soil Loss Equation (USLE) and retains the six factors in that equation. These factors represent the rainfall and runoff factor (R), soil erodibility factor (K), slope length and steepness factors (LS), cover and management factor (C), and the support practices factor (P). Developed by the USDA-Agricultural Research Service, and first released in 1993, this technology has been implemented in field offices of the USDA-Natural Resources Conservation Service and is being used nationally and internationally for prediction of rill and interrill erosion on cropland, range land and other land uses.

(ii) RUSLE 1.06

RUSLE is an erosion prediction model that predicts long time average annual soil loss resulting from raindrop splash and runoff from specific field slopes in specified cropping and management systems and from range land. This version of RUSLE is designed for use of RUSLE on mined lands, construction sites, and reclaimed lands. New features in version 1.06 include computation of deposition on concave slopes, in terrace channels, and in sediment basins; and improved computation of the effectiveness of ground cover on steep slopes at construction sites. RUSLE 1.06 was first released in 1998.

(iii) RUSLE2

RUSLE2 predicts long-term, average-annual erosion by water for a broad range of farming, conservation, mining, construction, and forestry uses. Its object-oriented, Windows interface allows dramatic scientific and graphical advances. RUSLE2 is derived from previous DOS RUSLE software based on the widely-used Revised Universal Soil Loss Equation (RUSLE) and can reuse much of the extensive data available for that model. It is also a key part of the emerging ARS Modular Soil Erosion System (MOSES). It was first released in 2000.

(iv) RUSLE FOR FOCS

This is Version for RUSLE for Field Officers Computing System (FOCS).

(c) KINEROS

The Kinematic Runoff and Erosion Model, KINEROS, is an event oriented, physically based model describing the processes of interception, infiltration, surface runoff and erosion from small agricultural and urban watersheds. The watershed is represented by a cascade of planes and channels; the partial differential equations describing overland flow, channel flow, erosion and sediment transport are solved by finite difference techniques. The spatial variation of rainfall, infiltration, runoff, and erosion parameters can be accommodated. KINEROS may be used to determine the effects of various artificial features such as urban developments, small detention reservoirs, or lined channels on flood hydrographs and sediment yield.

A subroutine to kinematically route unpressurized, free-surface flow through circular conduits (storm drains, culverts, etc.) has been added to the general source code. This type of element was available in the original version of KINEROS. For backwater conditions at the inlet, use a pond element to control inflow into the pipe. Sediment from upstream elements is also routed through the pipe element, with both deposition and erosion of deposited material being possible during a flow. The limitations of KINEROS lies in its inability to simulate evapotranspiration, interflow and groundwater flow.

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(d) Basin Scale Models

Several hydrologic models have been developed for use at the basin scale.

(i) AGNPS

The Agricultural Nonpoint Source Pollution Model (AGNPS) (Young et al., 1987) was developed by the United States Department of Agriculture - Agricultural Research Service (USDA-ARS). The AGNPS model is a distributed parameter, single event model capable of simulation on watershed up to 20,000 ha. AGNPS uses the standard Soil Conservation Service (SCS) curve number technique for runoff estimation (USDA-SCS, 1986) and the Universal Soil Loss Equation (Wischmeier and Smith, 1978) for sediment yield. The AGNPS model was primarily developed to simulate nonpoint-source nitrogen and phosphorus movement within a watershed, and is also capable of point-source simulations from wastewater treatment facilities.

(ii) EPIC

A similar model to AGNPS, the Erosion Productivity Impact Calculator (EPIC) model, was also developed by the United States Department of Agriculture (USDA-ARS). EPIC is a comprehensive model developed to assess the relationship between soil erosion and productivity (Williams and Sharpley, 1990). The EPIC model produces accurate results, but was originally designed to operate at the smaller field scale, due to the fact that it assumes soil type and management are homogeneous within the field.

(iii) ANSWERS

The Areal Nonpoint Source Watershed Environmental Response Simulation (ANSWERS) model was jointly developed by the Environmental Protection Agency (EPA) and Purdue University (Beasley and Huggins, 1982). ANSWERS is a single-event, distributed-parameter model to simulate hydrology, sediment transport, and routing through a basin. Size of the basin is generally limited to 10,000 ha, with the basin further divided into smaller independent elements. The use of these elements allows for considerable spatial detail with respect to topography, soils, and land use. This is, however, a time intensive process when modelling large basins (Engel and Arnold, 1991).

(iv) HSPF

Johansen and others (1984) developed the Hydrological Simulation Program - FORTRAN (HSPF) model to simulate both basin hydrology and water quality. A continuous-time model, HSPF allows simulation of contaminant runoff with instream water quality and sediment interactions. The instream component includes not only nutrient processes such as nitrogen and phosphorus movement, but also benthic algae, phytoplankton, and zooplankton. The HSPF model also simulates complex chemical processes including hydrolysis, biodegradation, and oxidation. The drawback to HSPF is the data intensive nature of the input parameters required. HSPF is capable of simulations on basins as large as 68,000 square miles, and the watershed can be subdivided into smaller subbasins.
(v) SWRRB

The Simulator for Water Resources in Rural Basins (SWRRB) model was developed to predict the effects of land management practices on water and sediment yields for large, ungauged, rural basins (Arnold et al., 1990; Williams et al., 1985). SWRRB was developed by modifying the CREAMS daily rainfall model (Knisel, 1980) for large, complex basins. Major additions to the CREAMS models include allowing simultaneous computations for several sub-watersheds within a large basin and adding components to simulate weather, return flow, pond and reservoir storage, crop growth, transmission losses, groundwater, and sediment routing. SWRRB operates on a daily time step and is capable of simulations up to 100 years or more. Being a continuous-time model, SWRRB can thus determine impacts of various land management decisions such as crop rotations, chemical applications, and planting and harvest dates. SWRRB allows basins to be divided according to land use, soils, and topography. Since SWRRB places a limit on the number of sub-basins within a watershed, some lumping of input parameters is necessary.

(vi) SWAT

The Soil and Water Assessment Tool (SWAT) is the hydrologic model used in the SWAT/GRASS linkage (Arnold et al., 1995). It is a continuous-time, basin-scale hydrologic model capable of complex long-term simulations including hydrology, pesticide and nutrient cycling, and erosion and sediment transport. SWAT owes its strength to the linkage with the Geographic Resources Analysis and Support System (GRASS). A preprocessing utility was programmed for the GRASS GIS to link it to the SWAT model for use in generating the input information used by SWAT (Arnold et al., 1993).

3.2.2 Selection of an Erosion Model

The Water Erosion Prediction Project (WEPP) was selected for modelling of sediment input into Masinga reservoir. The current model version for Windows (v2001.300) was released in April 2001 by Flanagan (2001). This model was selected for the present study because of the following reasons:

1. Its capabilities for estimating spatial and temporal distributions of soil loss; net soil loss for an entire hillslope or for each point on a slope profile can be estimated on daily, monthly, or average annual basis. Indeed, WEPP was preferred to KINEROS2 (Kinematic Runoff and Erosion Model), for example, because of the former’s capability to simulate sediment yield on a daily basis as required in this study. KINEROS2 is event-oriented and physically-based model incapable of simulating daily sediment yields from the catchment.
2. Since the model is process-based, it can be applied on a broad range of conditions that may not be practical or economical when field tests are used. Being physically-based, KINEROS2 cannot be applied on complex catchment conditions.
3. It can be used to simulate rill and interrill erosion separately. KINEROS2, for example, cannot differentiate between rill and interrill erosion.

3.2.3 Water Erosion Prediction Project (WEPP) Model Theory

The WEPP model may be used in both hillslope and watershed applications. The model is a distributed parameter, continuous simulation, erosion prediction model, implemented as a set of computer programs for personal computers. The distributed input parameters include rainfall amounts and intensity, soil textural qualities, plant growth parameters, residue

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decomposition parameters, effects of tillage implements on soil properties and residue amounts, slope shape, steepness, and orientation, and soil erodibility parameters. Continuous simulation means that the computer program simulates a number of years, with each day having a different set of input climatic data. On each simulation day a rain storm may occur, which then may or may not cause a runoff event. If runoff is predicted to occur, the soil loss, sediment deposition, sediment delivery off-site, and the sediment enrichment for the event will be calculated and added to series of sum totals. At the end of the simulation period, average values for detachment, deposition, sediment delivery, and enrichment are determined by dividing by the time interval of choice. The entire set of parameters important when predicting erosion are updated on a daily basis, including soil roughness, surface residue cover, canopy height, canopy cover, soil moisture, etc. This continuous updating relieves one of the difficult job of determining temporal distributions of important parameters, such as cover values.

In watershed applications, the WEPP model applies to field areas that include ephemeral gullies which may be farmed over and are known as concentrated flow gullies, or constructed waterways such as terrace channels and grassed waterways. For rangeland applications, it applies to areas that include gullies that are up to the size of ephemeral gullies in cropland, i.e. about 1 to 2 meters (3 to 6 ft) wide and 1 meter (3 ft.) deep. The hillslope routines of WEPP are used for the overland flow portion of the area and the watershed routines of WEPP are used on channels and impoundments.

The erosion predictions from the WEPP model are meant to be applicable to "field-sized" areas or conservation treatment units. When applied to a single hillslope, the model simulates a representative profile, which may or may not approximate the entire field. For large broad zones in which there is a definite slope shape dominating an entire field, one profile representation may be sufficient to adequately model the site. However, for very dissected landscapes, in which several different, distinct slope shapes exist, several hillslopes will need to be simulated (either as separate runs within the Hillslope Interface, or as a single watershed simulation in the Watershed Interface). The maximum "field" size is about 640 acres although an area as large as 2000 acres is needed for some range land applications. The model should not be applied to areas having permanent channels such as classical gullies and perennial streams, since the processes occurring in these types of channels are not simulated in WEPP. Use of the watershed application of WEPP is necessary to simulate flow, erosion, and deposition in ephemeral gullies, grassed waterways, terrace channels, other channels, and impoundments.

3.2.3.1 Hillslope Profile Model Components

The WEPP model as applied to hillslopes can be subdivided into nine conceptual components: climate generation, winter processes, irrigation, hydrology, soils, plant growth, residue decomposition, hydraulics of overland flow, and erosion. This section will give a brief description of each component.

3.2.3.1.1 Hydrology Component

The primary purpose of the WEPP surface hydrology component is to provide the erosion component with the duration of rainfall excess, the rainfall intensity during the period of rainfall excess, the runoff volume, and the peak discharge rate. A secondary purpose is to provide the amount of water which infiltrates into the soil for the water balance and crop
growth/residue decomposition calculations which are in turn used to update the infiltration, runoff routing, and erosion parameters.

The hydrology component of WEPP computes infiltration, runoff, soil evaporation, plant transpiration, soil water percolation, plant and residue interception of rainfall, depressional storage, and soil profile drainage by subsurface tiles (Fig. 3.1). Infiltration is calculated using a modified Green and Ampt infiltration equation. Runoff is computed using the kinematic wave equations or an approximation to the kinematic wave solutions obtained for a range of rainfall intensity distributions, hydraulic roughness, and infiltration parameter values. The water balance routines are a modification of the SWRRB water balance (Williams et al., 1985). Infiltration is linked with the evapotranspiration and percolation components to maintain a continuous water balance. Infiltrated water is added to the upper layer’s soil water content and routed through the lower soil layers. Percolation below the root zone is considered lost water from the WEPP water balance. The model maintains a continuous water balance on a daily basis using the equation:

\[
\Theta = \Theta_m + (P - I) \pm S - Q - ET - D - Q_d 
\]  

(3.2)

where \( \Theta \) is the soil water content in the root zone in any given day (m), \( \Theta_m \) is the initial soil water in the root zone (m), \( P \) is the cumulative precipitation (m), \( I \) is precipitation interception by vegetation (m), \( S \) is the snow water content (m), \( Q \) is the cumulative amount of surface runoff (m), \( ET \) is the cumulative amount of evapotranspiration (m), \( D \) is the cumulative

**Fig. 3.1:** Processes in WEPP hillslope hydrology include precipitation (rain or snow), Infiltration, runoff, plant transpiration, soil evaporation and percolation. After Flanagan (2001).
amount of percolation loss below the root zone (m), and \( Q_d \) is subsurface lateral flow or flow to drain tiles (m).

### 3.2.3.1.2 Erosion Component

The four hydrologic variables required to drive the erosion model are peak runoff, effective runoff duration, effective rainfall intensity, and effective rainfall duration. These variables are calculated by the hydrology component.

The erosion component of the WEPP model uses a steady-state sediment continuity equation to estimate the change in sediment load in the flow with distance downslope. Soil detachment in interrill areas is modelled as a function of rainfall intensity and runoff rate, while delivery of interrill sediment to rills is a function of slope and surface roughness. Detachment of soil in the rills is predicted to occur if the hydraulic shear stress of the flow exceeds a critical value, and the sediment already in the flow is less than the flow's transport capacity. Simulation of deposition in rills occurs when the sediment load in the flow is greater than the capacity of the flow to transport it. Adjustments to soil detachment are made to incorporate the effects of canopy cover, ground cover, and buried residue. The WEPP model also computes the effects of selective deposition of different sediment classes and estimates a sediment size distribution leaving a hillslope. An enrichment ratio of the sediment specific surface area is also estimated.

### 3.2.3.1.2.1 Governing Equations

The governing equations used in the WEPP erosion model are sediment continuity, detachment, deposition, shear stress in rills, and transport capacity.

(i) Sediment continuity equation

WEPP hillslope profile erosion model uses a steady-state sediment continuity equation to describe the movement of sediment in a rill:

\[
\frac{dG}{dx} = D_f + D_i \tag{3.3}
\]

where \( x \) represents distance downslope (m), \( G \) is sediment load (\( \text{kg.s}^{-1}.\text{m}^{-1} \)), \( D_i \) is interrill sediment delivery to the rill (\( \text{kg.s}^{-1}.\text{m}^{-2} \)) and \( D_f \) is rill erosion rate (\( \text{kg.s}^{-1}.\text{m}^{-2} \))

(ii) Detachment

Detachment occurs when hydraulic shear stress exceeds the critical shear stress of the soil and when sediment load is less than sediment transport capacity.

\[
D_f = D_c \left( 1 - \frac{G}{T_c} \right) \tag{3.4}
\]

Where \( D_c \) is detachment capacity by rill flow (\( \text{kg.s}^{-1}.\text{m}^{-2} \)), \( T_c \) is sediment transport capacity in rill (\( \text{kg.s}^{-1}.\text{m}^{-1} \)) and
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\[ D_e = K_r (\tau_f - \tau_c) \]  \hspace{1cm} (3.5)

Where \( k_r \) is a rill erodibility parameter (s.m\(^{-1}\)), \( \tau_f \) is flow shear stress acting on soil particles (pa), \( \tau_c \) is the rill detachment threshold parameter (critical shear stress of the soil) (Pa).

(iii) Deposition

Deposition occurs when sediment load is greater than sediment transport capacity

\[ D_f = \frac{\beta V_f}{q} (T_c - G) \]  \hspace{1cm} (3.6)

where \( V_f \) is effective fall velocity for sediment (m.s\(^{-1}\)), \( q \) is discharge per unit width (m\(^2\).s\(^{-1}\)), \( \beta \) is a raindrop-induced turbulence

(iv) Shear stress is expressed as

\[ \tau_{fe} = \frac{\gamma R \sin(\alpha) f_s}{f_t} \]  \hspace{1cm} (3.7)

Where \( \tau_{fe} \) is shear stress acting on soil at the end of the uniform slope (Pa), \( \gamma \) is the specific weight of water (kg.m\(^{-2}\).s\(^{-1}\)), \( \alpha \) is the average slope angle of the uniform slope, \( f_s \) is friction factor for the soil, \( f_t \) is total rill friction factor.

(v) Sediment transport capacity is given by

\[ T_c = k_t \tau_f \]  \hspace{1cm} (3.8)

where \( T_c \) is the sediment transport capacity (kg.s\(^{-1}\).m\(^{-1}\)), \( k_t \) is a transport coefficient (m\(^{0.5}\).s\(^{2}\).kg\(^{-0.5}\)), \( \tau_f \) is the hydraulic shear acting on the soil (Pa).

Interill erosion is conceptualized as a process of sediment delivery to concentrated flow channels, or rills, whereby the interrill sediment is then either carried off the hillslope by the flow in the rill or deposited in the rill. Sediment delivery from the interrill areas is considered to be proportional to the product of rainfall intensity and interrill erodibility parameter, \( K_i \).

3.2.3.1.3 Summary of other WEPP Components

Simulated climate for WEPP model simulations is normally generated using the CLIGEN model, which is a computer program run separately from the WEPP erosion model. CLIGEN creates climate input data files for WEPP which contain daily values for rainfall amount, duration, maximum intensity, time to peak intensity, maximum and minimum temperatures, solar radiation, wind speed, wind direction, and dew point temperature. The rainfall for a day is disaggregated into a simple single-peak storm pattern (time-rainfall intensity format) for use by the infiltration and runoff components of the model. Input climate files to WEPP can also be constructed so as to accept breakpoint rainfall data.

Winter processes modelled in WEPP include soil frost and thaw development, snowfall, and snow melting. Simple heat flow theory is used with the daily information on temperatures,
The impacts of tillage on various soil properties and model parameters are computed within the soils component of the WEPP model. Tillage activity during a simulation acts to decrease the soil bulk density, increase the soil porosity, change soil roughness and ridge height, destroy rills, increase infiltration parameters, and change erodibility parameters. Consolidation due to time and rainfall after tillage and its impacts on the soil parameters is also simulated.

The plant growth component for croplands calculates above and below ground biomass production for both annual and perennial crops in cropland situations, and for rangeland plant communities in rangeland situations. The plant growth routines in WEPP are based upon an EPIC (Williams et al., 1989) model approach, which predicts potential growth based upon daily heat unit accumulation. Actual plant growth is then decreased if water or temperature stresses exist. Several different types of management options for cropland and rangeland plants can be simulated.

Plant residue decomposition for croplands is based upon a "decomposition day" approach, which is similar to the growing degree day approach used in many plant growth models. Each residue type has an optimal rate for decomposition, and environmental factors of temperature and moisture act to reduce the rate from its optimum value. The WEPP model tracks the type and amounts of residue from the previous 3 crop harvests. The model also allows several types of residue management, including residue removal, shredding, burning, and contact herbicide application.

For rangelands the plant growth component simulates the aggregate above and below ground biomass production for the entire plant community. The plant growth routines in WEPP are based on the ERHYM-II (Wight, 1987) and SPUR models (Wight and Skiles, 1987). Plant growth for rangelands are based on a potential growth curve. Actual plant growth is initiated in the spring when temperature is above a threshold and is a function of water stress. Decomposition of surface litter is based on temperature and precipitation. Root biomass decomposition is based on temperature and soil water content.

The impacts of soil roughness, residue cover, and living plant cover on runoff rates, flow shear stress, and flow sediment transport capacity are computed in the hydraulics of overland flow section of the WEPP model. Rougher surfaces, fields with more residue cover, and closely spaced crops tend to increase the soil surface resistance to flow, which in turn
decreases runoff rates, decreases flow shear stress acting on the soil, and decreases sediment transport capacity of the flow.

### 3.2.3.2 WEPP Watershed Model Version

A watershed is defined as one or more hillslopes draining into one or more channels and/or impoundments. WEPP watershed model is a process-based, continuous simulation model built as an extension of the WEPP hillslope model (Flanagan and Nearing, 1995). The model contains three primary components: hillslope, channel, and impoundment as depicted in Figure 3.2. The smallest possible watershed includes one hillslope and one channel. Runoff characteristics, soil loss and deposition are first calculated on each hillslope with the hillslope component of WEPP for the entire simulation period. Main results are saved in a pass file that is used during the watershed routing. Then the model combines simulation results from each hillslope and performs runoff and sediment routing through the channels and impoundments each time runoff is produced on one of the hillslopes or channels, or if there is an outflow from one of the impoundments. Channel and impoundment parameters such as canopy height and impoundment water level are updated on a daily basis.

![Fig. 3.2: Schematic of a small watershed which the WEPP model could be applied to.](image)


The channel component can be further divided into hydrology and erosion components. The channel hydrology component computes infiltration, evapotranspiration, soil water percolation, canopy rainfall interception, and surface depressional storage in the same manner as the hillslope hydrology component. Channel erosion component is similar to hillslope erosion model component with major differences being: (1) the flow shear stress is calculated using regression equations which approximate the spatially varied flow equations, and (2) only entrainment, transport and deposition by concentrated flow are simulated in the channels.

Channel erosion is based on a steady-state sediment continuity equation. Sediment load in the channel is a function of the incoming upstream load (from hillslopes, channels, and impoundments) and the incoming lateral load (from adjacent hillslopes and impoundments),

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and the ability of the flow to detach channel bed material or soil particles. The flow detachment rate is proportional to the difference between: (1) flow shear stress exerted on the bed material and the critical shear stress, and (2) the transport capacity of flow and the sediment load. Net detachment occurs when flow shear stress exceeds the critical shear stress of the soil or channel bed material and when sediment load is less than transport capacity. Net deposition occurs when sediment load is greater than transport capacity.

### 3.2.4 Data Requirements

The Water Erosion Prediction Project (WEPP) model input parameters include daily rainfall or snowmelt amounts and intensity, and parameters that describe the soil textural characteristics and erodibility, the plant growth and residue decomposition, the management practices, and the topography. Among the parameters for erosion prediction, some are updated on a daily basis: ground cover, plant growth, residue tracking, soil surface sealing, soil surface roughness, soil moisture content, canopy cover characteristics (Flanagan and Nearing, 1995). Others are user-specified: baseline hydraulic conductivity, rill and interill erodibilities, and critical shear stress. These parameters are organized in file formats as described below.

The hillslope component of the WEPP erosion model requires a minimum of four input data files to run: 1) a climate file, 2) a slope file 3) a soil file, and 4) a plant/management file. For the case of irrigation and/or watershed option applications, additional input files are required.

#### 3.2.4.1 Climate Input File

The climate data required by the WEPP model includes daily values for precipitation, temperatures, solar radiation, and wind information. A stand-alone program called CLIGEN is used to generate either continuous simulation climate files or single storm climate files. To run CLIGEN, a stations file and a state database file are required.

#### 3.2.4.2 Slope Input File

The WEPP model requires information about the landscape geometry, which is entered by way of the slope input file. Required information includes slope orientation, slope length, and slope steepness at points down the profile. In the profile application of WEPP, the slope profile may be visualized as a line running up and down the hill, having a representative width which applies to the entire field or a portion of the field.

In the WEPP model, many types of nonuniformities on a hillslope can be simulated through the use of strips or Overland Flow Elements (OFE's). Each OFE on a hillslope is a region of homogeneous soils, cropping, and management. Slope shape is described by using pairs of distance to points from the top of the OFE and the slope at these points. This current version of the WEPP model allows simulation of up to 10 OFE's on an individual hillslope. All of the remaining input files (slope, soil, management, irrigation) must provide information for each OFE on which the hydrologic and erosion processes are to be simulated.

#### 3.2.4.3 Soil Input File

Information on soil properties to a maximum depth of 1.8 meters are input to the WEPP model through the soil input file. Information on up to 8 different soil layers may be input. WEPP internally creates a new set of soil layers based on the original set parameter values. If the entire 1.8 meters is parameterized, the new soil layers represent depths of 0-100 mm, 100-
200 mm, 200-400 mm, 400-600 mm, 600-800 mm, 800-1000 mm, 1000-1200 mm, 1200-400 mm, 1400-1600 mm, 1600-1800 mm. As with the slope file, soil parameters must be input for each and every Overland Flow Element (OFE) on the hillslope profile and for each channel in a watershed, even if the soil on all OFEs is the same.

3.2.4.4 Plant/Management Input File

The plant/management input file contains all of the information needed by the WEPP model related to plant parameters (rangeland plant communities and cropland annual and perennial crops), tillage sequences and tillage implement parameters, plant and residue management, initial conditions, contouring, subsurface drainage, and crop rotations.

3.2.5 Filling in Missing Rainfall Data Values

For some stations with incomplete rainfall data, it was necessary to fill in missing values. A linear multiple regression model was adopted for this purpose.

3.2.5.1 Linear Regression Model

The linear regression equation of $Y$ on $X_1, X_2, X_3, ..., X_n$ can be written as:

$$Y = a_0 + a_1X_1 + a_2X_2 + ... + a_nX_n + e$$

(3.9)

Which represents a hyperplane in $n$ dimensional space and where $Y$ is the dependent variable and $X_1, X_2, ..., X_n$ are the independent variables. $a_0$ is the least squares estimate of the intercept while $a_1, a_2, ..., a_n$ are the least squares estimates of the population data regression coefficients for $X_1, X_2, ..., X_n$ respectively. $e$ is the residual term. In this model, a regression coefficient estimates the effects of the independent variable on the dependent variable across the levels of the other independent variables. For instance, $a_1$ reflects the trends of change in $Y$ with changes in $X_1$ when other independent variables are held constant. Similarly, $a_2$ reflects the trends of change in $Y$ with changes in $X_2$ at constant $X_1, X_3, ..., X_n$.

The sample multiple correlation coefficient, $R$, is an index of overall model fit. It seeks to define how well a linear or other equation describes the relationship between variables. Correlation coefficient measures the goodness of fit of the equation actually assumed to the data.

If all variables are standardised, then the intercept will always equal zero and the regression coefficients, $a_i$, will represent standardised regression coefficients. In this case, the multiple regression equation becomes:

$$Y = a_1X_1 + a_2X_2 + a_3X_3 + ... + a_nX_n + e$$

(3.10)

In this model, several assumptions are necessary with regard to the structure of the population data in order to apply ordinary least squares (OLS) analysis:

1. The expected value of the residual term is zero
2. There is no serial correlation between residuals
3. The residual exhibit constant variance across values of $X_1, X_2, ..., X_n$
4. Covariance between the X’s and the residual term is zero

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(5) No complete multicollinearity is exhibited in the data

3.2.6 GLIGEN Weather Generator Program

The program CLIGEN is used to generate continuous simulation climate files for use by WEPP. The CLIGEN input files include a station file and state files that contain the statistical representation of weather data for the stations in each state. For each station, parameters describing the climate conditions on a monthly basis are used. These parameters included daily rainfall amounts by month, number of wet days following wet days by month, number of wet days following dry days by month, maximum and minimum temperatures, dew-point temperatures, solar radiation, time to peak rainfall intensity, and maximum daily 30 minute liquid precipitation. From the file of statistical data, CLIGEN generates, for as many simulation years as desired, the daily rainfall amount, duration, time to peak, and peak intensity as a ratio to average intensity. It also generates minimum and maximum daily temperatures and dew points, as well as solar radiation, wind direction, and wind velocity.

3.2.6.1 Building a CLIGEN Input Climate File from Existing Climate Data

It is extremely difficult at present to build a whole complete CLIGEN input parameter file for any location, but particularly difficult to do it for a site outside the US.

What is recommended is to use as many locally determined coefficients as possible, and enter them into the CLIGEN data base for a similar US climate. The most important variables are those related to daily precipitation amount and temperature. As long as most other variables are reasonably represented, CLIGEN will perform nicely. The CLIGEN weather stations statistics input file uses a fixed format. The first 17 lines of the input file refer to precipitation, temperature and solar radiation. In this project, only coefficients for lines 1-17 in the input file were computed. The other coefficients in lines 18-81 containing information on wind speed and direction were customized from the WEPP database. The description of the first 17 lines with their corresponding abbreviations is as follows:

Line 1: Station name, state id number, station id number, igcode
Line 2: Latitude, longitude, station years of record, itype
Line 3: Elevation above sea level (ft), TP5 – maximum 30 minute precipitation depth (inches), TP6 – maximum 6 hour precipitation depth (inches)
Line 4: Mean liquid equivalent precipitation depth (inches) for a day precipitation occurs (by month), MEAN P
Line 5: Standard deviation of daily precipitation value (inches) by month, S DEV P
Line 6: Skew coefficient of daily precipitation value (by month), SQEW P
Line 7: Probability of a wet day following a wet day (by month), P(W/W)
Line 8: Probability of a wet day following a dry day (by month), P(W/D)
Line 9: Mean maximum daily air temperature (°F) by month, TMAX AV
Line 10: Mean minimum daily air temperature (°F) by month, TMIN AV
Line 11: Standard deviation for daily maximum temperatures (°F) by month, SD TMAX
Line 12: Standard deviation for daily minimum temperatures (°F) by month, SD TMIN
Line 13: Mean daily solar radiation (Langleys) by month, SOL.RAD
Line 14: Standard deviation for daily solar radiation (Langleys) by month, SD SOL
Line 15: Maximum daily 30 minute liquid precipitation depth (inches) by month, MX .5 P
Line 16: Mean daily dew point temperature (°F) by month, DEW PT
Line 17: Represent cumulative distribution of computed time to peak rainfall intensity values, Time pk
3.2.6.2 Determination of Average Precipitation for a Wet Day (MEAN P)

For a given month, all precipitation over the period of record was divided by the number of wet days over the same period to compute the average precipitation on a wet day (MEAN P). This was accomplished in the skew coefficient sheet of the excel workbook Probability Fitting.

3.2.6.3 Determination of Standard Deviation of Precipitation Amounts on Wet Days (SDEV P)

For all wet days for a given month, a standard deviation (SDEV P) was computed. This is accomplished in the skew coefficient sheet of the excel workbook Probability Fitting.

3.2.6.4 Determination of Skew Coefficient (SQEW P)

Skew coefficient is determined using the probability fitting procedure as outlined below. Using a spread sheet, from a normal distribution table, 2 columns were formed, one with a normally distributed variate (x) and the other with the probability from a cumulative distribution for the normally distributed variate. In this project, values ranging from -4 to +4 were taken from a standard statistical table using an increment of 0.2.

In an adjacent column, for all normally distributed variates the quantity X is computed as

$$X = \left( \left[ \frac{x - \frac{g}{6}}{\frac{g}{6}} \right] + 1 \right)^3 - 1 \left( \frac{2S}{g} \right) + U$$

(3.11)

Where x is the normally distributed variate

- g is the skew coefficient (SQEW P)
- S is the standard deviation (SDEV P)
- U is the mean precipitation of a wet day (MEAN P)
- X is the daily precipitation amount that is generated.

The column with the daily precipitation amount is sorted in ascending order. In an adjacent column, the ratio of the number of values equal to and smaller for each value, divided by the total number of values in the column is computed giving a probability for each precipitation amount for each normally distributed variate. A graph with the X values on the Y-axis and probability on the X-axis is then plotted. This graph served as a comparison with measured data so that the skew coefficient could be adjusted until measured and generated are very similar.

For measured data, for every day with precipitation for a given month, over the entire period of record, the precipitation is arrayed from the smallest to the largest and the number of days with precipitation determined. Then beginning with the smallest precipitation, the cumulative distribution probability for each day with precipitation is determined. The amount of precipitation is plotted on the Y-axis, with the cumulative percentage on the X-axis, on the same graph as the generated values above. A typical graph is shown in Figure 3.3.
3.2.6.4 Determination of P(W/W)- Probability of a Wet Day Following a Wet Day and P(W/D), the Probability of a Wet Day Following a Dry Day.

The probability fitting procedures are also used to determine for precipitation: P(W/W) and P(W/D). The steps below are followed:

1. For each month in each year the number of wet days (WD) are determined
2. For each month in each year the number of wet days following wet days (W/W) are determined
3. For each month in each year the number of wet days following dry days (W/D) are determined
4. For each month in each year the ratio of (W/W)/WD is computed. Average for each month for all years is P(W/W) for that month.
5. For each month in each year the ratio of (W/D)/(ND-WD) is computed, where ND is number of days in month. Average for each month for all years is P(W/D)

![Graph for determination of skew coefficient, SQEW P](image)

Standard deviations for maximum and minimum temperatures and solar radiation are computed from the available data. Data on maximum daily 30 minute liquid precipitation depth, mean daily dew point temperature and time to peak rainfall intensity values are required to run with CLIGEN.

Wind data used in this project was directly imported from stations in the US database.

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3.2.7 Separation of Hydrograph Components

A stream flow hydrograph of a given storm is a hydrograph of total runoff and can be divided into three major components, namely: surface runoff, inter-flow and base flow. Surface runoff or overland flow is that part of precipitation that has no time to infiltrate into the soil. Instead, it travels over the ground surface to a drainage channel. Inter-flow, also called subsurface storm flow, is that surface water that infiltrates into the soil and moves laterally beneath the surface to a channel. Base flow, or groundwater flow, is the flow component contributed to the channel by groundwater. Groundwater occurs from surface-water infiltration to the water table and then moving laterally to the channel through the aquifer.

To separate the hydrograph into its three constituent components, a method developed by Barnes (1940) was employed. The three-component separation involves separating surface runoff, inter-flow, and groundwater flow. This method is based on Equation 3.12, and involves determining recession of surface flow, inter-flow, and base flow.

\[ Q_t = Q_0 K_r, \]  

Where \( Q_0 \) is the initial discharge at any time, \( Q_t \) is the discharge at time interval \( t \) later, and \( K_r \) is the recession or depletion constant dependent upon the units of time and is less than unity. The time interval is usually taken as 24 hours, or one day, but a smaller value may be desirable for smaller drainage basins.

During recession, there is no inflow to the drainage basin. Therefore, the rate of change of the drainage-basin storage \( S_t \) depends entirely on the stream flow discharge:

\[ \frac{dS_t}{dt} = -Q_t \]  

After a long time, the basin is completely depleted of water. This condition can be expressed as:

\[ S_t = 0, t \to \infty \]  

Subject to this condition, an expression for \( S_t \) in terms of \( Q_t \) can be obtained by inserting Equation (3.12) into Equation (3.13) and then solving:

\[ \frac{dS_t}{dt} = -Q_0 K_r, \]  

which yields

\[ S_t = \frac{Q}{\ln K_r} \]  

From Equation (3.16), the storage remaining at any time \( t \) in the basin can be computed if \( Q_t \) and \( K_r \) are known.
The storage of water in the basin is comprised of surface storage (both surface detention and channel storage), inter-flow storage, and groundwater storage. Clearly, then, the recession constant $K_r$ will be different for these three types of storage. In order to account for these types of storage, $K_r$ can be considered to be made up of the recession constant for surface storage, $K_{rs}$; the recession constant for inter-flow, $K_{ri}$; and the recession constant for groundwater storage, $K_{rb}$. Equation (3.12) plots as a straight line on semi-logarithmic paper, with $K_r$ defining the slope of the line. When stream flow is plotted, it is not a straight line, but a curve with gradually decreasing slope or increasing value of $K_r$. This, of course, is due to the different types of storage contributing to stream flow. When the time interval is assumed in days, typical values of recession constants of these storage coefficients are $K_{rs} = 0.1$ to $0.5$, $K_{ri} = 0.5$ to $0.85$, and $K_{rb} = 0.85$ to $0.99$. If inter-flow is not significant, $K_{ri}$ can be assumed to be zero.

Equation (3.12) can be written for recession of each component of stream flow, with $K_r$ replaced by the component’s recession constant. Thus, stream flow recession can be approximated by three straight lines representing the three components on semi-logarithmic paper (Barnes, 1940). In reality, the transition from one line to the next is not always sharp and it becomes difficult to identify the points marking the change in slope of the line. The three straight lines are constructed as follows. The stream flow recession is plotted on semi-logarithmic paper. The last portion of the curve represents the recession of groundwater runoff. This portion will plot as a straight line and is projected backward. The slope of this line is the value of $K_{rb}$. The difference between the total stream flow recession and this straight line is plotted, which gives the recession of inter-flow and surface runoff, and the procedure is repeated. In this manner, recessions of surface flow and inter-flow, along with their respective constants $K_{rs}$ and $K_{ri}$, can be established.

The procedure explained above was applied to stream flow data to obtain direct runoff resulting from precipitation. In Figure 3.4, the groundwater recession plots approximately as straight line, with slope $K_r$. By extending this straight line under the hydrograph to the point directly under the point of inflection E and to B on line AB, points B and J are connected arbitrarily by a straight line. The area under the hydrograph above BJH is considered to be direct flow and that area below BJH is considered to be groundwater flow. The direct runoff is replotted and a straight line IL with slope $K_r$ is fitted and extended to point I directly under inflection point E and to the beginning point M. The line MIL divides the replotted hydrograph into surface runoff on top and inter-flow below.

Application of this method required considerable smoothing of the runoff hydrograph. Contributions of direct runoff from various parts of the drainage basin often produced numerous humps in the hydrograph recession and precluded computations for surface flow and inter-flow. Despite these problems, groundwater flow from these irregular recessions was accomplished using this method.
PART B: SEDIMENTATION MODELLING

3.3 GSTARS 2.1 Theoretical Concepts

GSTARS 2.1 (Generalized Stream Tube model for Alluvial River Simulation version 2.1) is one of the most recent version of numerical models for simulating the flow of water and sediment transport in alluvial rivers (Yang and Simoes, 2000). It is an enhanced version of the GSTARS 2.0 model (Yang et al., 1998). This section describes the overall theoretical background of the model, its capabilities and limitations.

3.3.1 Purpose and Capabilities

GSTARS 2.1 is a generalized water and sediment-routing computer model used to solve complex river engineering problems for which limited data and resources are available.

GSTARS version 2.1 consists of four major parts. The first part is the use of both the energy and momentum equations for the backwater computations. This feature allows the program to compute the water surface profiles through combinations of sub-critical and supercritical flows. In these computations, GSTARS can handle irregular cross sections regardless of whether the channels are single or multiple separated by small islands or sand bars.

The second part is the use of the stream tube concept, which is used in the sediment routing computations. Hydraulic parameters and sediment routing are computed for each stream tube, thereby providing a transversal variation in the cross section in a semi-two-dimensional manner. Although no sediment or flow can be transported across the boundary of a stream tube, the position and width of a stream tube can change after each time step of computation. The scour or deposition computed in each stream tube give the variation of channel geometry in the vertical or lateral direction. The water surface profiles are computed first. The channel is then divided into a selected number of stream tubes with the following characteristics:

1. the total discharge carried by the channel is distributed equally among the stream tubes,
(2) stream tubes are bounded by channel boundaries and by imaginary vertical walls,
(3) the discharge along a stream tube is constant, and
(4) there is no exchange of water or sediments through stream tube boundaries.

Bed sorting and armouring in each stream tube follows the method proposed by Bennett and Nordin (1977), and the rate of sediment transport can be computed using any of the sediment transport functions (section 3.3.4.6).

The third part is the use of the theory of minimum energy dissipation rate (Yang, 1971, 1976; Yang and Song, 1979, 1986) in its simplified version of minimum total stream power to compute channel width and depth adjustments. The use of this theory allows the channel width to be treated as an unknown variable. Whether a channel width or depth is adjusted at a given cross section and at a given time step depends on which condition results in less total stream power.

The fourth part is the inclusion of a channel bank side stability criteria based on the angle of repose of bank materials and sediment continuity.

Some of the potential applications and/or features of GSTARS 2.1 are:
- It is able to compute hydraulic parameters for open channels with fixed as well as with movable boundaries
- It can be used for water surface profile computations with or without sediment transport
- It can compute water surface profiles through sub-critical and supercritical flow conditions, including hydraulic jumps, without interruption
- It can compute the longitudinal and transversal variations of flow and sediment conditions in a semi-two-dimensional manner based on the stream tube concept. If only one stream tube is selected, the model becomes one-dimensional. If multiple stream tubes are selected, both the lateral and vertical bed elevation changes can be simulated.
- The bed sorting and armouring algorithm is based on sediment size fractions and can provide a realistic simulation of bed armouring process
- It can simulate channel geometry changes in width and depth simultaneously based on minimum total stream power
- The channel side stability option allows simulation of channel geometry change based on the angle of repose of bank materials and sediment continuity

3.3.2 Limits of Application

Although GSTARS 2.1 is intended to be used as a general engineering tool for solving fluvial hydraulic problems, it does have the following limitations from the theoretical point of view:
- It is a quasi-steady flow model. Water discharge hydrographs are approximated by bursts of constant discharges. Consequently, it should not be applied to rapid, varied, unsteady flow conditions
- It is a semi-two-dimensional model for flow simulation and semi-three-dimensional model for simulation of channel geometry change. It should not be applied to situations where a truly two-dimensional or truly three-dimensional model is needed for detailed simulation of local conditions
- It is based on the stream tube concept. The phenomena of secondary current, diffusion, and super-elevation are ignored.
- Many of the methods and concepts used in GSTARS 2.1 are simplified approximations of real phenomena. Those approximations and their limits of validity are, therefore, embedded in the model.

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3.3.3 The Backwater Model

The hydraulic computations in GSTARS 2.1 are based on a model of gradually varied flow. Mixed flow regimes and hydraulic jumps can be calculated by selectively using the energy and the momentum equations. This section presents the basic governing equations for flow computations.

For quasi-steady flows, discharge hydrographs are approximated by bursts of constant discharge. During each constant discharge burst, steady state equations are used for backwater computations. GSTARS 2.1 solves the energy equation based on the standard-step method. However, when a hydraulic jump occurs, the momentum equation is used instead.

3.3.3.1 Energy Equation

The basic equation used in most of the water surface profile computations is the energy equation. This can be written as

$$z_1 + y_1 + \alpha_1 \frac{V^2}{2g} = z_2 + y_2 + \alpha_2 \frac{V^2}{2g} + h,$$  \hspace{1cm} (3.17)

in which $z$ = bed elevation; $y$ = water depth; $V$ = flow velocity; $\alpha$ = velocity distribution coefficient; $h$ = total energy loss between sections 1 and 2; $g$ = gravitational acceleration; and subscripts 1 and 2 denote sections 1 and 2, respectively. This equation is valid when the channel’s bottom slope is small, i.e. when $S_o < 5\%$.

Equation (3.17) is solved using a trial-and-error procedure based on the standard step method (Henderson, 1966). The initial surface elevation is guessed, and that is iteratively improved by using

$$Z = \tilde{Z} - \frac{H - \tilde{H}}{1 - F_r^2 (1 + 0.5 C_L) \frac{3h_f}{2R}} \hspace{1cm} (3.18)$$

Where $Z$ = water surface elevation; $F_r$ = Froude number; $R$ = hydraulic radius; $h_f$ = friction loss; $C_L$ = energy loss coefficient; and the tilde is used to denote that the respective quantities are computed from the guessed value for the first iteration, and from the previously computed values for the remaining iterations. Computations proceed in the upstream direction for subcritical flows and in the downstream direction for supercritical flows. The Froude number is computed from

$$F_r^2 = \frac{\alpha V^2}{gW \cos \theta} \hspace{1cm} (3.19)$$

where $W$ = cross section width.

3.3.3.2 Flow Transitions

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Before starting the backwater computations, it is necessary to determine the flow regime, i.e., whether the flow conditions are supercritical, sub-critical, or critical. For this purpose, the normal and critical depths are computed along the study reach. The normal depth is set equal to a very large value when horizontal or adverse slopes are encountered. For the reaches where an hydraulic jump is detected, the momentum equation is used:

\[ \frac{Q\gamma}{g} (\beta_2 V_2 - \beta_1 V_1) = p_1 - p_2 + W_g \sin \theta - F_f \]  

(3.20)

where \( \gamma \) = unit weight of water; \( \beta \) = momentum coefficient; \( p \) = pressure acting on a given cross section; \( W_g \) = weight of water enclosed between sections 1 and 2; \( \theta \) = angle of inclination of channel; and \( F_f \) = total external friction force acting along the channel boundary. If the value of \( \theta \) is small (\( \sin \theta \approx 0 \)) and if \( \beta_1 = \beta_2 = 1 \), equation 3.20 becomes

\[ \frac{Q^2}{A_1 g} - \frac{Q^2}{A_2 g} = \frac{Q}{A_1 g} \tilde{y}_1 + \frac{Q}{A_2 g} \tilde{y}_2 \]  

(3.21)

where \( \tilde{y} \) = depth measured from water surface to the centroid of the cross section containing flow.

Equation (3.21) is solved by an iterative trial-and-error procedure.

### 3.3.3.3 Normal, Critical, and Sequent Depth Computations

The normal depth is computed by satisfying the equation

\[ g(D) = Q - k(D) \sqrt{S_0} = 0 \]  

(3.22)

where \( k(D) \) = conveyance, which is a function of the depth \( D \); and \( S_0 \) = bottom slope. For adverse and horizontal slopes, the normal depth is set to very high value.

Critical depth occurs where the Froude number has a value of 1 for a given discharge. In GSTARS 2.1, the critical depth is calculated by satisfying equation:

\[ f(D) = 1 - \alpha(D) \frac{Q^2 W(D)}{g A^2(D)} = 0 \]  

(3.23)

where \( W(D) \) = channel’s top width at a depth \( D \); and \( A(D) \) = channel cross-sectional area at depth \( D \).

Sequent depths for a given discharge are the depths with equal specific forces. The specific force of a natural channel can be expressed by

\[ SF(D) = \frac{Q^2}{A_g} + A_m \tilde{y} \]  

(3.24)

where \( SF(D) \) = specific force corresponding to a water depth \( D \); \( A_t \) = total flow area; and \( A_m \) = flow area in which motion exists. In GSTARS 2.1, the sequent depth is computed where
hydraulic jumps occur. An iterative trial-and-error procedure is used to find the sequent water surface elevation. The bisection method is used to solve equation

\[ SF(D_a) - SF(D_b) = 0 \]  

where \( D_a \) = computed supercritical water surface elevation, and \( D_b \) = desired sub-critical sequent water surface elevation.

### 3.3.3.4 Flow Resistance

One of the fundamental assumptions in GSTARS 2.1 is that a uniform flow formula can be used to compute the friction losses. This formula is used to compute the total conveyance, \( k \). The total conveyance \( k \) is used to determine the friction slope, \( S_f \), for a specified discharge:

\[ S_f = \left(\frac{Q}{k}\right)^2 \]  

In GSTARS 2.1; Manning’s, Chezy’s or Darcy-Weisbach formula can be used to compute \( k \).

### 3.3.4 Sediment Routing

Sediment transport occurs when the flow exceeds a certain threshold and becomes capable of moving the particles that constitute the bed. When the channel’s bed becomes mobile, erosion or deposition can occur. These bed changes depend on many parameters: (1) hydraulic conditions such as flow velocity and depth, (2) bed composition such as size of particles that constitute the bed, and (3) the amount and type of sediments entering the channel (supply rates). In this section, the sediment transport and bed evolution model employed in GSTARS 2.1 is presented.

#### 3.3.4.1 Governing Equations

##### 3.3.4.1.1 Sediment Continuity Equation

The basis for sediment routing in GSTARS 2.1 is the conservation of mass of sediments. In one-dimensional unsteady flow, the sediment continuity equation can be written as

\[ \frac{\partial Q_s}{\partial x} + \eta \frac{\partial A_d}{\partial t} + \frac{\partial A_s}{\partial t} - q_s = 0 \]  

where \( \eta \) = volume of sediment in a unit bed layer volume (1- porosity); \( A_d \) = volume of bed sediment per unit length; \( A_s \) = volume of sediment in suspension at the cross section per unit length; \( Q_s \) = volumetric sediment discharge; and \( q_s \) = lateral sediment inflow.

Assuming that the change in suspended sediment concentration in a cross section is much smaller than the change of the river bed and that during a time step, the parameters in the sediment transport function for a cross section remain constant then equation (3.27) can be simplified to yield

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which is the governing equation used in GSTARS 2.1 for routing sediments in rivers, streams
and reservoirs.

### 3.3.4.2 Discretization of the Governing Equations

The procedure used in GSTARS 2.1 to discretize equation (3.28) employs a finite difference
uncoupled approach. In order to accomplish the discretization process, the change in the volume of bed sediment
due to deposition or scour, $\Delta A_d$, is written as

$$\Delta A_d = (a p_{i,t-1} + b p_i + c p_{i,t+1}) \Delta Z_i$$

where $p = \text{wetted perimeter}$; $\Delta Z = \text{change in bed elevation (positive for aggradation and negative for scour)}$; $i = \text{cross section index}$; and $a, b, \text{and } c$ are constants that must satisfy

$$a + b + c = 1$$

Using expression (3.29), the partial derivative terms in equation (3.28) can be approximated
and inserted back in equation (3.28) to obtain

$$\Delta Z_{i,k} = \frac{2\Delta t(Q_{s,i-1,k} - Q_{s,i,k})}{\eta_i (ap_{i-1} + bp_i + cp_{i+1})(\Delta x_i + \Delta x_{i-1})}$$

where $k = \text{size fraction index}$; $\eta_i = \text{volume of sediment in a unit bed layer at cross section } i$;
and $Q_{s,i,k} = \text{computed volumetric sediment discharge for size class } k \text{ at cross section } i$. The
total bed elevation change for a stream tube at cross section $i$, $\Delta Z_i$, is computed from

$$\Delta Z_i = \sum_{k=1}^{N} \Delta Z_{i,k}$$

where $N = \text{total number of size fractions present in cross section } i$. The new channel cross
section at station $i$, to be used at the next iteration, is determined by adding the bed elevation change to the old bed elevation.

### 3.3.4.3 Numerical Stability

The formulation described in the previous section is subject to numerical stability constraints. Since GSTARS 2.1 uses an explicit method to solve the sediment routing equation, the Courant-Friedrichs-Lewy (CFL) condition for numerical stability should be defined. In this case, the CFL stability criterion is given by

$$\Delta t \leq \frac{\Delta x}{c_s}$$

where $c_s$ is kinematic wave speed of the bed changes.
3.3.4.4 Streamlines and Stream Tubes

GSTARS 2.1 routes sediments using stream tubes. By definition, a streamline is a conceptual line to which the velocity vector of the fluid is tangent at each and every point, and at each instant in time. Stream tubes are conceptual tubes whose walls are defined by streamlines (Fig. 3.5).

For steady and incompressible fluids, the total head, \( H_t \), along a stream tube of an ideal fluid is constant:

\[
\frac{p}{\gamma} + \frac{V^2}{2g} + h = H_t = \text{Constant}
\]

where \( p \) = pressure acting on the cross section; \( \gamma \) = unit weight of water; \( V \) = velocity; \( g \) = acceleration due to gravity; and \( h \) = hydraulic head.

In this model, however, \( H_t \) is reduced along the direction of the flow due to friction and other local losses. The backwater profiles are computed first. Then, the cross sections are divided into several sections of equal conveyance. These regions of equal conveyance are treated as stream tubes, and the computed locations of their boundaries are the defining streamlines, across which no water can pass. The defined stream tubes are thus used as if they were conventional channels with known hydraulic properties.

Stream tube locations are computed for each time step, therefore they are allowed to vary with time. Sediment routing is carried out using a sediment transport formula. This is done independently for each stream tube and for each time step. Bed material composition is computed for each tube at the beginning of the time step, and bed sorting and armouring computations are also carried out separately for each stream tube. In GSTARS 2.1, lateral variations of bed material composition are accounted for, and this variation is included in the computations of the bed material composition and sorting for each stream tube. Although no

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**Fig. 3.5: Schematic representation illustrating the use of stream tubes by GSTARS 2.1**

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material is allowed to cross stream tube boundaries during a time step, lateral movement of sediment is accomplished by the lateral variation of the stream tube boundaries from time step to time step. The change in bed levels are computed from the sediment continuity equation and are updated before the algorithm proceeds to the next time step. The time marching proceeds sequentially in this manner, until the desired time is reached.

This approach allows the computation of cross-sectional variations in the hydraulic and sediment parameters in a quasi-two-dimensional manner. Conventional one-dimensional models are unable to deal with this situation, but GSTARS 2.1 can, since erosion or deposition are computed separately within each stream tube, depending on the hydraulics, bed composition, transport capacity, and sediment supply conditions for each stream tube.

3.3.4.5 Bed Sorting and Armouring

GSTARS 2.1 computes sediment transport by size fraction. As a result, particles of different sizes are transported at different rates. Depending on the hydraulic parameters, the incoming sediment distribution, and bed composition, some particle sizes may be eroded, while others may be deposited or may be immovable. Finer particles may be eroded, leaving a layer of courser particles for which there is no carrying capacity. No more erosion may occur for those hydraulic conditions, and the bed is said to be armoured. This armour layer prevents the scour of the underlying materials and the sediment available for transport becomes limited to the amount of sediment entering the reach. However, future hydraulic events such as an increase of flow velocity may increase the flow carrying capacity, causing the armour layer to break and restart the erosion processes in the reach.

Many different processes may occur simultaneously within the same channel reach. These depend not only on the composition of the supplied sediment, but also on the bed composition within that reach. The bed composition may vary within the reach both in space and time. In order to model these type of events, GSTARS 2.1 uses the bed composition accounting procedure proposed by Bennett and Nordin (1977). The overall process is illustrated in Figure 3.6.

For a given time step, erosion of the bed or banks will take place when the sediment transport capacity at a given cross section exceeds the load incoming from the upstream cross section. When erosion takes place, sediment transport may be constrained by availability. The materials available for entrainment are those exposed at the bed surface. Therefore, for each simulation step, the only material available for erosion is the sediment contained in the active layer. This implies that the active layer thickness should always be at least as thick as the expected maximum depth of scour ($\Delta Z$).

3.3.4.6 Sediment Transport Functions

Most sediment transport formulas were developed for computing the total bed-material load without breaking it into load by size fraction. In GSTARS 2.1, these formulas have been modified to account for transport by size. The total carrying capacity for a particular river section, $C_i$, is computed by using the following relationship:

$$C_i = \sum_{j=1}^{N} \left[ p_j + (1 - r) p_j^* \right] C_i$$

(3.35)

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where $p_i$ = percentage of material of size fraction $i$ available in the bed; $p_i^*$ = percentage of material of size fraction $i$ incoming into the reach; $C_i$ = capacity for each size fraction; $N$ = number of size fractions; and $r$ = a factor ($0 \leq r \leq 1$).

The factor $r$ is a weighting factor that allows the inclusion of incoming sediment into the carrying capacity of the flow. The hydraulic parameters used to compute the sediment carrying capacities in each reach are computed as weighted averages from the hydraulic parameters for the nearby stations. For each station $i$, the representative values of the area ($A_{Ri}$), depth ($D_{Ri}$), velocity ($V_{Ri}$), and friction slope ($S_{Ri}$) are computed as follows:

$$A_{Ri} = aA_{i-1} + bA_i + cA_{i+1}$$  \hfill (3.36)

$$D_{Ri} = aD_{i-1} + bD_i + cD_{i+1}$$  \hfill (3.37)

$$V_{Ri} = aV_{i-1} + bV_i + cV_{i+1}$$  \hfill (3.38)

$$S_{Ri} = aS_{i-1} + bS_i + cS_{i+1}$$  \hfill (3.39)

The weighting parameters $a$, $b$, and $c$ can be chosen in any combination to satisfy equation (3.30).

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The rate of sediment transport, $C_i$, can be computed using any of the following methods:

- DuBoys’ 1879 method
- Meyer-Peter and Müller’s 1948 method
- Laursen’s 1958 method
- Toffaleti’s 1969 method
- Englund and Hansen’s 1972 method
- Ackers and White’s 1973 method
- Revised Ackers and White’s 1990 method
- Yang’s 1973 sand and 1984 gravel transport methods
- Yang’s 1979 sand and 1984 gravel transport methods
- Parker’s 1990 method.
- Yang’s 1996 modified formula

The selection of the appropriate sediment transport function remains an unsolved problem since no particular equation or method can be used under all circumstances. However, guidelines based on those given by Yang (1996) could be helpful when choosing the sediment transport formula for a particular study:

1. Field and measured data should be used as much as possible, within the time, budget, and manpower limits of each particular study.
2. As many formulas as possible should be examined, based on assumptions used in their derivation and range of data used to determine its coefficients. Those consistent with the data and measurements obtained in step 1 should be selected.
3. If more than one formula survived step 2, sediment transport rates are computed with those formulas and those that best agree with any field measurements in step 1 are selected.
4. In the absence of measured sediment loads for comparison, the following guidelines could be considered:
   - Meyer-Peter and Müller’s formula is used when the bed material is coarser than 5 mm.
   - Toffaleti’s formula is used for large sand-bed rivers.
   - Yang’s (1973) formula is used for sand transport in laboratory flumes and natural rivers. Yang’s (1979) formula is used for sand transport when critical unit stream power at incipient motion can be neglected.
   - Parker’s (1990) or Yang’s (1984) gravel formulas are used for bed load or gravel transport.
   - Yang’s (1996) modified formula for high-concentration flows is used when the wash load or concentration of fine material is high.
   - Ackers and White’s or Engelund and Hansen’s formulas are used for subcritical flow condition in the lower flow regime.
Laursen’s formula is used for laboratory flumes and shallow rivers with fine sand and coarse silt.

Since most of the sediments entering the study reservoir are fine silt and clay, the cohesive sediment part of GSTARS 2.1 was chosen for use in conjunction with the non-equilibrium sediment transport option. Hence, only the sediment transport function dealing with cohesive sediments is described here. In GSTARS 2.1, only Yang’s modified formula (Yang et al., 1996) describes a case of cohesive sediment transport with high concentration of wash load.

### 3.3.4.6.1 Yang’s Modified Formula

The existence of high concentration of wash load can significantly affect the flow viscosity, sediment fall velocity, and the relative density or relative specific weight of sediment. For a given set of hydraulic conditions, non-equilibrium sediment transport of varying rates may occur because of a varying rate of high concentration of wash load. Yang et al. (1996) rewrote Yang’s 1979 formula in the following form for sediment-laden flow with high concentration of wash load:

\[
\begin{align*}
\log C_n &= 5.165 - 0.153 \log \frac{\omega_m d}{V_{m}} - 0.297 \log \frac{U^*}{\omega_m} \\
&+ \left(1.780 - 0.360 \log \frac{\omega_m d}{V_{m}} - 0.480 \log \frac{U^*}{\omega_m}\right) \log \left(\frac{\gamma_s}{\gamma_m} \frac{V S}{\gamma_s - \gamma_m \omega_n}\right),
\end{align*}
\]

where \(C_n\) = total sediment concentration in parts per million by weight; \(\omega_m\) = particle fall velocity in sediment-laden flow; \(V_{m}\) = kinematic viscosity of sediment-laden flow; \(d\) = sediment particle diameter; \(U^*\) = shear velocity; \(V S\) = unit stream power; \(V\) = average flow velocity; \(S\) = water surface or energy slope; and \(\gamma_s, \gamma_m\) = specific weights of sediment and sediment-laden flow, respectively.

### 3.3.4.7 Cohesive Sediment Transport

In this study, cohesive sediments are those whose particles pass through a 62.5 μm sieve, a definition that follows the nomenclature of the American Geophysical Union (Lane, 1947). At present, the equations for computing the transport potential of cohesive sediments implemented in GSTARS 2.1 are considered state-of-the-art (Partheniades, 1986; Mehta et al., 1989) since reliable predictive techniques are still not available.

The transport of silt and clay is computed separately from the remaining size fractions. The model recognises the presence of clay if any of the particle size fractions given in the input has a geometric mean grain size, \(d_{\text{mean}}\), smaller than 0.004 mm. Similarly, the presence of silt is recognised if a size fraction has \(d_{\text{mean}}\) between 0.004 and 0.0625 mm. The transport of fractions with \(d_{\text{mean}} \geq 0.0625\) mm is computed by traditional transport equations as described by Yang (1996). For smaller fractions, the methods described in this section are used.

### 3.3.4.7.1 Deposition

The occurrence of erosion or deposition is controlled by the value of the bed shear stress, \(\tau_b\). Deposition of clay and silt takes place when \(\tau_b\) is smaller than the critical bed shear stress for
deposition, \( \tau_{cd} \). In this case, the deposition is governed by integrating

\[
\frac{dC}{dt} = -\frac{p \omega_s C}{h}
\]  

(3.41)

where \( C = \) depth-averaged concentration of sediments, \( h = \) the water depth, and \( \omega_s = \) the settling velocity of the sediment. \( p \) is a parameter representing the probability for deposition.

When \( \omega_s \) does not depend on the concentration of suspended sediments (unhindered settling), equation (3.41) can be integrated analytically to yield

\[
\frac{C}{C_0} = \exp\left\{ -\frac{\omega_s \Delta t}{h} \left( 1 - \frac{\tau_b}{\tau_{cd}} \right) \right\}
\]  

(3.42)

where \( C_0 \) and \( C \) are the concentration at the beginning and end of time step \( \Delta t \). The time of residence, \( \Delta t \), is obtained from \( \Delta x/V \), where \( \Delta x \) is the reach length and \( V \) is the velocity of the flow. Using equation (3.42), GSTARS 2.1 computes sediment concentration. The concentration obtained is converted into volume and deposited on the bed.

### 3.3.4.7.2 Erosion

Erosion of silt and clay takes place when \( \tau_b \) is greater than the critical bed shear stress for particle scour, \( \tau_{cs} \). GSTARS 2.1 recognises two modes of erosion of cohesive beds: particle erosion and mass erosion. The first mode corresponds to the state where the erosion proceeds particle by particle, or aggregate by aggregate. The second mode corresponds to a state where the bed is destroyed by the eroding currents and entire blocks of mud are swept away. Particle erosion takes place when \( \tau_b > \tau_{cs} \). Mass erosion takes place when \( \tau_b \) increases past the critical bed shear stress, \( \tau_{cm} \) for mass erosion. The following equations are used for the particle and mass erosion rates, respectively (Partheniades, 1965; Ariathurai and Krone, 1976):

\[
\tau_{cs} < \tau_b \leq \tau_{cm}: \quad E_1 = \frac{1}{A} \frac{dm}{dt} = M_1 \left( \frac{\tau_b}{\tau_{cs}} - 1 \right)
\]  

(3.43)

\[
\tau_b > \tau_{cm}: \quad E_2 = \frac{1}{A} \frac{dm}{dt} = M_2 \left( \frac{T_e}{\Delta t} \right)
\]  

(3.44)

where \( m = \) mass; \( t = \) time; \( \Delta t = \) time step; \( M_1, M_2 = \) material constants that depend on mineral composition, salinity, organic material, etc., with units of mass per unit area and time; \( A = \) bottom area; and \( E_1 = \) particle erosion rate per unit of area; \( E_2 = \) mass erosion rate per unit of area; and \( T_e = \) characteristic time of erosion.

### 3.3.4.8 Non-equilibrium Sediment Transport

In rivers and streams, the exchange of sediment between the bed and the fractions in transport is instantaneous (equilibrium process). In reservoirs and estuaries however, the spatial-delay and/or time-delay effects are important. Hence, reservoir sedimentation processes and siltation of estuaries are essentially non-equilibrium processes. To model this processes, GSTARS 2.1 uses the method developed by Han (1980) for suspended load. This method is
based on the analytical solution of the convection-diffusion equation which computes the non-equilibrium sediment transport rate as:

\[
C_i = C_i^* + \left( C_{i-1}^* - C_{i-1}^* \right) \exp \left\{ -\frac{\alpha \omega_s \Delta x}{q} \right\} + \\
\left( C_{i-1}^* - C_i^* \right) \left\{ \frac{q}{\alpha \omega_s \Delta x} \left[ 1 - \exp \left\{ -\frac{\alpha \omega_s \Delta x}{q} \right\} \right] \right. \\
\left. \right. \\
\] (3.45)

where \(C = \) sediment concentration; \(C^* = \) sediment carrying capacity; \(q = \) discharge of flow per unit width; \(\Delta x = \) reach length; \(\omega_s = \) sediment fall velocity; \(i = \) cross-section index (increasing from upstream to downstream); and \(\alpha = \) a dimensionless parameter. Equation (3.45) is employed for each of the particle size fractions in the cohesionless range, i.e., with diameter greater than 62.5 \(\mu m\). The parameter \(\alpha\) is a recovery factor. Han and He (1990) recommended a value of 0.25 for deposition and 1.0 for entrainment.

### 3.3.5 Tributary Inflow

Although GSTARS 2.1 is limited to single stem channels, it is possible to include the contributions of water and sediment by tributaries into the modelled reach. At channel junctions (Fig.3.7) continuity requires that

\[
Q_c = Q_A + Q_B \\
\] (3.46)

If cross section B is located at the tributary, and cross section A and C represent the computational cross sections used to model the tributary effects, then using the principle of energy conservation

\[
\left( z + Y + \alpha \frac{V^2}{2g} \right)_{A} = \left( z + Y + \alpha \frac{V^2}{2g} \right)_{C} + h_f \\
\] (3.47)

The energy losses, \(h_f\) are computed from friction alone. Losses due to bends, contractions and expansions are ignored.

![Fig. 3.7: Channel junction](image)
3.3.6 Data Requirements for GSTARS 2.1

Application of the GSTARS 2.1 computer model requires the use of appropriate data. The input file is divided in four parts: the hydraulics data, the sediment data, the printout control, and the stream power minimisation data. The hydraulic and printout parts are always required. The other two parts are optional. The stream power minimisation data can only be present if the sediment data is included.

3.3.6.1 Hydraulics Data

The hydraulics data required to run GSTARS 2.1 is composed of channel geometry and hydrologic data. Channel geometry requirements include data on cross section geometry, channel roughness, and loss coefficients. Hydrologic data consists of water discharges and stages.

3.3.6.1.1 Channel Geometry, Roughness, and Loss Coefficient Data

The first step to model a river/reservoir system using GSTARS 2.1 involves the approximation of the channel’s bed and banks in a semi-two-dimensional manner. The reach to be modelled must be described by a finite number of discretized cross sections. Cross section geometry is described by X-Y coordinate pairs, i.e., by coordinate pairs with lateral location and bed elevation. Bed elevations (Y) must be taken using a common datum for the entire reach and must always be positive. Lateral locations (X) must be given using a reference point for each cross section, and the coordinate pairs must be entered in order of increasing X coordinate, i.e., starting from the left hand side of the cross section and marching towards the right-hand side while looking downstream as shown in Figure 3.8. The cross sections should be perpendicular to the direction of the flow and extend all the way from margin to margin of the river/reservoir, i.e., they should extend completely across the channel between high ground of both banks.

The number and positions of the cross sections are arbitrary. However, they should be chosen to best represent the geometry of the study channel reach. In GSTARS 2.1, each cross section

Fig. 3.8: Schematic representation of the discretization of a reach by three cross-sections
represents a portion of the channel upstream and downstream from its actual location as shown in Figure 3.8. More cross sections are required where there are significant changes in channel geometry and/or hydraulic characteristics. A larger number of cross sections will approximate the channel reach geometry with more accuracy than a smaller number will. Ideally, as many cross sections as practicable should be used. In the case where too few measured cross sections are available, they may have to be interpolated, especially at abrupt transitions.

Each cross section can be divided into several regions, or channel divisions, of constant roughness. Each one of these regions would have its own value of the roughness coefficient. In GSTARS 2.1, up to nine channel divisions may be defined. Therefore up to nine different roughness coefficient values may be entered for each cross section. Manning, Chezy, or Darcy-Weisbach equations can be selected to compute the local friction slope, with corresponding roughness coefficients entered from left to right across the section.

The local energy loss coefficients account for the hydraulic impacts of bends, natural and man-made structures, etc., at or upstream from the cross section. The default value for the local energy loss is zero. Internally, GSTARS 2.1 sets an additional coefficient of loss to 0.1 for contractions and to 0.3 for expansions.

3.3.6.1.2 Discharge and Stage Data

The hydraulic data necessary for a numerical simulation are water stages and corresponding surface elevation at certain points (boundary conditions). The inflow discharge hydrograph entering the study reach, i.e., at the station farthest upstream, must be given for the period of analysis. The water stage hydrograph must be given for the station farthest downstream.

In GSTARS 2.1, discharge hydrographs are given in tables with the discretized values in multiples of a fixed time increment. Corresponding water surface elevations are given either in tabular format or as stage-discharge rating curves. There are three options for entering data on discharge and stage.

(1) Discharge hydrograph with a stage-discharge rating curve. In this case, the water discharges for the reach are given in the form of a hydrograph, and the corresponding water stages are given as a rating curve, i.e., as a function of the discharge.
(2) Table of discharges with a rating curve at the control section. This type of input is a particular form of the first one. It may be useful when discharges are known from periodic records, at fixed time intervals, in form of a table.
(3) Stage-discharge table at a control section. In this case, the information for the control station is given in a table with discharges and corresponding water stages.

3.3.6.2 Sediment Data

In cases where sediment routing computations are required, sediment data must be given to the model. This includes bed material size distributions for the reach of study, sediment inflow hydrograph entering the reach and its particle size distribution.

3.3.6.2.1 Sediment Inflow Data

The inflow sediment hydrograph must be given for the section farthest upstream of the study reach. In this model, it can be given in form of either discretized sediment discharges or a
sediment rating curve. In the latter case, sediment discharge is specified as a function of water discharge in the form

\[ Q_s = aQ^b \]  

(3.48)

where \( Q_s \) = incoming sediment discharge; \( Q \) = water discharge; and \( a, b \) are coefficients to be supplied to the model \((a, b > 0)\).

Caution should however be exercised when using a relationship similar to equation (3.48). In most practical cases, equation (3.48) represents only an approximation, and often a very poor one (Yang and Simoes, 2000). However, the facility to use equation (3.48) is provided by GSTARS 2.1 for those cases in which its use may be warranted.

By default, the size distribution of the incoming sediment is set equal to the gradation given for the cross section farthest upstream. However, specified gradations are given as a function of the water discharge, and their input must be given in tabular format. The model interpolates the gradations for water discharges falling in between the specified discharges, but does not extrapolate for water discharges outside that range.

### 3.3.6.2.2 Sediment Gradation Data

Sediment mixtures are characterised by gradation curves. A common way of depicting bed gradation distributions is by a graph that shows, for each grain size, the percentage of bed sediments with a smaller size. In GSTARS 2.1, these information must be given for the bed composition of all the cross sections, and for the grain size distribution of the incoming sediment discharges.

The size classes of sediments must first be defined by entering the lower and upper bound of that class. Then the number of size fractions is entered. A maximum of 10 size fractions may be defined in GSTARS 2.1. For specific size fractions, different values for the dry specific weight, \( \gamma_{ms} \), can be used.

### 3.3.6.2.3 Temperature Data

Water temperature data is necessary for kinematic viscosity computations. For a given reach, this must be specified for each time step of the run.

### 3.3.6.2.4 Cohesive Sediment Transport Parameters

The parameters necessary to model cohesive sediment transport are schematically represented in Figure 3.9. Because these parameters are highly case dependent, and because they vary within orders of magnitude, GSTARS 2.1 does not assume default values for these quantities. When modelling cohesive sediment transport, field data should be relied on as much as possible.

In Figure 3.9, STDEP = shear threshold for deposition of clay and silt; STPERO = shear threshold for particle erosion of clay and silt; STMERO = shear threshold for mass erosion of clay and silt; ERMASS = slope of the erosion rate curve for mass erosion; and ERSTME = erosion rate of clay and silt when the bed shear stress is equal to STMERO. Finally, a last parameter is needed, ERLIM, which is the threshold value for the percentage of clay in the
Fig. 3.9: Schematic representation of the parameters necessary to model transport of cohesive sediments.

bed composition above which the erosion rates of gravels, sands, and silts are limited to the erosion rate of clay. Yang and Simoes (2000) found values of ERLIM to have a large range of variation ($7\% \leq \text{ERLIM} \leq 80\%$).

3.3.6.3 Tributary Inflow Data

The information necessary to model the effects of tributary flow are the tributary’s water discharge, the inflow sediment and its composition. This is needed if sediment transport computations in the model are active. Data on how sediment mixing takes place among the stream tubes is also necessary.

The information above is set-up in separate files. There must be one file for each tributary and these file names are passed on to GSTARS 2.1. All data for the tributary are tabulated in the same format as those in the main channel.
CHAPTER FOUR

SUSPENDED SEDIMENT GENERATOR

4.1 Description of Study Catchment

4.1.1 Introduction

The Masinga dam catchment area is some 7335 km\(^2\) in extent, lying to the east of the Aberdare mountains and south of Mount Kenya. Its location on the Kenyan map is shown on Fig. 4.1. Fourteen sub-catchments (Fig. 4.9) were derived from Masinga catchment and their areas are shown in Tables 4.7 and 4.8. In physical terms, the catchment can be broadly divided into four zones based upon the landform units of the Kenya Soil Survey (Table 4.1).

Population pressure and urban development have been increasing rapidly in recent years, leading to the extension of arable land onto extremely steep slopes. Progressive destruction of forest cover has occurred. The legal slope limit for cultivation has recently been increased from 35 to 55%. The lowlands have been subject to overgrazing, leading to increased rates of soil erosion and reduced water conservation.

![Fig. 4.1: Location of Masinga and Tana catchments](image)

The combined effects of easing of enforcement of laws relating to conservation plus the population pressure have led to an accelerated rate of soil loss and increase in river sediment loads. The accelerated soil degradation led to several soil and water conservation projects such as the EEC funded Machakos and ODA funded Embu study (Atkins, 1984). These projects have concentrated on the lower rainfall areas where moisture conservation is as much a priority as erosion control. Little work has so far been conducted in the higher rainfall areas,
despite the apparently high soil losses, where surplus water disposal is an important consideration.

### 4.1.2 Geology

The geology of Masinga catchment can be broadly divided into volcanic rocks in the north and west and pre-cambrian basement complex in the south-east. This distinction is significant to both soil formation and landforms. Other geologic formations of limited extent include igneous intrusions of granite and dolerite into the basement system and an area of quaternary sandstone between Murang’a and Sagana.

### 4.1.3 Landform and Slope

The landform in the area ranges from steep mountainous terrain with strong relief in the West, to undulating plains with subdued relief in the south-east. The variation in the slope classes is illustrated in Figure 4.2.

<table>
<thead>
<tr>
<th>Zone name</th>
<th>Description</th>
<th>Elevation (m)</th>
<th>Mean annual rainfall(mm)</th>
<th>Proportion of total study area, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mountainous</td>
<td>Upper slopes of Aberdares</td>
<td>2500-4000</td>
<td>&gt;1800</td>
<td>3.5</td>
</tr>
<tr>
<td>Uplands</td>
<td>Upper foot slopes of Aberdares and Mt. Kenya</td>
<td>1500-2500</td>
<td>1400-1800</td>
<td>24.9</td>
</tr>
<tr>
<td>Intermediate</td>
<td>Lower foot slopes of Aberdares and Mt. Kenya</td>
<td>1200-1500</td>
<td>900-1400</td>
<td>15.4</td>
</tr>
<tr>
<td>Lowlands</td>
<td>Undulating plains and plateaux</td>
<td>900-1200</td>
<td>600-900</td>
<td>53.6</td>
</tr>
</tbody>
</table>

### 4.1.4 Climate

The climate of Masinga catchment ranges from semi-arid in the east to humid near the western watershed. Four fairly distinct 3-month seasons are recognisable. From December to February, dry conditions prevail. Between March and May easterly winds bring heavy rainfall (the ‘long rains’). The following three months are mainly dull, dry and cool. The short rains occur during September to November when the wind is again easterly.

The rainfall is influenced strongly by orographic effects. Values range from about 600 mm on the eastern boundary to over 2000 mm on the Aberdare mountains. The average monthly rainfall amounts at seven stations within the catchment are shown in Table 4.2 and on Figure 4.3. Available data on rainfall intensities suggest that 2 year 24 hour values increase eastwards from about 50 mm to over 80 mm but no relationship is evident between rainfall intensities and regional characteristics.

Erosive rains tend to be concentrated in the early part of the rainy seasons, especially in the semi-arid lowlands. This is at a time of severe moisture deficit, when vegetative cover is at a

*Saenyi, Wycliffe W.*
minimum and farmland is clean-cultivated for sowing. The land is therefore extremely prone to erosion at this time.

**Fig. 4.2: Slope classification map**

**Table 4.2: Mean Monthly Rainfall at the Seven Stations Within the Catchment**

<table>
<thead>
<tr>
<th>Month</th>
<th>Gatere Forest Station 90.36259 2590m asl (10yrs)</th>
<th>Murang’a Gateigoro School 90.37063 1520m asl (25yrs)</th>
<th>Murang’a District Office 90.37007 1280m asl (70yrs)</th>
<th>Thika Bendor Plantation 90.37047 1580m asl (47yrs)</th>
<th>Kerugoya Hospital Sagana 90.37031 1570m asl (36yrs)</th>
<th>Mitubiri Kitito C.Estate 90.37016 1460m asl (41yrs)</th>
<th>Kangundo Kithimani D.O. 91.37074 1280m asl (21yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>112</td>
<td>45</td>
<td>33</td>
<td>35</td>
<td>32</td>
<td>47</td>
<td>39</td>
</tr>
<tr>
<td>Feb</td>
<td>142</td>
<td>42</td>
<td>42</td>
<td>40</td>
<td>31</td>
<td>42</td>
<td>35</td>
</tr>
<tr>
<td>Mar</td>
<td>212</td>
<td>131</td>
<td>110</td>
<td>105</td>
<td>99</td>
<td>114</td>
<td>105</td>
</tr>
<tr>
<td>April</td>
<td>511</td>
<td>403</td>
<td>310</td>
<td>231</td>
<td>353</td>
<td>247</td>
<td>149</td>
</tr>
<tr>
<td>May</td>
<td>483</td>
<td>285</td>
<td>203</td>
<td>143</td>
<td>312</td>
<td>123</td>
<td>74</td>
</tr>
<tr>
<td>June</td>
<td>157</td>
<td>46</td>
<td>45</td>
<td>30</td>
<td>51</td>
<td>17</td>
<td>9</td>
</tr>
<tr>
<td>July</td>
<td>107</td>
<td>39</td>
<td>20</td>
<td>18</td>
<td>53</td>
<td>7</td>
<td>3</td>
</tr>
<tr>
<td>Aug</td>
<td>103</td>
<td>52</td>
<td>23</td>
<td>20</td>
<td>67</td>
<td>7</td>
<td>3</td>
</tr>
<tr>
<td>Sept</td>
<td>93</td>
<td>39</td>
<td>26</td>
<td>21</td>
<td>52</td>
<td>11</td>
<td>4</td>
</tr>
<tr>
<td>Oct</td>
<td>271</td>
<td>160</td>
<td>120</td>
<td>74</td>
<td>178</td>
<td>82</td>
<td>58</td>
</tr>
<tr>
<td>Nov</td>
<td>351</td>
<td>24</td>
<td>189</td>
<td>158</td>
<td>188</td>
<td>210</td>
<td>207</td>
</tr>
<tr>
<td>Dec</td>
<td>151</td>
<td>93</td>
<td>74</td>
<td>79</td>
<td>70</td>
<td>110</td>
<td>79</td>
</tr>
<tr>
<td>Total</td>
<td>2693</td>
<td>1575</td>
<td>1196</td>
<td>955</td>
<td>1486</td>
<td>1019</td>
<td>764</td>
</tr>
</tbody>
</table>
Fig. 4.3: Mean monthly rainfall for 7 weather stations within Masinga catchment
The seasonal variation of average daily temperatures is small and is only of the order of 5°C. Maximum mean temperatures of 25.5°C - 31°C are generally experienced in February or March prior to the onset of the main rainy season while minimum mean temperatures of 21°C-24°C occur in the month of July. The diurnal range of temperature averages 13°C during the year but varies between a maximum of 17.5°C in February and a minimum of 11.0°C in the month of May.

4.1.5 Soils and Erodibility

Lithosols and Histosols occur at the highest altitudes in the Aberdare Range, with Humic Andosols at slightly lower elevations. Over much of the rest of the basalt foot slopes, deep, friable clays (Eutric Nitisols) predominate. On basement complex the soils are mostly coarser textured and shallower; they are classified as Acrisols, Luvisols and Ferralsols. Broad soil units are shown in Figure 4.4. An assessment of the relative erodibility of the soil units is given in Table 4.3.

4.1.6 Land Use

Masinga catchment has an estimated population of 2 million, according to 1998 census. Most of these are assumed to be engaged in agricultural activities. During the aerial survey of 1980, 26,600 ha were under cultivation while 11,600 ha was left fallow giving a total of 38,200 ha or 37% of the catchment area being used for cropping. Almost all the cultivation is taking place in the south-east, north-west and generally in the western area where rainfall is higher (Fig 4.5). The remainder of the area is used for grazing, with only scattered cultivation.
Table 4.3: Erodibility of Soil Units

<table>
<thead>
<tr>
<th>Unit</th>
<th>Principal Soils</th>
<th>Texture</th>
<th>Depth (m)</th>
<th>Possible Erodibility k-factor (relative)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>soils on Volcanic Rocks and Ash</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M2</td>
<td>Humic Andosols</td>
<td>CL-C</td>
<td>&lt;0.5-1.8</td>
<td>moderate to high</td>
</tr>
<tr>
<td>M9</td>
<td>Dystric Histosols and Lithosols</td>
<td>Variable</td>
<td>&lt;0.5-0.8</td>
<td>high</td>
</tr>
<tr>
<td><strong>Soils on Basic Igneous Rocks</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R1</td>
<td>Humic Nitisols and Humic Andosols</td>
<td>C</td>
<td>1.8</td>
<td>low</td>
</tr>
<tr>
<td>R2</td>
<td>Humic Nitisols</td>
<td>C</td>
<td>1.8</td>
<td>low</td>
</tr>
<tr>
<td>R3</td>
<td>Eutric Nitisols with Chromic Cambisols and Chromic Acrisols</td>
<td>C</td>
<td>Variable</td>
<td>moderate</td>
</tr>
<tr>
<td>L1</td>
<td>Rhodic Ferralsols</td>
<td>C</td>
<td>1.2-1.8</td>
<td>low</td>
</tr>
<tr>
<td>L2</td>
<td>Eutric Nitisols</td>
<td>C</td>
<td>1.2-1.8</td>
<td>low</td>
</tr>
<tr>
<td>L4</td>
<td>Rhodic Ferralsols and Chromic Cambisols (incl. Lithic phase)</td>
<td>C</td>
<td>&lt;0.5-1.8</td>
<td>moderate to high</td>
</tr>
<tr>
<td>L11</td>
<td>pellic Vertisols (incl. Stony phase)</td>
<td>C</td>
<td>1.2-1.8</td>
<td>low to moderate</td>
</tr>
<tr>
<td><strong>Soils on Basement Complex Gneisses</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H12</td>
<td>Ferralic Cambisols and Rhodic or Orthic Ferralsols</td>
<td>SCL-SC</td>
<td>&lt;0.5-0.8</td>
<td>moderate</td>
</tr>
<tr>
<td>H15</td>
<td>Regosols/Lithosols</td>
<td>SCL-C</td>
<td>&lt;0.5</td>
<td>high</td>
</tr>
<tr>
<td>Um19</td>
<td>Chromic/Ortho/ Ferric Acrisols</td>
<td>SC-C</td>
<td>&lt;0.5-0.8</td>
<td>high</td>
</tr>
<tr>
<td>Um20</td>
<td>Rhodic/Orthic Ferralsols</td>
<td>SCL-C</td>
<td>&lt;0.5-0.8</td>
<td>high</td>
</tr>
<tr>
<td>F16</td>
<td>Ferralic Arenosols</td>
<td>Variable</td>
<td>0.8-1.8</td>
<td>high</td>
</tr>
<tr>
<td><strong>Soils on Gneiss rich in Ferromagnesian Minerals</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up4</td>
<td>Pellic Vertisols and Eutric Nitisols</td>
<td>C</td>
<td>&lt;0.5-1.8</td>
<td>moderate</td>
</tr>
</tbody>
</table>

Source: Atkins (1984)

Suggested range of values of k (metric units): Low (0-0.15), moderate (0.15-0.40), High (>0.4)
4.1.6.1 Cultivation

The only soils extensively cultivated in the eastern half of the area are the vertisols. Where this soil type is found outside reserved ranching areas, cultivation is more or less 100%. Land preparation is carried out mainly by hired tractors, but also by oxen and by hand. The crop grown is mostly maize, interplanted with grams and beans. Some sorghum and millet are also grown.

There are only scattered areas of cultivation in the east on soils other than vertisols. Such cultivation is on slopes of up to 15% and rather severe erosion is taking place, including gullyng. The contrast in the appearance of crops in the two soil types is quite striking; maize on vertisols being reasonably well grown and green, while millet on adjacent well drained red clays usually fail.

Crops are planted in both rainy seasons if rains appear adequate, but in years of poor rainfall some areas, particularly those on the red clays are left fallow. The above pattern of cultivation extends westwards to the central part of the area, but the amount of cultivation on the red clay soils increases with increased rainfall towards the west. In times of good rainfall, crops here give at least significant yields despite the rather low standard of husbandry.

In the south-west of Masinga catchment area, over 1000 ha of sisal has been grown in the past but much of this is not being maintained. The high rainfall area in the extreme south-west has coffee and tea estates in addition to the normal subsistence crops.

The north-west of the catchment area on the brown clay soils has extensive cultivation of maize interplanted with sunflower and pigeon peas. On the dark greyish brown soils, in depressions which tend to be waterlogged, maize is often planted after the rains to grow on residual moisture.

Other minor crops found in the catchment area include mangoes, bananas, pawpaw, cotton, marrow, sweet potatoes and sugar cane.

In general, the standard of cultivation and crop husbandry is rather low and in only a few cases have any physical conservation measures been constructed. However, contour cultivation is usually practised. Storm-water diversions have been constructed recently on arable land with 10 to 15% slopes. As can be expected, both splash and gully erosion are fairly severe where slopes of up to 15% are common. Erosion is less on vertisols which have rather gentle slopes, but small gullies are common on slopes of as little as 2% on this soil type, particularly near the edge of the watercourses. The well structured soils of the high rainfall zone are relatively resistant to erosion, but the very steep slopes make special protection measures essential.

4.1.6.2 Rangeland

Land which is considered unsuitable, or for some other reasons is not available for cultivation is utilised for grazing the rather large number of cattle, goats and sheep. There are areas set aside for co-operative ranches although at present they are still used for individual grazing without improved management. Cattle and goats are herded throughout the remainder of the available range areas and no form of grazing control is exercised.

Saenyi, Wycliffe W.
With a stocking rate of about 1.5 livestock units per ha of rangeland, no management systems, and a series of three years of below average rainfall, the present degraded state of the rangeland is expected. Much of the area is almost completely denuded of grass and litter cover and moderate to fairly severe splash erosion is prevalent. Where water has been concentrated in cattle tracks and paths, gullies have formed. Another serious consequence of raindrop impact on the bare soil is the capping effect which is widespread throughout the rangeland with the exception of the vertisols, leading to reduced infiltration rates and hence an increase in runoff. Over a number of years, this has had the effect of altering the soil conditions so that only the more drought resistant species can survive. In some areas bush intensification is in evidence although not widespread.

Fig. 4.5 Land use map

Near Ekarakara hill, controlled grazing is being undertaken in individual plots and in many of these, the range is making an excellent recovery. However, the condition of cattle is generally poor at present that not all can survive until the next rainy season.

In general, the whole catchment area is suitable for cattle production under a high standard of range management.

4.1.6.3 Agricultural Potential

The limiting factor in utilising the area for agriculture is the lack of moisture for plant growth due to the rather low rainfall and its great variation from season to season. The only part of the catchment area which is highly suitable for arable cropping is the extreme western edge but intensive soil conservation measures are required due to the rather steep slopes. Areas adjacent to the above can be considered suitable for short season crops on soils with high water storage capacities.
In the remainder of the catchment area, the vertisols can be considered marginally to moderately suitable for arable cropping of short-term and drought resistant varieties due to their low runoff characteristics, imperfect drainage, high water storage capacity and relatively high inherent fertility.

Provided that water can be made available, gentler sloping vertisol areas are suitable for flood irrigation. The red and brown clays, due to topography and soil characteristics, are only considered suitable for sprinkler irrigation. A final decision on the suitability of any area for irrigation will of course depend on the economics of water supply.

The present land utilisation conforms reasonably well with the above assessment but lack of soil conservation measures, a rather low standard of husbandry on arable land, gross overstocking and lack of management on range areas is resulting in deterioration of the land resources.

4.1.7 Drainage

Masinga catchment lies within three of the Ministry of Water Development’s designated drainage basins; 4A (Sagana), 4B (Upper Tana), 4C (Thika) and 4D (Lower Tana) for purposes of the river gauging network. Their boundaries, drainage pattern and locations of river gauging stations are shown on Figure 4.6. The majority of the Sagana river, which changes to its name to the Tana river downstream of Sagana town, lies in basin 4A and in a small part of basin 4B. The Chania and Thika rivers upstream of Thika town are within basin 4C. The catchment downstream of the Thika–Tana confluence (now submerged by the Masinga reservoir) and upstream of Masinga dam lies in basin 4D.

The dense network of river gauging stations within and near the catchment area is operated and maintained by the Water Department, Ministry of Water Development. At the moment, many of them are closed leaving only a few operational as shown on Figure 4.6.

The catchment contains a large range in elevations, rising from 1,000 m above sea level (a.s.l.) at Masinga dam to peaks of over 2,300 m a.s.l. on the south-western slopes of Mt. Kenya. Masinga reservoir, first filled in 1981, floods 45 km up the Tana river to the Thika-Sagana road bridge. This has altered the flow characteristics of the lower part of the catchment. There are a large number of small storage ponds and reservoirs, particularly around Thika. River abstractions for irrigation and water supplies are increasing and are probably starting to affect low flows significantly in some areas.

Basin water balances show that the Tana basin (4B) contributes 55% of the total inflow to Masinga reservoir (Atkins, 1984). Sagana (4A) and Thika (4C) contribute 24% and 21% respectively. For individual rivers, annual runoff ranges from 100 mm around the dam to 1200 mm on the upper slopes of the Aberdares.

4.2 Database for Masinga Catchment

Both the hillslope profile and watershed versions of the WEPP model requires data on climate, soil, slope, and plant/management to run.
4.2.1 Data Source

Data for this study were obtained from various ministries and departments in Kenya. Ministry of water (M.O.W) provided data on river flow discharges and stages, Tana and Athi Rivers Development Authority (TARDA) provided data on soils, land use, topographic, bathymetry of the reservoir and some information on suspended sediment in rivers draining into Masinga reservoir, Kenya Electricity Generating Company (KenGen) provided data on reservoir levels, and Meteorological department, Dagoretti corner, Nairobi provided climatic data.

4.2.2 Climate Data

The climate data required by the WEPP model includes daily values for precipitation, temperatures, solar radiation, and wind information. In this project, seven weather stations within Masinga catchment were selected for hillslope profile and watershed simulations. For each station, daily rainfall data covering a period of 20 years were collected from the Meteorological department. However, some stations had incomplete data thus making it necessary to fill in missing values. A linear multiple regression model described in section 3.2.5.1 was adopted for this purpose.

4.2.2.1 Application of the Linear Multiple Regression Model

The model was first calibrated using the available rainfall data for the stations with missing values and the rainfall data for the surrounding weather stations. The calibrated model was then used to estimate the values of missing rainfall data.

Saenyi, Wycliffe W.
(a) Filling in the missing rainfall data for station 9036259 for March 1981

Standardised variables were used in this case. The implication is that the intercept is always equal to zero and the coefficients are standardised regression coefficients. After fitting the data to the linear regression model, the standardised regression coefficients were found to be: 0.310, 0.202, 0.399, 0.082, 0.143 and 0.244. The calibrated model is written as:

\[ Y = 0.310X_1 + 0.202X_2 + 0.399X_3 + 0.082X_4 + 0.143X_5 + 0.244X_6 \]  

(4.1)

Where \( Y \) refers to the missing rainfall data for station 9036259. \( X_1, X_2, X_3, X_4, X_5 \) and \( X_6 \) refers to the surrounding available rainfall data for stations 9036307, 9037007, 9037031, 9037222, 9137020 and 9137048 respectively.

To establish how good this equation fit the data, a scatter diagram of the measured and estimated rainfall data for station 9036259 was plotted (Fig. 4.7). The correlation coefficient was found to be 0.55. This rather low value of the correlation coefficient shows that there is poor correlation between the measured and estimated values. However, the function gives an indication on the range and magnitude of expected rainfall values. Hence, it was applied to fill in missing rainfall values.

![Fig. 4.7: Linear correlation graph](image)

Using the calibrated linear multiple regression model, missing rainfall data values for station 9036259 were estimated for March 1981.

(b) Filling in the missing rainfall data for station 9137020 for the period 1/1/98-31/12/00

---

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The above procedure was repeated for station 9137020 and the standardised regression coefficients found to be: 0.091, 0.044, 0.082, 0.059 and 0.287. The calibrated model thus became:

\[ Y = 0.091X_1 + 0.044X_2 + 0.082X_3 + 0.059X_4 + 0.287X_5 \]  \hspace{1cm} (4.2)

Where \( Y \) refers to the missing rainfall data for station 9137020. \( X_1, X_2, X_3, X_4, \) and \( X_5 \) refers to the surrounding available rainfall data for stations 9036259, 9036307, 9037007, 9037031, and 9037222 respectively. The correlation coefficient, \( R \), was found to be 0.63.

Similarly, using the calibrated linear multiple regression model, missing rainfall data values for station 9137020 were estimated for the period between 1/1/1998 and 31/12/2000.

### 4.2.2.2 Application of CLIGEN Model

Climate data availed for use in this study was not complete. Duration of precipitation, time to peak precipitation, peak intensity parameters were missing. It was therefore necessary to estimate these parameters using CLIGEN model.

For a given station, MEAN P, SDEVP, SQEW P, P(W/W), P(W/D), TMAX AV, TMIN AV, SD TMAX, SD TMIN, SOL.RAD, SD SOL, MX .5, DEW PT, and Time pe coefficients were determined from the available weather data for a period of 10 years and entered into the CLIGEN data base. CLIGEN then generated climatic data for 20 years (desired period of simulation). The above procedure was repeated for the other six stations within Masinga catchment. The estimated data values were then used for running the WEPP model. An example of a complete climate file is shown in Table 4.4. Columns 1 to 13 show day, month, year, precipitation, duration of precipitation, time to peak precipitation, peak intensity, maximum temperature, minimum temperature, solar radiation, wind velocity, wind direction and dew-point temperature respectively. It should be noted here that actual observed precipitation data was used (column 4 of climate file) in WEPP simulations.

### Table 4.4: Climate file

<table>
<thead>
<tr>
<th>da mo year</th>
<th>prcp</th>
<th>dur</th>
<th>tp</th>
<th>ip</th>
<th>tmx</th>
<th>tmn</th>
<th>rad</th>
<th>w-vl</th>
<th>w-dir</th>
<th>tdew</th>
</tr>
</thead>
<tbody>
<tr>
<td>(mm)</td>
<td>(h)</td>
<td>(C)</td>
<td>(l/d)</td>
<td>(m/s)</td>
<td>(Deg)</td>
<td>(C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>10.5</td>
<td>2.42</td>
<td>0.02</td>
<td>5.81</td>
<td>19.1</td>
<td>6.4</td>
<td>243.0</td>
<td>7.4</td>
<td>311.0</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.00</td>
<td>0.00</td>
<td>17.1</td>
<td>3.8</td>
<td>425.0</td>
<td>8.2</td>
<td>301.0</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>1</td>
<td>4.9</td>
<td>1.64</td>
<td>0.07</td>
<td>6.93</td>
<td>17.9</td>
<td>4.1</td>
<td>657.0</td>
<td>5.4</td>
</tr>
</tbody>
</table>

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4.2.3 Slope Data

Slope shape is described by using pairs of distance to points from the top of the OFE and the slope at these points. The slope profile must be described to the end of the field, or to a concentrated flow channel, grassed waterway, or terrace. For each of the 31 hillslopes identified from Masinga catchment, slope shape was derived using the slope classification map (Fig. 4.2). A sample slope data file is shown in Figure 4.8.

![Diagram showing slope shape and data](image)

4.2.4 Soil Data

Soil profile data for different locations within Masinga catchment was obtained from Atkins (1984). Information on soil name, texture, albedo, initial saturation level of the soil profile porosity were obtained from Atkins and soil classification map (Fig. 4.4). Baseline interill and rill erodibility parameters, baseline critical shear parameter, effective hydraulic conductivity of surface soil were internally computed by WEPP using the textural properties of the soil. A typical example of a soil file as used in WEPP is shown in Table 4.5.

4.2.5 Plant/Management Data

Plant/management data consists of the following: (a) plant growth parameters, tillage and other implement parameters, initial OFE or channel specific conditions and parameters, contouring parameters, and drainage parameters (b) tillage sequences and other surface-disturbing dated-sequences of implements, and management information. Parameters listed in (a) above were obtained from WEPP user’s manual (Flanagan, 2001) while the dated-
Table 4.5: Soil data as used by WEPP

<table>
<thead>
<tr>
<th>Soil File Name</th>
<th>Soil Texture</th>
<th>Albedo</th>
<th>Initial Sat Level (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F3</td>
<td>C</td>
<td>0.21</td>
<td>75</td>
</tr>
</tbody>
</table>

- **Infiltration Erodibility:** (kg/m²•s²) 
- **Runoff Erodibility:** (s/m) 
- **Critical Shear:** (N/m) 
- **Eff Hydr Conductivity:** (mm/h) 

Table 4.5: Soil data as used by WEPP

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (mm)</th>
<th>Sand (%)</th>
<th>Clay (%)</th>
<th>Organic (%)</th>
<th>CEC (meq/100</th>
<th>Rock (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>250</td>
<td>15.9</td>
<td>18.1</td>
<td>1.700</td>
<td>10.3</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>1000</td>
<td>49.4</td>
<td>18.5</td>
<td>0.680</td>
<td>7.4</td>
<td>37.1</td>
</tr>
<tr>
<td>3</td>
<td>1000</td>
<td>54.2</td>
<td>12.5</td>
<td>0.190</td>
<td>5.0</td>
<td>50.6</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

sequences in (b) were assigned by the author based on his experience of the catchment land use. An example of a plant/management input file is shown in Table 4.6.

Table 4.6: A plant/management file for WEPP

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
</tr>
</thead>
</table>

4.2.6 Channel Input Data

Channel parameters varied from subcatchment to subcatchment and from channel to channel. Peak runoff rate at the channel, sub-watershed or watershed outlet was estimated by a method used in the Chemicals, Runoff, and Erosion from Agricultural Management Systems Saenyi, Wycliffe W.
(CREAMS) model (Knisel, 1980). The method used in the computation of friction slope equated the channel bed slope to friction slope. The channel erodibility and critical shear stress used WEPP estimated values. The bare soil and total Manning roughness coefficients used were from the CREAMS document table II - 28 (Foster et al., 1980).

4.2.7 Watershed Structure Data

The watershed structure data files were created according to how the watershed was divided and what the overland flow directions were. These files provide the water and sediment routing linkages for the WEPP watershed components. They show the sequence of runoff as it flows from one element into the other element until finally it exits at the watershed outlet.

4.3 WEPP Model Application Procedure

4.3.1 Methodological Steps for Running WEPP Hillslope Profile Model

1. Masinga catchment was subdivided into various subcatchments depending on river network and topography of the region. Fourteen subcatchments were created (Fig. 4.9, and Tables 4.7 & 4.8).

2. For each subcatchment, representative hillslopes were created based on the slope, landscape geometry and direction of surface runoff. A total of 31 representative hillslopes were identified from Masinga catchment. Eight of the 31 hillslopes drain into the Thika arm of the reservoir while the remaining 23 drain into the Tana arm (Fig. 4.10). Detailed description of each hillslope is shown in Tables 4.7 & 4.8.

3. Each representative hillslope was assigned the climate data according to which Thiessen polygon it falls in (Fig. 4.9).

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4. Soil, slope and plant/management information were assigned to each representative hillslope using the available catchment soil, slope and land use maps respectively.

5. For each hillslope, WEPP model was ran in hillslope profile mode for a period of 20 years. The results obtained are presented in section 4.4.

### Table 4.7: Description of Tana arm created hillslopes

<table>
<thead>
<tr>
<th>Hillslope</th>
<th>Slope class</th>
<th>% area occupied</th>
<th>Area (km²)</th>
<th>Hillslope Rep. Width (km)</th>
<th>Length of hill - slope (km)</th>
<th>Area of 1 hill - slope (km²)</th>
<th>Number of hill - slopes</th>
<th>Aspect of hill slope (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4AA SUBCATCHMENT (566 km²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4AA1</td>
<td>E</td>
<td>25</td>
<td>142</td>
<td>2.5</td>
<td>0.8</td>
<td>2.0</td>
<td>71</td>
<td>64</td>
</tr>
<tr>
<td>4AA2</td>
<td>D</td>
<td>12.5</td>
<td>70</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>20</td>
<td>64</td>
</tr>
<tr>
<td>4AA3</td>
<td>C</td>
<td>31.25</td>
<td>177</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>50.6</td>
<td>150</td>
</tr>
<tr>
<td>4AA4</td>
<td>B</td>
<td>31.25</td>
<td>177</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>50.6</td>
<td>145</td>
</tr>
</tbody>
</table>

| 4AB SUBCATCHMENT (631 km²) | | | | | | | | |
| 4AB1 | D | 40 | 252 | 3.5 | 1.0 | 3.5 | 72 | 20 |
| 4AB2 | C | 60 | 379 | 3.5 | 1.0 | 3.5 | 108 | 330 |

| 4AC SUBCATCHMENT (457 km²) | | | | | | | | |
| 4AC1 | D (av.) | 100 | 457 | 3.5 | 1.0 | 3.5 | 130.6 | 300 |

| 4AD SUBCATCHMENT (515 km²) | | | | | | | | |
| 4AD1 | D (av.) | 100 | 515 | 3.5 | 1.0 | 3.5 | 147.1 | 280 |

| 4BA SUBCATCHMENT (346 km²) | | | | | | | | |
| 4BA1 | D | 40 | 138 | 3.0 | 0.8 | 2.4 | 57.5 | 340 |
| 4BA2 | C | 60 | 208 | 3.0 | 0.8 | 2.4 | 86.7 | 49 |

| 4BB SUBCATCHMENT (286 km²) | | | | | | | | |
| 4BB1 | D (av.) | 30 | 86 | 3.0 | 1.0 | 3.0 | 28.7 | 15 |
| 4BB2 | C | 70 | 200 | 3.5 | 1.0 | 3.5 | 57.1 | 35 |

| 4BC SUBCATCHMENT (225 km²) | | | | | | | | |
| 4BC1 | B | 60 | 135 | 3.5 | 1.0 | 3.5 | 38.6 | 25 |
| 4BC2 | A2 | 40 | 90 | 3.0 | 0.8 | 2.4 | 37.5 | 20 |

| 4BD SUBCATCHMENT (531 km²) | | | | | | | | |
| 4BD1 | D | 55 | 292 | 3.5 | 0.8 | 2.8 | 104.3 | 275 |
| 4BD2 | C | 45 | 239 | 3.5 | 0.8 | 2.8 | 85.4 | 320 |

| 4BE SUBCATCHMENT (564 km²) | | | | | | | | |
| 4BE1 | D | 35 | 198 | 3.0 | 1.0 | 3.0 | 66 | 340 |
| 4BE2 | B (av.) | 30 | 169 | 3.0 | 1.0 | 3.0 | 56.3 | 230 |
| 4BE3 | A1 | 35 | 197 | 3.0 | 1.0 | 3.0 | 65.7 | 190 |

| 4BF SUBCATCHMENT (505 km²) | | | | | | | | |
| 4BF1 | B | 50 | 252.5 | 3.5 | 1.0 | 3.5 | 72.2 | 220 |
| 4BF2 | A1 | 50 | 252.5 | 3.5 | 1.0 | 3.5 | 72.2 | 200 |

| 4DD SUBCATCHMENT (273 km²) | | | | | | | | |
| 4DD1 | A1 | 50 | 136.5 | 3.0 | 1.0 | 3.0 | 45.5 | 210 |
| 4DD2 | A2 | 50 | 136.5 | 3.0 | 1.0 | 3.0 | 45.5 | 190 |
### Table 4.8: Description of Thika arm created hillslopes

<table>
<thead>
<tr>
<th>Hillslope</th>
<th>Slope class</th>
<th>% area occupied</th>
<th>Area (km²)</th>
<th>Hillslope Rep. Class</th>
<th>Length of hill - slope (km)</th>
<th>Area of 1 hill - slope (km²)</th>
<th>Number of hill - slopes</th>
<th>Aspect of hill slope (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4CB1</td>
<td>D</td>
<td>50</td>
<td>461</td>
<td>3.0</td>
<td>1.0</td>
<td>3.0</td>
<td>153.7</td>
<td>270</td>
</tr>
<tr>
<td>4CB2</td>
<td>C</td>
<td>30</td>
<td>277</td>
<td>3.0</td>
<td>1.0</td>
<td>3.0</td>
<td>92.3</td>
<td>253</td>
</tr>
<tr>
<td>4CB3</td>
<td>A2</td>
<td>20</td>
<td>184</td>
<td>3.0</td>
<td>1.0</td>
<td>3.0</td>
<td>61.3</td>
<td>195</td>
</tr>
<tr>
<td>4CC1</td>
<td>B (av.)</td>
<td>20</td>
<td>248.2</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>70.9</td>
<td>195</td>
</tr>
<tr>
<td>4CC2</td>
<td>A2</td>
<td>40</td>
<td>496.4</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>141.8</td>
<td>180</td>
</tr>
<tr>
<td>4CC3</td>
<td>A1</td>
<td>40</td>
<td>496.4</td>
<td>3.5</td>
<td>1.0</td>
<td>3.5</td>
<td>141.8</td>
<td>160</td>
</tr>
<tr>
<td>4BG1</td>
<td>B (av.)</td>
<td>40</td>
<td>108.8</td>
<td>2.8</td>
<td>1.0</td>
<td>2.8</td>
<td>38.9</td>
<td>150</td>
</tr>
<tr>
<td>4BG2</td>
<td>A2 (av.)</td>
<td>60</td>
<td>163.2</td>
<td>2.8</td>
<td>1.0</td>
<td>2.8</td>
<td>58.3</td>
<td>260</td>
</tr>
</tbody>
</table>

#### Schematic of Masinga catchment

Fig. 4.10: Schematic of Masinga catchment showing hillslopes and channels draining into the reservoir. Note here that the shape of the reservoir is is exaggerated. Figure serves only to show how Masinga catchment was split into representative hillslopes.

### 4.3.2 Methodological Steps for Running WEPP Watershed Model

1. To apply WEPP watershed version, all the hillslopes were imported into the watershed structure with their associated climatic, soil, slope and plant/management data. This was done for both Tana and Thika branches of Masinga catchment (Fig. 4.10).
2. Climate, soil, slope and plant/management information were assigned to each channel in the watershed structures using the catchment Thiessen polygon, soil, slope and land use maps respectively. Figures 4.11 and 4.12 shows watershed structures for Tana and Thika branches respectively. Because of the many number of hillslopes for Tana branch, its structure was split into three substructures to make it easier to run the WEPP model.

![Diagram of Tana branch substructures](a)

![Diagram of Tana branch substructures](b)

![Diagram of Tana branch substructures](c)

Fig. 4.11: Shows the structure of Tana branch split into substructures
(a) Upper Tana substructure
(b) Mid Tana substructure
(c) Lower Tana substructure

3. For each branch, WEPP model was run in watershed mode for a period of 8 years (from 1981 to 1988). Then the program was run for 20 years (from 1981 to 2000). The results obtained were processed and are presented in section 4.4.
4.4 WEPP Results and Discussion

4.4.1 Hillslope Profile Results and Discussion

WEPP can be instructed to output a very large data file which contains daily values of about 100 different parameters. In this project, the parameters of interest included daily values of sediment yield leaving profile, runoff, evapotranspiration, soil evaporation and total soil water. Because of the large amount of data, only one subcatchment (4AA) was selected for presentation. Graphical representation of simulated results from the 4AA subcatchment hillslopes for the year 1981 are shown in Figures 4.13 – 4.16. Daily rainfall values and temperatures on these hillslopes are also presented.

Total soil water simulated by WEPP model exhibited a two-trough shape with the maximum being about 650 mm (for 4AA2 hillslope). These shape is more pronounced for the 4AA3 and 4AA4 hillslopes. This can be attributed to the fact that some substantial amount of water from the root zone is lost through deep percolation. The fact that soils on these hillslopes contain high percentage of sand explains why there is more deep percolation from the two hillslopes as compared to the other hillslopes (4AA1 and 4AA2) in the same sub-catchment. The highest amount of total soil water occurs for hillslope 4AA3 due to the fact that the top soil layer is very deep (600 mm) thus retaining more water in the root zone. Hillslopes 4AA1 and 4AA2 exhibit a small variation in total soil water, with their trough not well pronounced. The reason for this trend could be that these hillslopes, being in forested area, are covered with vegetation throughout the year, hence keeping the total soil water more or less constant.
Fig. 4.13: 4AA1 hillslope model results for the year 1981
Fig. 4.14: 4AA2 hillslope model results for the year 1981
Fig. 4.15: 4AA3 hillslope model results for the year 1981
Fig. 4.16: 4AA4 hillslope model results for the year 1981
4.4.1.1 Validation of WEPP Hillslope Profile Results

To validate the results obtained from WEPP simulations, two parameters were selected for this purpose. The runoff computed by WEPP model is compared to runoff obtained from separation of hydrograph components (section 3.2.7). This was done for Tana-Sagana sub-catchment and for the year 1981. The second parameter, evapotranspiration, was selected because of the availability of monthly field computed values for Thika (Shaw, 1989). WEPP model-simulated evapotranspiration was compared with field computed values for Thika sub-catchment for the year 1981.

Daily model-simulated runoff is compared with runoff obtained from separation of hydrograph components using least square analysis (Fig. 4.17). The calculated coefficient of determination is 0.97 and is significant at the 0.05 probability level. The intercept and the slope of the regression equation between daily model-simulated runoff and runoff from hydrograph analysis are not different from zero and unit, respectively, at 0.05 probability level, which indicates statistically a good agreement between the WEPP model runoff calculations and hydrograph derived runoff. This comparison shows that the model estimates are representative of runoff from Masinga catchment.

Model simulated evapotranspiration and average field computed potential evapotranspiration for Thika area are shown in Fig. 4.18. The average field computed evapotranspiration values are obtained from Gethumbwini weather station at Thika. The calculated standard error between model simulated and average field computed evapotranspiration is 7.86 mm. Good agreement between simulated and field computed evapotranspiration indicates that the model is capable of predicting potential evapotranspiration component of the WEPP model with reasonable accuracy.

\[ y = 0.8705x + 0.0849 \]
\[ R^2 = 0.9675 \]

![Fig. 4.17: Least square analysis between simulation runoff and estimated runoff from separation of hydrograph components for 1981](image)

*Saenyi, Wycliffe W.*
Based on the above validation, it is assumed that the model would also perform well for other parameters such as sediment yield leaving profile, soil evaporation, total soil water, etc.

![Comparison of model-simulated and field computed potential evapotranspiration of Thika sub-catchment](image)

**Fig. 4.18: Comparison of model-simulated and field computed potential evapotranspiration of Thika sub-catchment**

### 4.4.1.2 Sediment Yield Results and Discussion

Sediment yield estimates based on WEPP hillslope profile for all the representative hillslopes are tabulated in Tables 4.9 & 4.10 for Tana and Thika branches, respectively. A Hillslope on mountainous area (4AD1) having high slopes and high rainfall yielded the highest sediment (302 t/ha/year). Hillslopes 4CB1, 4CB2 & 4BE1 on gentle slopes with high rainfall yield relatively high sediment but lower than that from 4AD1. Quite high sediment yield was obtained from hillslope 4CB3 because of being on high slopes, despite receiving relatively low rainfall. However, hillslopes on gentle slopes and having moderate or low rainfall yielded low sediment. The average sediment yield estimate from WEPP model for Masinga catchment is 11.4 t/ha/year. The average for Tana branch is 9.9 t/ha/year while for Thika branch is 15.3 t/ha/year. These results are compared with estimates obtained by other researchers on this catchment.

Estimates of suspended sediment yields in the upper Tana basin produced by Ongwenyi (1978) are 630 t/km²/year (6.3 t/ha/year) and 310 t/km²/year (3.1 t/ha/year) for Tana and Thiba rivers, respectively. These values are comparable to estimates obtained from WEPP model (Tables 4.9 & 4.10). Except for the extreme value of 302 t/ha/year from hillslope 4AD1, other values obtained from WEPP are comparable to estimates by Ongwenyi. However, it should be noted that the results of WEPP model are based on land use data of 1985 only and therefore the likelihood of non-stationary sediment response produced by land use change from 1985 to 2000 was not accounted for. This could be the major source of error in WEPP results obtained in this study.

*Saenyi, Wycliffe W.*
Table 4.9: Results of WEPP hillslope profile simulations for Tana branch

<table>
<thead>
<tr>
<th>Hillslope and weather station</th>
<th>Soil type and coverage (%)</th>
<th>Land use</th>
<th>Average annual Precipitation (mm)</th>
<th>Average annual runoff (mm)</th>
<th>Average Annual soil loss (Kg/m²)</th>
<th>Average Annual sed. yield (t/ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4AA SUBCATCHMENT (566 km^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4AA1 Kerugoya</td>
<td>M9 (49), M2 (49), R1(2)</td>
<td>forest</td>
<td>1280.1</td>
<td>133.8</td>
<td>0.617</td>
<td>3.316</td>
</tr>
<tr>
<td>4AA2 Kerugoya</td>
<td>R1(95), R2 (5)</td>
<td>forest, tea, grass, maize</td>
<td>1280.1</td>
<td>84.39</td>
<td>38.78</td>
<td>18.63</td>
</tr>
<tr>
<td>4AA3 Kerugoya</td>
<td>R1(32), R2(65), R3(3)</td>
<td>tea, grass, maize</td>
<td>1280.1</td>
<td>69.87</td>
<td>65.35</td>
<td>13.27</td>
</tr>
<tr>
<td>4AA4 Kerugoya</td>
<td>R1(32), R2(60), R3(8)</td>
<td>tea, maize, grass</td>
<td>1280.1</td>
<td>69.27</td>
<td>30.61</td>
<td>7.656</td>
</tr>
<tr>
<td>4AB SUBCATCHMENT (631 km^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4AB1 Gatere</td>
<td>M9(50), M2(50)</td>
<td>forest, tea, grass</td>
<td>2503.25</td>
<td>106.8</td>
<td>44.24</td>
<td>14.94</td>
</tr>
<tr>
<td>4AB2 Kerugoya</td>
<td>R3(70), R2(22), R1(8)</td>
<td>tea, grass, coffee, maize+beans</td>
<td>1280.01</td>
<td>110.2</td>
<td>21.57</td>
<td>21.61</td>
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<td>4AC SUBCATCHMENT (457 km^2)</td>
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</tr>
<tr>
<td>4AC1 Gatere</td>
<td>R1(50), M2(50)</td>
<td>forest, tea, grass, maize</td>
<td>2503.25</td>
<td>90.64</td>
<td>182.5</td>
<td>15.16</td>
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<tr>
<td>4AD SUBCATCHMENT (515 km^2)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4BA SUBCATCHMENT (346 km^2)</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4BA1 MurangaD</td>
<td>H12(40), R2(60)</td>
<td>rangeland, coffee, maize+beans</td>
<td>1093.51</td>
<td>35.57</td>
<td>6.734</td>
<td>5.366</td>
</tr>
<tr>
<td>4BA2 MurangaD</td>
<td>H12(40), R2(60)</td>
<td>rangeland, coffee, maize+beans</td>
<td>1093.51</td>
<td>36.11</td>
<td>6.339</td>
<td>4.059</td>
</tr>
<tr>
<td>4BB SUBCATCHMENT (286 km^2)</td>
<td></td>
<td></td>
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<tr>
<td>4BB1 Kerugoya</td>
<td>R1(100)</td>
<td>forest</td>
<td>1280.01</td>
<td>136.6</td>
<td>3.552</td>
<td>13.20</td>
</tr>
<tr>
<td>4BB2 Kerugoya</td>
<td>R1(40), R2(60)</td>
<td>forest, tea, grass, maize</td>
<td>1280.01</td>
<td>91.32</td>
<td>32.20</td>
<td>12.93</td>
</tr>
<tr>
<td>4BC SUBCATCHMENT (225 km^2)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4BC1 Kerugoya</td>
<td>R2(100)</td>
<td>tea, grass, coffee, maize+beans</td>
<td>1280.01</td>
<td>86.60</td>
<td>10.16</td>
<td>9.619</td>
</tr>
<tr>
<td>4BC2 MurangaD</td>
<td>R3(70), L2(30)</td>
<td>maize, coffee, grass</td>
<td>1093.51</td>
<td>38.07</td>
<td>9.642</td>
<td>1.367</td>
</tr>
<tr>
<td>4BD SUBCATCHMENT (531 km^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4BD1 MurangaS</td>
<td>M2(70), R1(20), R2(10)</td>
<td>tea, grass, coffee, maize+beans</td>
<td>1316.81</td>
<td>2.29</td>
<td>0.288</td>
<td>0.451</td>
</tr>
<tr>
<td>4BD2 MurangaD</td>
<td>R2(80), R3(10), H12(10)</td>
<td>grass, coffee, maize+beans+ pigeon peas</td>
<td>1093.51</td>
<td>15.00</td>
<td>21.56</td>
<td>1.715</td>
</tr>
<tr>
<td>4BE SUBCATCHMENT (564 km^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4BE1 Gatere</td>
<td>M2(45), R1(25), R2(30)</td>
<td>tea, grass, coffee, maize+beans</td>
<td>2503.25</td>
<td>232.8</td>
<td>33.10</td>
<td>36.28</td>
</tr>
<tr>
<td>4BE2 MurangaS</td>
<td>R2(60), R3(40)</td>
<td>coffee, maize+beans</td>
<td>1316.81</td>
<td>1.09</td>
<td>1.147</td>
<td>0.119</td>
</tr>
<tr>
<td>4BE3 MurangaD</td>
<td>L2(40), L1(25), Um20(35)</td>
<td>maize+ pigeon peas, rangeland, cotton</td>
<td>1093.51</td>
<td>4.97</td>
<td>3.104</td>
<td>0.037</td>
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<tr>
<td>4BF SUBCATCHMENT (505 km^2)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4BF1 Thika</td>
<td>R2(40), R3(40), F16(20)</td>
<td>coffee, grass, maize+beans+ pineapples, cabbage</td>
<td>895.09</td>
<td>12.29</td>
<td>10.12</td>
<td>30.73</td>
</tr>
<tr>
<td>4BF2 Mitubiri</td>
<td>R3(40), F16(20), H15(20), Um20(20)</td>
<td>sisal, grass, maize+ pigeon peas, rangeland</td>
<td>887.83</td>
<td>4.17</td>
<td>0.812</td>
<td>0.021</td>
</tr>
<tr>
<td>4DD SUBCATCHMENT (273 km^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Sediment yield estimates based on samples collected in 1974-77 in wider Masinga basin was reported by Starman (1980). These are 7.9, 6.1, 20.4, 91.7, 85.6, 9.9, and 19.2 t/ha/year for Thego, Upper Sagana, Lower Sagana, Mathi, Maragua, Chania, and Thika rivers, respectively. No information on calculation procedures is provided. In view of the limited number of samples used to derive sediment yield estimates and the short period of record, the accuracy of these estimates must be very suspect.

Generally, information on sediment transport in Tana and Thika rivers are lacking. Therefore, sediment yields have to be estimated by procedures of doubtful reliability. It is even fortunate that some suspended sediment sampling has been undertaken within the study area during the past 50 years. However, most of the available data relate to infrequent sampling and to restricted periods of time and cannot be used to develop accurate assessments of sediment yield. Problems of paucity of data are compounded by the very great variability of runoff totals during the study period and by the likelihood of non-stationary sediment response produced by land use change, which make it difficult to extrapolate short-term measurements. Existing estimates of sediment loads for rivers in the study area are largely based on these unreliable data and are therefore extremely tentative. The various debates and inconsistencies (Atkins, 1984) that have been developed concerning the precise magnitude of these loads are a reflection of this uncertainty. Regular reservoir sedimentation surveys may, however, assist in improving such estimates.

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In order to consider the likely magnitude of sediment input to Masinga reservoir and the spatial pattern of sediment yield within the study area it will be necessary to review the various sources of data and the reliability of the associated estimates.

Since land use data was only available for one year, mapping land use on a yearly basis will assist to improve the accuracy of WEPP results. Climatic data parameters estimated by CLIGEN model might be another cause of uncertainty in WEPP results. To alleviate this problem, it is imperative that all these parameters are measured.

4.4.2 WEPP Watershed Results and Discussion

As in WEPP hillslope model, the output files from WEPP watershed model simulations have many parameters. For this study, the parameters of interest included event-based values of sediment yield at the watershed outlet, runoff volume and surface runoff. A typical sample of the output file for Tana branch is shown in Table 4.11.

<table>
<thead>
<tr>
<th>Date</th>
<th>Precipitation depth (mm)</th>
<th>Runoff volume (m³)</th>
<th>Runoff (m³/s)</th>
<th>Sediment yield (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/5/1981</td>
<td>35.8</td>
<td>13.79</td>
<td>0.00219</td>
<td>175.1</td>
</tr>
<tr>
<td>30/5/1981</td>
<td>55.7</td>
<td>16798.20</td>
<td>1.60828</td>
<td>344999.7</td>
</tr>
<tr>
<td>17/7/1981</td>
<td>29.8</td>
<td>0.15</td>
<td>0.00003</td>
<td>0.0</td>
</tr>
<tr>
<td>20/7/1981</td>
<td>25.5</td>
<td>0.33</td>
<td>0.00006</td>
<td>0.0</td>
</tr>
<tr>
<td>4/8/1981</td>
<td>24.4</td>
<td>0.77</td>
<td>0.00012</td>
<td>0.0</td>
</tr>
<tr>
<td>12/10/1981</td>
<td>47.2</td>
<td>52.49</td>
<td>0.00748</td>
<td>775.9</td>
</tr>
<tr>
<td>25/10/1981</td>
<td>20.8</td>
<td>0.22</td>
<td>0.00004</td>
<td>0.0</td>
</tr>
<tr>
<td>15/3/1982</td>
<td>32.5</td>
<td>809.49</td>
<td>0.10102</td>
<td>12087.0</td>
</tr>
<tr>
<td>13/6/1982</td>
<td>38.5</td>
<td>189.92</td>
<td>0.02260</td>
<td>2689.6</td>
</tr>
</tbody>
</table>

4.4.2.1 Validation of WEPP watershed Results

Two approaches were adopted for validation of WEPP watershed results: (1) Validation using volumetric analysis and (2) validation by water balance method. In the first approach, the volume of sediment generated by WEPP model for the period 1981 – 1988 was determined. Then, the volume of sediments deposited in the reservoir was computed using the hydro-survey data of 1988 & initial reservoir bathymetric data of 1981. The two values were compared to establish whether they are of the same order of magnitude.

As the second approach to check the WEPP watershed results, a water balance was done for runoff computed from Tana subcatchment and Thika subcatchment by WEPP model and stream flow for Tana and Thika rivers, respectively.

4.4.2.1.1 Validation by Volumetric Analysis of Sedimentation Process

(a) Volumetric computation using the survey data of 1981 & 1988

The volume of sediments deposited in the reservoir between 1981 and 1988 was computed using the method of weighted averages. Within 8 years of operation (1981 to 1988), 99.73x10⁶ m³ of sediment was deposited in Masinga reservoir. This translates to about 6.39%
reduction in reservoir storage capacity. Expressed on a yearly basis, this amounts to about 0.8% reduction in reservoir volume per year.

(b) Computation of sediment input into Masinga reservoir using WEPP watershed results for the period between 1981 & 1988

Daily sediment yield output values were summed up for the period 1981 to 1988. The value obtained (5.944x10^8 kg) is the yield from an area of 25.1 km^2. This value was scaled to a figure representative of the whole catchment area (7335 km^2), assuming sediment yield is proportional to the contributing area. This computation gave a value of 1.737x10^11 kg as total sediment yield from Masinga catchment for the period 1981 to 1988. Converted to volume using bulky density of 1380 kg/m^3, this sediment mass occupies 125.85 million cubic metres. If all these sediments enter the reservoir and are all trapped there (100% trap efficiency), the storage capacity of the reservoir would be reduced by the same volume (125.85x10^6 m^3). Assuming 10% is deposited in streams, trapped by plant roots, etc. and does not reach the reservoir, the amount entering the reservoir would be 113.26 million cubic metres. This translates to about 7.2% reduction in storage volume of the reservoir for the period between 1981 and 1988 or 0.9% reduction per year.

Comparing the volume reductions due sediment deposits computed in (a) and (b) above, it can be seen that the two values are in close agreement; 6.4% reduction using survey data (initial rangeline of 1981 and 1988 hydro-survey) and 7.2% reduction using WEPP model. This implies that WEPP model can simulate sediment yield into the reservoir with reasonable accuracy. The rate of siltation in Masinga reservoir for the period 1981 to 1988 was found to be 0.8 – 0.9% per year.

4.4.2.1.2 Validation by Water Balance Method

The water-balance method is a measurement of continuity of flow of water. This should hold true for any time interval and should apply to any drainage basin. In this project, observed monthly runoff volumes in rivers (Tana & Thika) were computed from their corresponding monthly discharge hydrographs for the period between 1981 and 1985 using the trapezoidal formula. Values of runoff volumes for all events in each month were summed up to obtain monthly runoff volumes from WEPP model for the period between 1981 and 1985. A water-balance between the model estimated monthly runoff volume and observed river flow volume was done for both Tana and Thika (Figures 4.19 & 4.20). Areal rainfall for each subcatchment is also plotted alongside the water-balance graph to show the watershed response to rainfall.

It can be envisaged from the graphs that WEPP simulates peak runoff volumes fairly well. Time to peak for both model-estimated and observed runoff volume coincide. However, peak values for model-estimated runoff volumes are lower than their corresponding observed values. This could be attributed to the fact that in WEPP model, percolation below the root zone is considered lost from the WEPP water balance. The program does not simulate low flow events reasonably well. It can be seen from the graphs that WEPP depicts low flows as zeros or values close to zero. Another anomaly is that the model estimated runoff volumes for two events; November 1984 and February 1985 are more or equal to the observed runoff volumes. In reality, this is impossible. The plausible explanation for this anomaly could be that the observed runoff volumes are erroneous.

In general, however, the program successfully models the peaks and satisfactorily shows how the subcatchments respond to rainfall. Therefore, WEPP was applied to Masinga catchment...
to model sediment yield from the catchment, sediment input into Masinga reservoir, and hence, suspended sediment load in rivers Tana & Thika.

![Graph showing water-balance (runoff) for Tana](image1)

![Graph showing monthly areal rainfall for Tana](image2)

**Fig. 4.19**: (a) Water-balance (runoff) for Tana, (b) Monthly areal rainfall for Tana
Fig. 4.20: (a) Water-balance (runoff) for Thika, (b) Monthly areal rainfall for Thika

4.4.2.2 Prediction of Volume of Sediment Input into the Reservoir in 2000

From the watershed model output, sediment yield values for all the events between 1981 & 2000 were summed up to obtain the total mass (kg) of sediments entering the reservoir. This value was converted to volume by multiplying it with the bulky density of 1380 kg. A total volume of sediments of 175.7 million cubic metres was obtained from Masinga catchment between 1981 & 2000. Similarly, assuming 10% is deposited in streams, trapped by plant roots, etc. and does not reach the reservoir, the amount entering the reservoir was found to be...
158.13 million cubic metres. Since the reservoir storage capacity is 1560 million cubic metres, reduction in storage volume of 10.13% was determined using WEPP model for the period between 1981 and 2000.

### 4.4.2.3 Sediment Routing and Suspended Load Computation

Sediment routing through channels either cause deposition or erosion in channel segments of the watershed. A given rainfall event can caused deposition in channel segments if the peak value of sediment yield at the watershed outlet is lower than the peak value of sediment yield from any one of the hillslopes contributing sediment to the channel segments. During such an event, the sum totals of sediment yield from individual hillslopes is more than the sediment yield at watershed outlet. This implies that some of the sediment delivered into the channels from hillslopes during the rainfall event is deposited there. On the other hand, erosion occurred in some channel elements during some rainfall events. The sediment routing results showed that sediment yield at the watershed outlet is more than the sum total of sediment yield from individual hillslopes implying that erosion took place in the channel segments.

From the watershed model output, suspended load at the two inlets of the reservoir were computed and part of the results are tabulated (Table 4.12). These values are generally lower than the field measured ones by Pacini (1994). However, they are comparable since they are in the same order of magnitude. There is high fluctuation in sediment load with maximum being about 200 mg/l for Thika river and about 160 mg/l for Tana river. Minimum value for both rivers is zero as shown in Figures 4.21 and 4.22. The mean values of suspended load for Thika and Tana are 45.2 mg/l and 23.7 mg/l, respectively. It should be noted that only rainfall events with runoff volume more than 0.005 m$^3$ were listed and that higher suspended load is always associated with heavy rainfall storms. When no rainfall occurs, suspended load is taken to be zero, which is not true in nature.

<table>
<thead>
<tr>
<th>Date</th>
<th>Runoff volume (m$^3$)</th>
<th>Sediment yield (kg)</th>
<th>Suspended load (mg/l) (WEPP)</th>
<th>Suspended load (mg/l) (Observed) After Pacini (1994)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/5/1981</td>
<td>13.79</td>
<td>175.1</td>
<td>12.698</td>
<td>34.3</td>
</tr>
<tr>
<td>30/5/1981</td>
<td>16798.20</td>
<td>344999.7</td>
<td>20.538</td>
<td>81.0</td>
</tr>
<tr>
<td>17/7/1981</td>
<td>0.15</td>
<td>0.0</td>
<td>0.0</td>
<td>13.8</td>
</tr>
<tr>
<td>20/7/1981</td>
<td>0.33</td>
<td>0.0</td>
<td>0.0</td>
<td>6.7</td>
</tr>
<tr>
<td>4/8/1981</td>
<td>0.77</td>
<td>0.0</td>
<td>0.0</td>
<td>11.2</td>
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<td>12/10/1981</td>
<td>52.49</td>
<td>775.9</td>
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<tr>
<td>25/10/1981</td>
<td>0.22</td>
<td>0.0</td>
<td>0.0</td>
<td>15.2</td>
</tr>
<tr>
<td>15/3/1982</td>
<td>809.49</td>
<td>12087.0</td>
<td>14.932</td>
<td>37.2</td>
</tr>
<tr>
<td>13/6/1982</td>
<td>189.92</td>
<td>2689.6</td>
<td>14.162</td>
<td>45.3</td>
</tr>
</tbody>
</table>

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4.4.2.4 Sediment Rating Curves

From the WEPP watershed model output, sediment rating curves were developed for both Tana and Thika rivers. First, sediment concentration in the flow (kg/m³) was computed from runoff volume (m³) and sediment yield (kg). Sediment discharge (kg/s) was then computed by multiplying sediment concentration with water discharge (m³/s). Plotting the logarithm of
sediment discharge (metric tonnes per day) versus logarithm of water discharge (m$^3$/s) gave the following relationships:

\[ y = 0.8215x - 0.8651 \]
\[ R^2 = 0.9175 \]

Fig. 4.23: Sediment rating curve (Tana)

\[ y = 0.7139x - 0.0594 \]
\[ R^2 = 0.9458 \]

Fig. 4.24: Sediment rating curve (Thika)

for Tana;

\[ Q_s = 0.421Q^{0.822}, \quad R^2 = 0.917, \quad n = 616 \]  \hspace{1cm} (4.3)

and for Thika;

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\[ Q_s = 1.062Q^{0.714}, \quad R^2 = 0.946, \quad n = 204 \]  

where \( Q_s \) is the sediment discharge in metric tonnes per day, \( Q \) is the water discharge in \( m^3/s \), \( R^2 \) is the coefficient of determination, and \( n \) is the number of sediment yield events.

The above two functions were used as input into GSTARS 2.1 sedimentation model (Section 5.2.3). However, it should be noted that these functions are only approximations, and often very poor ones (Yang and Simoes, 2000). It must also be recognised that any estimate of suspended sediment load derived using a rating curve is likely to involve a very considerable margin of error (Walling, 1977; Walling and Webb, 1981). Since there was no reliable data on suspended sediment load in river Tana and Thika, it was worthy developing sediment rating curves from WEPP model results, then use these curves as input into GSTARS 2.1 sedimentation model.

### 4.4.3 Recommendations for Improving WEPP Simulated Results

Only little information on climate, soils, slope and land use of Masinga catchment was available for WEPP simulations. Because of this data scarcity, it is plausible that model results are not completely accurate. The following recommendations could help to improve on the simulated results:

(i) Installation of more climatological stations and rain-gauges to improve the existing network.

(ii) Soil loss measurement should be encouraged. A simple programme of soil loss monitoring is recommended to obtain quantitative information on soil losses from agricultural land. A simple practical programme which is likely to produce valid results is recommended in preference to a more complex programme which is likely to pose difficulties in provision of trained manpower and financial resources. A series of fixed-boundary micro-plots and field plots are recommended for soil loss monitoring, combined with sediment sampling. Measured results will provide a strong basis for comparison with model simulated results.

(iii) Rainfall data should be measured on daily basis to avoid gaps of missing information. In this project, missing rainfall data was filled in using multiple linear regression model, which gave only approximate values. These approximations are therefore embedded in WEPP results. To improve on WEPP simulations, it will be necessary to have reliable measured rainfall data devoid of gaps of missing values.

(iv) It is necessary to install recording rain-gauges in all weather stations. Duration of precipitation, time to peak precipitation, peak intensity parameters can then be determined using data from recording rain-gauges. In this project, the above rainfall parameters were missing and had to be computed using the CLIGEN model. The uncertainty in this computed parameters could be a major source of error in WEPP model simulations.

(v) Baseline interill and rill erodibility parameters, baseline critical shear parameter, effective hydraulic conductivity of surface soil were internally computed by WEPP using the textural properties of the soil. Measurement of these parameters in situ would give more realistic and representative site specific values.

(vi) Land use data for each year from 1981 to 2000 should be used in WEPP simulations. In this study, only land use data for 1985 was available to run with WEPP model. Hence, the transient sediment response produced by land use changes was not accounted for in the WEPP simulations. This could be another source of uncertainty in WEPP results.
CHAPTER FIVE

RESERVOIR SEDIMENTATION MODELLING

5.1 Description of Masinga Reservoir

5.1.1 Introduction

Masinga dam was built during 1980 – 1981 to create a storage reservoir upstream of the Seven Forks Development Scheme. Its main function was to regulate the flow of the Tana River into the other four reservoirs (Fig. 5.1). Benefits in the river regulation are readily translated into an increased output of hydroelectric power through higher flexibility of reservoir operation and increased life spans of the reservoirs below Masinga dam. The Seven Forks Development Scheme owes its name to seven tributaries that join the Tana river below Kamburu reservoir. At these points, new dams are planned, as at Mutonga and Grand Falls. The Scheme provides over 60% of electricity consumed in the country today. In some months this proportion is even higher (Mr. Muriithi, Kenya Power Generating Co. Ltd., personal communication). Table 5.1 shows the storage capacity and the theoretical generating power at each step in the cascade.

<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Year commissioned</th>
<th>Storage volume (x10^6 m³)</th>
<th>Generating Capacity (MW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masinga</td>
<td>1981</td>
<td>1560</td>
<td>40</td>
</tr>
<tr>
<td>Kamburu</td>
<td>1975</td>
<td>146</td>
<td>84</td>
</tr>
<tr>
<td>Gitaru</td>
<td>1978</td>
<td>20</td>
<td>145</td>
</tr>
<tr>
<td>Kindaruma</td>
<td>1968</td>
<td>18</td>
<td>140</td>
</tr>
<tr>
<td>Kiambere</td>
<td>1988</td>
<td>585</td>
<td>140</td>
</tr>
</tbody>
</table>

Within the Seven Forks cascade which provides the location for the Kamburu, Gitaru and Kindaruma hydro-electric schemes, the river falls through a vertical height of some 270 m in a distance of 30 km so that channel slopes are of the order of 90 m/km. Above the Seven Forks cascade the river profile is considerably flatter and over a distance of some 50 km above the head of the Kamburu reservoir river channel slopes are of the order of only 1 m/km. This reach extends upstream to more steeply falling sections in the vicinity of Tana Power Station, where the Sagana, Maragua and Mathioya rivers join to form the mainstream Tana river. Thus, the reservoir basin is admirably located to provide regulation for the Seven Forks hydro-electric schemes because the relatively low channel slopes ensured an economic relationship was obtained between capacity and height of dam while at the same time the reservoir is able to regulate the flow of all major tributaries above the Kamburu reservoir.

During the planning of Masinga dam and of the other reservoirs, little effort was put into the assessment of possible impacts of the developments on the reservoir sedimentation process and the Tana river downstream. The aim of this section is to describe some physical characteristics of Masinga reservoir which have to be taken into consideration in the study of sedimentation.

5.1.2 Topography of Reservoir Basin

The basin lies within a plateau of gently rolling topography broken by occasional rounded hills such as Rongori and Twoinoini hills where more resistant intrusions project above the

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general level. Within this plateau, the Tana and its tributary, the Thika have incised relatively deep and fairly narrow valleys of low longitudinal slope. Thus the lake impounded by the dam form a long narrow sinuous strip of water some 45 km long with an average width of approximately 2 km and an extensive arm on the southern side protruding along the lower reaches of the Thika river valley. The lake is therefore well contained within surrounding high ground except in the vicinity of the dam where there are two low saddles close to the full supply level of 1056.5 m. These saddles are located within 1 km of the left abutment of the main embankment, and their crests are 1056 m and 1052 m respectively. The first of these, nearest the dam incorporates the emergency spillway facilities while the second is closed by a secondary earthfill embankment some 500 m in length with a maximum height of 8 m.

Fig. 5.1: Location of Masinga catchment and reservoir

5.1.3 Geology and Landform of Reservoir Basin

Masinga and the other reservoirs of the Seven Forks Development Scheme are underlain by the rocks of the basement complex. These are composed of schists and gneisses of precambrian age which underwent several stages of metamorphosis. Their characteristics are homogeneous throughout the Tana basin. They are highly impermeable and represent sites suitable for the location of reservoirs.

In the Masinga area the lack of water storage capacity is associated with a very fast runoff response to precipitation. There are no permanent streams other than those which descend from the volcanic hills in the west and north of the Upper Tana River Catchment (UTRC).

Groundwater resources are limited to cracks within the metamorphic basement and scarce aquifers formed in alluvial deposits. Traditional watering points for the local population are
temporary wells dug in dry riverbeds and, of course, the Tana river. For this reason, this part of the UTRC had until recently very low population density and low level of development.

Around the reservoir, the soils are shallow and well drained. A longitudinal transect through the UTRC would show a trend from clay soils in the west to progressively coarser textures and finally sandy soils in the east. The main soil types are ferralsols, acrisols and vertisols. Surface capping of soils is common in particular with the vertisols and is accompanied by high soil erodibility. Deep gullies and evidence of rill erosion even under conditions of gentle slope (<5%) is a testimony of low infiltration capacity. Because of the prevailing low slope, scarce rainfall and high evaporation, it is believed that erosion from these areas does not contribute significant amounts of sediment to the reservoir; though it is difficult to establish quantitative estimates.

5.1.4 Climate and Land use

Air temperatures, rainfall, solar radiation, wind run and pan evaporation data were obtained from the Meteorological Department for the Masinga Farm Meteorological Station situated at the dam site. Summary figures for the one-year period from April 1992 to March 1993 are presented in Table 5.2.

Wind roses for the area of Masinga reservoir indicate that the prevailing wind blows in the South easterly direction and the component of this wind along the axis of the reservoir basin is in the upstream direction. Winds having a downstream component have very low frequency of occurrence. The maximum wind velocity recorded in the vicinity of the reservoir is less than 10.8 m/s. Thus wave action and wind set-up at the dam is small.

Table 5.2: Summary of meteorological data for Masinga Farm (station no. 9037222).

<table>
<thead>
<tr>
<th>After Pacini (1994).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean daily air temperature</td>
</tr>
<tr>
<td>Mean daily wind run</td>
</tr>
<tr>
<td>Mean daily solar radiation</td>
</tr>
<tr>
<td>Mean daily pan evaporation</td>
</tr>
<tr>
<td>Total yearly pan evaporation</td>
</tr>
<tr>
<td>Total yearly precipitation</td>
</tr>
<tr>
<td>*mean dew point temperature</td>
</tr>
<tr>
<td>* calculated from altitude according to the relationship (Republic of Kenya, 1967, cited by Pacini, 1994)</td>
</tr>
</tbody>
</table>

\[
\text{Temperature at dew point (F) = 69.7-2.5*Altitude(ft*1000)}
\]

In the region close to the reservoir, rains are scarce and unpredictable. The long-term mean annual rainfall is around 700 mm with the short rains (October to December) bringing usually the largest amount of precipitations (Atkins Land and Water Management, 1984). The long-term mean potential evapotranspiration is between 2000 and 2100 mm and determines a moisture deficit over most of the year. Consequently, agricultural output is low and inconsistent. According to the relationship between altitude and evaporation derived by Dunne (1978) for the Tana river basin, Penman open-water evaporation at Masinga should be around 2000 mm/annum. This figure is close to 90% of the total yearly pan evaporation at Masinga Farm in 1992 to 1993. Calculations of open water evaporation from the meteorological data reported above lead to lower values of daily Penman evaporation. This could be due to an underestimate of wind run. The values for wind run reported by the Meteorology department for station 9037222 are about half of the values reported for other meteorological stations in the region (Republic of Kenya, 1967, cited by Pacini, 1994).
The commonest crops grown in Masinga area is the drought-resistant Katumani Maize interplanted with pigeon peas and beans. Sisal plantations were once popular but have been progressively abandoned because of the fall in the demand for natural fibres. More details about agricultural land use in the area are presented by Mutisya (1994, cited by Pacini, 1994). Livestock breeding (goats and zebu cows) is popular and supported by the relative proximity of Nairobi and the densely populated centres of Thika and the lower slopes of the Nyandarua range.

In contrast to the western part of the UTRC where livestock is stable-fed and kept in low numbers, in the Masinga area herds are moving every day to new grazing grounds. Livestock density is beyond the carrying capacity of the system and is considered to represent a major contributing factor of the enhancement of soil erosion through the reduction of soil cover by overgrazing and trampling (Atkins, 1984).

5.1.5 Morphology of Masinga Reservoir

Baseline physical data for Masinga are presented in Table 5.3. The reservoir is narrow and dendritic with high shoreline development. The mean depth is only 13.8 m for a maximum of 47 m indicating that extended shallow areas are likely to dry out as water fluctuates in the reservoir. The bathymetric map of Masinga reservoir (Fig. 5.2) was traced by linear projection of 86 echo-sounding transects conducted in 1988 (Data provided by the Tana and Athi Rivers Development Authority, TARDA). The approximate contours of the reservoir (0 m level) were derived comparing the transects with altitudinal contour isopleths on 1:50,000 Ordinance Survey maps and field notes provided by the TARDA technical staff. The bathymetric map is instructive since it shows that the area at depth greater than 40 m includes mainly the bed of former river and slightly wider area close to the dam wall. A large part of the reservoir is characterized by lower depths, particularly in the upper sections. It is here that higher water level fluctuations are expected to occur. Of the two arms that compose the western part of Masinga reservoir, the Tana arm carries a large amount of the flows. During the flood period, water circulates from the Tana arm towards Thika river.

Table 5.3: Physical limnology of Masinga reservoir

<table>
<thead>
<tr>
<th>Location</th>
<th>Latitude</th>
<th>Longitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>0°47’ - 1°00’S</td>
<td>37°16’ - 37°36’E</td>
</tr>
<tr>
<td>Altitude (a.s.l.)</td>
<td>1056.5</td>
<td></td>
</tr>
<tr>
<td>Drainage area (km²)</td>
<td>7335</td>
<td></td>
</tr>
<tr>
<td>Surface area (km²)</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>Drainage/surface ratio</td>
<td>58.7</td>
<td></td>
</tr>
<tr>
<td>Volume (10⁶m³)</td>
<td>1480</td>
<td></td>
</tr>
<tr>
<td>Reservoir length (km)</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>Mean depth (m)</td>
<td>13.8</td>
<td></td>
</tr>
<tr>
<td>Maximum depth (m)</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>Reservoir shoreline (km)</td>
<td>285</td>
<td></td>
</tr>
<tr>
<td>Shoreline development ratio</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td>Hydraulic residence time</td>
<td>3 months - 2 years</td>
<td></td>
</tr>
</tbody>
</table>

The upstream section of the reservoir has a very narrow channel and is well protected from the action of winds. The reservoir offers a reduced fetch to the dominant winds that tend to follow the monsoon circulation.

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Morphometric conditions offered by the indented shoreline and the position of the major axis of the reservoir perpendicular to the winds are conducive to the development of a highly stable water column and single yearly mixing events (Tundisi, 1988, cited by Pacini, 1994).

Morphometric characteristics such as these were the main factors isolated by Henry and Tundisi (1988, cited by Pacini, 1994) to explain differences in stability among reservoirs in Brazil.

5.1.6 Water Level Fluctuations and the Draw-down Zone

The reservoir is characterized by water level fluctuations of high amplitude. Periodically a vast surface area is dried out and flooded determining frequent horizontal water movements between the pelagic zone and littoral areas. Changes in water level since filling are illustrated in Fig. 5.8. On the 1st of April 1988, water level in the reservoir was 13.8 m below the fill line. Consequently, more than half of the surface area was uncovered. During 1992, a periodic drought exposed 50% of the reservoir surface and brought the water surface 9.5 m below level. Figure 5.3 illustrates the surface and volume capacity curves of the reservoir that could be used to calculate the draw-down area. The capacity curve was obtained from TARDA records and presents the theoretical volume of the reservoir at the time of filling. Total volume was 1560x10⁶ m³ at filling, while today it is alleged that it is about 1480x10⁶ m³ (TARDA staff, personal communication). The depth-surface area curve was obtained by planimetric integration of the depth contours of the bathymetric map. High water level fluctuations as these are common in reservoirs situated in semi-arid areas (Thornton and Rast, 1993, cited by Pacini, 1994).

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The frequent drying of shallow areas has adverse consequences for the littoral vegetation and fish. A draw-down depth in excess of 3 m causes ecological instability and disrupts breeding sites for fish (Bernacek 1984, cited by Pacini, 1994). This effect is of particular significance for the Tilapia fishery (Oreochromis, Tilapia and Sarotherodon) as these fishes deposit their eggs in nests built in littoral areas. The extent of drawdown and its negative effects on the fishery were underestimated at the time of construction (Pacini et al. 1994).

Fig. 5.3: Surface and capacity curves for Masinga reservoir. Superimposed are the low water levels of April 1988 and 1992 showing the respective decrease in surface area. The thick line represents the capacity curve, while the thin line relates to the surface area. After Pacini (1994).

5.1.7 Longitudinal Zonation in Reservoirs

Reservoirs represent an intermediate condition between lotic and lentic environments. From the river inlets to the dam wall they exhibit continuous gradients of depth, water quality, light penetration and biomass. Three separate zones have been identified (Thornton 1990, cited by Pacini, 1994). The riverine zone is the closest to the limnological conditions characteristic of rivers. It is characterized by shallow depths, high turbidity and an unstable water column. The continuous riverine input and the lack of stratification ensure mixing and oxygenated conditions throughout the year. The riverine zone is characterized by high nutrient levels and high concentrations of suspended solids. It is in this zone that the coarser sediments settle out. The transition zone follows along the gradient with a progressive deepening and widening of the reservoir bed. The gradual decrease of water currents facilitates the deposition of fine sediments. The deepest water are found in the third zone which is called lacustrine to underline its similarity to the limnological characteristics of lakes. This zone tends to be more oligotrophic, with good light penetration and stable stratification. Figure 5.4 illustrates the establishment of longitudinal gradients in reservoirs.

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Living organisms distribute themselves naturally along the physico-chemical gradients formed by the succession of the three zones to develop under the most suitable conditions. Reservoirs are also of great interest to limnologists since they allow the simultaneous observation of different equilibria and the interplay of different controlling factors that in lakes are realized on a yearly time scale.

Fig. 5.4: Conceptual model of reservoir zonation (adapted from Thornton, 1990, cited by Pacini, 1994).

The extension of the 3 zones illustrated in Fig. 5.4 is dependent upon the residence time which is typically highly variable in reservoirs situated in semi-arid areas. During flood period, the riverine zone will tend to penetrate further into the reservoir displacing the zonation on Fig. 5.4 towards the right. In Masinga, the reservoir zonation model has been useful for the interpretation of longitudinal gradients of oxygen, temperature and chemical determinants (Pacini, 1994). The change in the measured parameters was gradual and did not allow the net distinction of the three zones.

5.1.8 Stratification and Thermal Gradients in Tropical Lakes

The thermocline was classically defined as the thermal boundary between two distinct water masses (Birge 1897, cited by Pacini, 1994) and conventionally identified with occurrence of temperature gradient of 1°C/m.

Circulation in lakes is a consequence of a balance between the stirring action of the winds and the stabilizing action of solar radiation. Other processes such as internal waves, heat exchange with the bottom sediments and the influence of inflows are of minor impact unless particular situations occur. The stability of the separation of the water column is proportional to the thermal gradient. Temporary separation of the water surface from the layers immediately underneath can be produced under hot and calm conditions usually during the early afternoon. Circulation is then restored at the onset of night-time cooling. In temperate regions, it is commonly observed that a gradient of 1°C/m is necessary to preserve the separation of water masses beyond average diurnal fluctuations of temperature and wind stress. In the tropics,
stable thermoclines can persist over extended periods supported by thermal gradients much inferior to this.

The density of water is maximum at 4°C but decreases rapidly with increasing temperatures. At the high temperatures encountered in tropical lakes, a small thermal gradient is capable of producing a significant density gradient and therefore an effective separation between non-mixing water masses. In South Africa, Duncan (1964, cited by Pacini, 1994) stated that a gradient of 0.1°C/m was sufficient for the establishment of a stable thermocline. Balon chose 0.2°C as the critical gradient for the identification of the thermocline in Kariba reservoir (Pacini, 1994). Such values are close to the detection limit of ordinary thermometers and electrodes employed during usual field studies.

In stratified lakes of temperate regions thermal gradients are often much greater than 1°C/m. This is related to the occurrence of extreme air temperatures during the yearly cycle. The hypolimnion in such lakes tends to remain all year round at a temperature which is close to the mean water temperature at the time of circulation. In tropical lakes instead, because of the minor oscillations in temperature during the year, the difference between temperatures at the bottom and at the top of a strongly stratified water column can be very small. Masinga reservoir, being a tropical lake, is expected to have very small thermal gradients as well.

5.2 Database for the Masinga Reservoir

In this section, the data used to run the numerical simulations on reservoir sedimentation is described. This comprised of the bathymetric data, discharge into the reservoir for both Tana and Thika arms, dam levels, water temperatures, and sediment data. As is often true in many water and sediment transport modelling, it is not always possible to have all the required data, and so, one must manage with what is available. In this study, discharge data, water levels and temperatures were readily available. However, very little information was available on sediment load in the rivers entering the reservoir. In this latter case, a suspended sediment generator was developed to compute the sediment load in the rivers discharging into the reservoir as described in chapter 4. Some information on sediments was obtained from literature for similar studies while in some cases, it was entirely assumed.

5.2.1 Bathymetric Data

The data corresponds to the stretch between Tana bridge and Masinga dam as well as the Thika arm as shown in Figure 5.2. Actual survey data measured on the reservoir basin was availed for use in this study. These consisted of two sets: 1981 survey data, measured before the reservoir was filled up and the 1988 echo-sounding survey conducted when the reservoir was full. For both sets of data, measurements were done for a total of 86 cross sections representing the main reach of approximately 45 km in length and a tributary (Thika arm) of about 15.1 km. This information was furnished by Tana and Athi Rivers Development Authority (TARDA), an organisation charged with the responsibility of managing Tana and Athi river basins. Detailed bathymetric data is tabulated in appendix A.

5.2.2 Hydrological Data

The hydrology data consists of daily flows and water temperatures at the upstream end of the reach, and of daily reservoir levels at the downstream end. The input of these data into the model was done by use of the stage-discharge table at control station.
Head loss due to friction on the channel bed was computed by the average slope method while Manning’s equation was chosen for roughness calculations. The reservoir channel was divided into 3 stream tubes to represent the main channel, left, and right flood plains. No stream power minimisation control was requested in this study.

5.2.2.1 Discharge

Daily discharge data for rivers entering Masinga reservoir was obtained from the Hydrology Section of the Ministry of Water Development (MOWD) which is responsible for collecting and evaluating data on rivers in Kenya. Their database systems in the headquarter of MOWD was established in 1983. Data was availed for the period from 1980 – 2000. The data was interpolated linearly in cases of missing values within 7 consecutive days. According to the MOWD National Water Master Plan (1992), the combined mean monthly discharge at the inlets of the reservoir is 119.5 m$^3$/s. The network of gauging stations within and near the study area is shown in Figure 4.6 while graphs showing daily discharge for Tana and Thika arms are shown in Figures 5.6 and 5.7, respectively.

5.2.2.2 Reservoir Levels

Daily reservoir levels were collected from the Kenya Power Generating Company (KENGEN) for the period 1981 – 2000. This information is depicted in Figure 5.8 for the period 1982 to mid 1993. The mean value for reservoir levels is about 1054 m a.s.l. Initially, the minimum water level required for power generation to commence was 1033 m a.s.l. However, this value has been raised to 1035 m a.s.l. to reduce the problem of cavitation on runner blades and guide vanes of the turbines (Personal communication with the plant chief engineer, Mr. Kimani, 2001). In general, the reservoir levels fluctuate between the highest value of 1057.56 m a.s.l. and the lowest value of 1042.73 m a.s.l. However, the water level dropped to the lowest value ever observed, 1018.68 m a.s.l. during the severe drought experienced in Masinga Catchment in 1999/2000.

![Tana Discharge Hydrograph](image)

**Fig. 5.6: Discharge Hydrograph for Tana (1980-1997)**

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Fig. 5.7: Discharge hydrograph for Thika (1980 – 1994)

Fig. 5.8: Reservoir levels (1982 – 1993)

5.2.2.3 Water Temperatures

Daily water temperatures were collected from TARDA for the period 1981 – 2000. Mean water temperatures for reservoir is about 18°C, but ranges from the lowest value of about 13°C to the highest value of about 23°C. Figure 5.9 illustrates the variation of daily water temperatures for the year 1981.
Fig. 5.9: Water temperatures (1981)

5.2.3 Sediment Data

The incoming sediment discharge is specified as a function of water discharge. The functions were derived from the results of the WEPP model watershed simulations and were obtained using the least squares method. For each branch, a sediment rating curve was obtained as described in Section 4.4.2.4.

The bed material and incoming suspended load have a relatively high percentage of very fine particles, in silt and clay range. The incoming sediment distribution is given as a function of water discharge. In this project, incoming sediment distribution was varied such that the simulated cross-sections approximately fit the corresponding observed ones.

The bed material distribution over the simulated reach is known at specific locations and is interpolated for sections lying between those locations. The locations where the bed material distribution is specified do not need to coincide with the cross sections specified for the reach. For example, in this simulation, there are 50 cross sections, but only 10 sets of gradation curves are used. The range of bed material size distribution is shown in Figures 5.10 to 5.19.

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Fig. 5.12: Gradation curve at station 35 km upstream of dam

Fig. 5.13: Gradation curve at station 30 km upstream of dam

Fig. 5.14: Gradation curve at station 5 km upstream of dam

Fig. 5.15: Gradation curve at station 45 km upstream of dam

Fig. 5.16: Gradation curve at station 20 km upstream of dam

Fig. 5.17: Gradation curve at station 25 km upstream of dam
Nine sediment size fractions were specified for this study. Table 5.4 shows their lower and upper boundaries with corresponding dry specific weights. The angle of repose of bank materials was taken as 32° for both Tana and Thika arms.

### Table 5.4: Sediment size fractions and corresponding dry specific weights

<table>
<thead>
<tr>
<th>Number of size fractions</th>
<th>Sediment Size Fractions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>Lower bound (mm)</td>
</tr>
<tr>
<td>1</td>
<td>2.56-4</td>
</tr>
<tr>
<td>2</td>
<td>0.0040</td>
</tr>
<tr>
<td>3</td>
<td>0.0080</td>
</tr>
<tr>
<td>4</td>
<td>0.0160</td>
</tr>
<tr>
<td>5</td>
<td>0.0310</td>
</tr>
<tr>
<td>6</td>
<td>0.0820</td>
</tr>
<tr>
<td>7</td>
<td>0.1250</td>
</tr>
<tr>
<td>8</td>
<td>0.2500</td>
</tr>
<tr>
<td>9</td>
<td>0.5000</td>
</tr>
<tr>
<td>10</td>
<td>2.0000</td>
</tr>
</tbody>
</table>

Reservoir sedimentation processes are essentially non-equilibrium, hence the need to specify the non-equilibrium parameters. Since modelling was done for a reach with mixed characteristics (river-like upstream and reservoir-like downstream), different values for the recovery factors were defined. The reach is predominantly in depositional mode. Therefore only the recovery factor for deposition is important in this case and varied between 0.25 and 0.001. Recovery factor for scour was, however, taken to be 1.

Since most of the sediments entering the reservoir are fine grained silt and clay, the cohesive sediment part of GSTARS 2.1 was used. Yang’s 1996 modified formula was selected for computing sediment transport capacity while some of the cohesive sediment transport parameters were obtained from literature for a similar case (Yang and Simoes, 2000). Under normal circumstances, these parameters, which characterise the particles with a diameter smaller than 62.5 μm, should be determined in situ or by laboratory tests. They are highly dependent on the local conditions and may vary widely from case to case, always requiring field verification. In this study, it was not possible to determine these parameters in the field.
Hence, the characteristics of the cohesive sediments were assumed as follows: the threshold value for the shear stress above which there is mass erosion is 11.491 Pa; the erosion rate of cohesive sediment when the bed shear stress equals 11.491 Pa is 1.197 Pa/hour; finally, the last entry indicates the threshold for the percentage of clay, in the bed, above which the erosion rates of the other particle sizes become limited to the erosion rate of clay. This value was assumed to be 10%. Other cohesive sediment transport parameters were used as calibration parameters (section 5.3).

The thickness of the active layer was taken to be 1 m while the percentage of wash load present in the flow was 0.5%.

5.2.4 Harmonising WEPP Results and GSTARS 2.1 Model Input Data

To harmonise WEPP predicted sediment inflow data and GSTARS 2.1 model input requirement, sediment input into the reservoir was computed using the WEPP derived sediment rating curves (section 4.4.2.4) and the result compared with amount required by GSTARS 2.1 to replicate the measured reservoir cross-sections. Indeed, it was found out that Tana branch delivered only a third of the amount required by GSTARS 2.1 to simulate the measured cross-sections. This underestimation on the part of WEPP model could be attributed to several reasons: (1) the fact that bank erosion is not considered by WEPP could have contributed to lower WEPP estimated sediment yield, (2) It was assumed that runoff, hence, sediment enters the reservoir via two inlets only; Tana and Thika. There are, however, other subsidiary tributaries whose contribution of sediment to the reservoir could be high enough not to be ignored. In this study, sediment contribution by subsidiary tributaries was ignored, apparently leading to lower sediment input into the reservoir, (3) Sediment yield from the sides of the reservoir was not taken into account. This could be another reason for a lower estimate of sediment yield into the reservoir. It was therefore necessary to multiply the amount computed using the sediment rating curve for Tana branch by three in order to harmonise the WEPP output and GSTARS 2.1 required input.

WEPP model also underestimated the amount required by GSTARS 2.1 to replicate measured reservoir cross-sections for Thika arm. In this case, WEPP delivered only about 0.77 of the amount required by GSTARS 2.1. To harmonise WEPP output and GSTARS 2.1 required input, WEPP output was multiplied by a factor of 1.3.

5.3 Application of GSTARS 2.1 to Masinga Reservoir

In this section, model calibration was carried out to verify whether GSTARS 2.1 can really and accurately reproduce reservoir deposition over 8 years of operation. Secondly, prediction of reservoir morphological change over 20 years of operation (1981 – 2000) was done. That notwithstanding, computation of the volume of sediment deposits was carried out to establish reduction in reservoir capacity.

5.3.1 Calibration of GSTARS 2.1 Model

The model was calibrated for the period between 1981 and 1988. This period was selected because of the availability of hydrographic survey data measured in 1981 and 1988 (TARDA). The cross-sections measured in these two years were used for the calibration of GSTARS 2.1 model. The cross-sections of 1981 were used for the initial reservoir geometry in the calibration. The whole computational domain of 45 km from Tana Bridge to Masinga dam and Thika arm of 15.1 km was divided into 50 cross sections and 3 stream tubes.
5.3.1.1 Calibration Parameters

The following parameters were adjusted to fit the simulated cross-sections to the measured cross-sectional data:

- Shear threshold for deposition of clay and silt (STDEP), used to determine the initial condition for deposition,
- Shear threshold for particle erosion of clay and silt (STMERO),
- Slope of the erosion rate curve for mass erosion (ERMASS),
- Size gradation distribution of the incoming sediment

Some parameters were obtained from literature for similar conditions or approximated. A comprehensive description of all the other model input parameters is done in section 5.2.

To calibrate GSTARS 2.1, first, size gradation distribution of incoming sediment was varied and predicted reservoir cross-sections obtained. The predicted cross-sections and thalweg were then compared with measured ones to establish whether the two sets match. The reason for varying size gradation distribution of incoming sediment was to distribute sediment deposits longitudinally in such a way that predicted and measured cross-sections and thalweg approximately match. Normally, the bed load and coarser fraction of the suspended load are deposited at the mouth of the reservoir, while fine sediments with lower settling velocities are transported and deposited deeper into the reservoir or near the dam wall. Table 5.5 shows the size gradation distribution of the incoming sediment as a function of discharge which gave the best approximation of the measured cross-sections and thalweg.

Table 5.5: Percentages of incoming sediment as a function of discharge

<table>
<thead>
<tr>
<th>Size class symbol</th>
<th>Discharge (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.6</td>
</tr>
<tr>
<td>clay</td>
<td>0.098</td>
</tr>
<tr>
<td>vfslt</td>
<td>0.091</td>
</tr>
<tr>
<td>fslt</td>
<td>0.27</td>
</tr>
<tr>
<td>mslt</td>
<td>0.206</td>
</tr>
<tr>
<td>cslt</td>
<td>0.046</td>
</tr>
<tr>
<td>vfsnd</td>
<td>0.202</td>
</tr>
<tr>
<td>Fsnd</td>
<td>0.045</td>
</tr>
<tr>
<td>msnd</td>
<td>0.041</td>
</tr>
<tr>
<td>Other</td>
<td>0.001</td>
</tr>
</tbody>
</table>

Legend

<table>
<thead>
<tr>
<th>Size class symbol</th>
<th>Description</th>
<th>size range (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay</td>
<td>clay</td>
<td>&lt;0.004</td>
</tr>
<tr>
<td>vfslt</td>
<td>very fine silt</td>
<td>0.016 – 0.004</td>
</tr>
<tr>
<td>mslt</td>
<td>medium silt</td>
<td>0.031 – 0.016</td>
</tr>
<tr>
<td>cslt</td>
<td>coarse silt</td>
<td>0.062 – 0.031</td>
</tr>
<tr>
<td>vfsnd</td>
<td>very fine sand</td>
<td>0.125 – 0.062</td>
</tr>
<tr>
<td>Fsnd</td>
<td>fine sand</td>
<td>0.25 – 0.125</td>
</tr>
<tr>
<td>msnd</td>
<td>medium sand</td>
<td>0.5 – 0.25</td>
</tr>
</tbody>
</table>

The second step in the calibration process was to vary the sedimentation and erosion parameters, each time comparing the predicted and measured cross-sections and thalweg.
According to the model, an erosion parameter, STMERO, of 0.479 Pa, ERMMASS of 0.63 per hour, and a sedimentation parameter, STDEP, of 0.101 Pa yielded the best results.

5.3.1.2 Results of Calibration and Discussion

The computed thalweg were compared with the observed ones as shown in Figures 5.20 and 5.21, for Thika and Upper Tana arms, respectively. It can be seen that the predicted and observed thalweg for Thika arm agreed well for many stations. For Tana arm, predicted and observed thalweg deviate slightly from each other near the mouth of the reservoir but they agree quite well for many stations downstream. These results indicate that GSTARS 2.1 can simulate longitudinal reservoir bed evolution with reasonable accuracy. That notwithstanding, the results imply that the model can describe selective fractional deposition and erosion processes within the reservoir under unsteady flow conditions fairly well.
Sediment Management in Masinga Reservoir, Kenya – Reservoir Sedimentation Modelling

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wall. The other cross-sections are given in Appendix B. Comparisons of the simulated and observed cross-sections was carried out (Figures 5.22 – 5.28). For most of these stations, the simulated cross-sections are fairly in good agreement with the measured ones except for stations near the confluence and dam wall where deposition seems much more than expected.

![Fig. 5.22: Cross-section 75](image1)

There is a bigger deviation between the measured and observed cross-sections for the sections around the confluence (Figures 5.24, 5.25, and 5.27). This could be attributed to the formation of eddy and/or secondary currents at the confluence of the two arms (Thika and Tana). Since GSTARS 2.1 is based on stream tube concept, the presence of secondary and eddy currents phenomena makes the model to fail. It can also be seen that there is big deviation between model estimated and observed data for cross sections near the dam wall. This could be attributed to the backwater flow and formation of eddy currents making the program to fail. Otherwise, for other cross-sections, the observed and simulated bed elevation changes are in good agreement.

![Fig. 5.23: Cross-section 60](image2)

Model-estimated cross sections do not correspond favourably with observed cross sections for deeper parts of the channels (Section 69, Appendix B). In such cases, GSTARS 2.1 seems to underestimate the amount of deposited sediments within these deeper parts. It does not show

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the accumulation of sediments in these narrow deeper parts as observed in the field. The reason for this discrepancy could be the failure of GSTARS 2.1 to model local flow conditions existing in these areas. The program however, seems to simulate deposition on flood plains quite well as evidenced in the coincidence of observed and model-estimated data points on flood plains for most cross-sections.

**Fig. 5.24: Cross-section 25**

A survey on all plotted cross sections reveals that GSTARS 2.1 model poorly approximates measured data for complex cross sections (Section 81, Appendix B). The reason for this could be the failure of GSTARS 2.1 to split such complex cross-sections into stream-tubes of equal conveyance. It performs well for simple cross sections whose geometrical shapes are adequately defined.

**Fig. 5.25: Cross-section 26**

Generally, sedimentation occurred at the mouths of the reservoir (Tana and Thika), at the confluence of the two arms and near the dam wall. There is little sedimentation for the rest of the other stations. Observations also shows that most of the material was deposited in the old river channel and only little on the terraces (flood plains).
Fig. 5.26: Cross-section 42

Fig. 5.27: Cross-section 23

Fig. 5.28: Cross-section 2
In general, the 8-year simulation indicated that the model can compute changes in reservoir bed elevation, net erosion and sedimentation with reasonable accuracy. These results proved that GSTARS 2.1 model can be used to predict, with confidence, future changes in reservoir bed elevations due to erosion and deposition.

5.3.2 Prediction Results and Discussion

The focus of the sedimentation modelling is to see how the reservoir bed elevations change after 20 years of reservoir operation. Various procedures used to quantify the amount and pattern of sediment deposition and erosion within the reservoir do exist (Morris & Fan, 1998). In the present study, a quasi-1D numerical model, GSTARS 2.1 was applied to model sediment accumulation in Masinga reservoir. After calibration of the model using measured data, predictive computations were carried out over a period of 20 years to assess possible changes in reservoir bed elevations during the two decades. The simulated changes on some selected cross sections due to deposition are given in Figures 5.29 to 5.35. From the plotted cross sections, it was found that most deposition occurred along the thalweg with deposition depths typically in the range of 1.5 m to 3 m. River terraces submerged by the reservoir are elevated 2 to 4.8 m above the channel thalweg. They showed a consistent increase in sediment accumulation of 0.1 to 0.5 m. The trend of sedimentation is similar to that observed during calibration simulations. Most of the sediment delivered to the reservoir was deposited along the main channel and little on the reservoir terraces. The model predicted more sedimentation at the mouth of the reservoir, at the confluence of the two arms (Tana and Thika), and near the dam wall.
Section 60

Fig. 5.30: Cross-section 60

Section 25

Fig. 5.31: Cross-section 25

Section 26

Fig. 5.32: Cross-section 26
Fig. 5.33: Cross-section 42

Fig. 5.34: Cross-section 23

Fig. 5.35: Cross-section 2
5.3.2.1 Computation of Volume of Sediment Deposits in the Reservoir in 2000

The computation of the volume of sediment deposited in Masinga reservoir in 2000 was based on new cross-sectional data obtained from the results of GSTARS 2.1 model simulation. Volume calculations were done using the method of weighted averages. During the predictive period, from 1981 to 2000, the total computed deposited sediment volume in the reservoir is 213.76 million m$^3$. This translates to 13.7% reduction in reservoir volume. This value is higher than the one computed using WEPP model (175.7 million m$^3$) for the same period. This discrepancy can be explained in terms of the difference in dry bulky density and wet bulky density of sediments as used in volume computations. Because of the water content, wet bulky density is higher than dry bulky density making the volume computed with GSTARS 2.1 higher than WEPP computed value.

A wet bulky density correction value should be applied to harmonise WEPP and GSTARS 2.1 calculated values. Since wet bulky density of deposited sediment varies with depth (Nachteebel et al., 1992) and time, the rate of sedimentation should be expressed in terms of thickness per unit time. However, the porosity, the water content, and thus the original thickness of the sediment layer will decrease with increasing depth of burial because of compaction. It is therefore necessary also to express the rate of sedimentation of solids in weight per unit of time, especially when comparing sediment accumulation in water reservoir with sediment yield from its drainage area. However, this comparison is only possible if sediment cores from selected locations within the reservoir during different seasons and different years are collected. This information was not collected for this study because of lack of resources.

5.3.2.2 Effect of Widening Cross-section on Sedimentation Process

It was realised that some parts of the reservoir are not sufficiently covered by the existing cross sections. An attempt was made to investigate the effects of a wider cross section in between two narrow cross sections on the sedimentation process. Conceptually, when flow enters enlarged cross-sections, its velocity reduces. Hence, sediment transport capacity of flow decreases and sediments settle out at the reservoir bed. This phenomenon was demonstrated by introducing imaginary cross-sections between sections 31 & 33 (Fig. 5.36), 36 & 37, 71 & 72, 63 & 62, 56 & 55, 54 & 53, 51 & 50, 50 & 25, and 23 & 26.

Fig. 5.36: Imaginary section between two stations
GSTARS 2.1 was then run and the simulation results obtained for two imaginary sections are shown in Figures 5.37 and 5.38. It was established that more deposition occurred on the imaginary wider cross section than on either of the two adjacent narrower cross-sections for both 1988 and 2000 simulations. The simulation results from imaginary cross-sections were then incorporated in computation of volume of sediment deposits in the reservoir. It was found out that the volume of deposition increased by about 0.6%. This results elucidates the fact that modelling results can be improved by having cross-sections at shorter intervals so that wider and narrower sections of the reservoir geometry are adequately defined.

![Fig. 5.37: Imaginary cross-section between sections 31 & 33](image1)

![Fig. 5.38: Imaginary cross-section between sections 36 & 37](image2)

5.3.2.3 Masinga Reservoir Sedimentation Patterns

Sediment management in a reservoir requires an analysis to determine how sediment deposits are distributed in the reservoir. This is a difficult aspect of reservoir sedimentation because of the complex interaction between hydraulics of flow, reservoir operating policy, inflow sediment load, and changes in the reservoir bed elevation. The traditional approach to analyzing the distribution of deposits has relied on empirical methods, all of which require a great deal of simplification from the actual physical problem.

Depth data for 1981, 1988 and 2000 were used to produce bathymetric maps of Masinga reservoir. The source of information about the bottom configuration of the reservoir for 1981 was obtained from initial bathymetric survey before impoundment. The 1988 hydrographic survey provided the bottom configuration for 1988, whereas, the 2000 depth data were obtained from GSTARS 2.1 simulated results. The processing and visualisation of these depth data sets were performed using a two-dimensional BOSS SMS computer-based model (1996) to generate 2-D depth contour maps. This software package produces a topographic surface through unbiased interpolation of elevation (depth) between randomly or regularly spaced
data points. The resulting 2-D pictures are compared to establish the trend and pattern of sedimentation between 1981 & 1988, and between 1988 and 2000. Changes in the spatial pattern of bathymetry indicate the specific locations where sediment has been deposited.

The 1981 bathymetric map of Masinga reservoir (Fig. 5.39) shows that depths are greatest in the area near the dam wall and along the old river channels. With zero depth at the 1056.5 m reference level, maximum depths, just over 48.8 m, exist along the old river channel in the area near the dam wall as shown on Figure 5.39. The two arms of the reservoir generally has depths less than 35 m. Tana and Thika arms have maximum depths of about 34 m and 31 m, respectively.

In general, a similar geographic pattern of reservoir bathymetry is evident from the map produced from the 1988 hydrographic survey data (Fig. 5.40) when compared to the 1981 bathymetry. Shallow areas are evident in the two arms of the reservoir upstream of the confluence whereas portions of the old river channel, main arm of the reservoir, and the area near the dam wall have depths greater than 12 m. However, some reduction in depths occurred in the reservoir between 1981 and 1988. Areas with high reduction in depths corresponds to locations with greater sediment deposition. Substantial deposition occurred at the mouths of the reservoir forming deltas. Between 1981 and 1988, maximum depths along the old river channel in the area near the dam wall dropped from about 48.8 m to 48.4 m. Maximum depths for Tana and Thika arms dropped to about 33.3 m and 30.9 m, respectively. Greater reduction in depth occurred in area around the dam wall and at the mouths of the reservoir implying that there is more deposition in these region. Generally, little deposition can be noticed on reservoir terraces.
Fig. 5.40: Bathymetry of Masinga reservoir in 1988

Preliminary bathymetric map generated for 2000 (Fig. 5.41) showed a similar trend in the sedimentation pattern as in 1988. The deltas at the mouths of the reservoir have widened. There is further drop in depths mainly in the old river channel. The area near the dam wall recorded the highest drop in depth, from 48.4 m in 1988 to about 47.7 m in 2000. From Fig. 5.41, it can be seen that sediments have filled up the deeper parts of the reservoir flattening the middle old river channel section.

More details on changes of the reservoir bed elevations and hence its morphology can be seen by zooming in at specific locations. Figures 5.42 to 5.45 show changes in the reservoir bathymetry with time at specific areas. It can be concluded from these pictures that there is progressive sedimentation in the reservoir from 1981 to 1988 and from 1988 to 2000. Figures 5.42 and 5.43 typically shows the development of deltas at the mouths of Masinga reservoir. From site observation, vegetation has covered the exposed delta deposits and thus attracted additional deposition until the deltas now take on characteristics of floodplains. It can also be seen (Fig. 5.45) that deposition has occurred near the dam wall filling the once deepest area of the reservoir. Fig. 5.44 shows the progress of deposition in a region around the Island (the recreational resort area of Masinga reservoir). It can be seen that the island is expanding towards the north.
Fig. 5.41: Bathymetry of Masinga reservoir in 2000

Fig. 5.42: Bathymetry Masinga reservoir at the Mouth of Thika arm (a) in 1981, (b) in 1988, and (c) in 2000
Analysis of the sedimentation pattern indicate that siltation has occurred primarily in the deeper main part of the reservoir, at the mouths, at the confluence of the two arms, and downstream sections of the impoundment. From the maps, zones of greatest deposition are coincident with west-east oriented sections of the main old Tana River channel. A wind-and density-driven undercurrent (south-west to north-east monsoon oriented) is thought to be responsible for limiting sedimentation in the south-west sections of the main old Tana River channel (Pacini, 1994). Maximum wind speeds between 5.7 and 10.8 m/s have been recorded at the station near the reservoir site (Atkins, 1984). The upstream section (Tana and Thika arms) of the reservoir has a very narrow channel and is well protected from the action of the winds.

Sediment deposition is occurring throughout the reservoir, but wave action is responsible for agitating and re-suspending sediment within the shallower Thika arm of the reservoir (Atkins, 1984). In addition to the normal sorting that occurs when a sediment-laden stream enters a water body, wind-induced re-suspension of finer-sized particles in the shallower Thika arm of Masinga reservoir is thought to be responsible for enhancing the spatial gradient (Pacini, 1994). Qualitative analysis of geographic variations in bottom sediments indicates a pronounced east-west transition. The largest particles in the sediment load entering the impoundment (ie. fine sand or larger) remain deposited at the mouths of the reservoir whereas finer-sized gray muck and black ooze predominate in main part of the reservoir. If this pattern of depth-related sediment deposition continues, additional volume storage losses will occur primarily in the old river channels and at the area near the dam wall.
Sedimentation in Masinga reservoir has a pattern of deposition that is markedly unique. Deposition in deeper areas is resulting in a reservoir with a nearly uniform bottom depth in the old river channels of between 30 to 46 m. Investigation of the spatial pattern of sediment deposition indicates that deposition is primarily in the upper ends of the arms of the reservoir, at the confluence of the two arms and at the downstream end of the reservoir.

**Fig. 5.44: Bathymetry Masinga reservoir in the area around the Island (a) in 1981, (b) in 1988, and (c) in 2000**
5.3.3 Recommendations for Improving GSTARS 2.1 Simulated Results

The following recommendations could help to improve the results of GSTARS 2.1 simulations carried out in this study:

(i) Installation of more river gauging stations and reviving the abandoned ones to improve data on the hydrological regime of the area. Data should be collected on a more regular and uniform basis than before.

(ii) Installation of additional sediment monitoring stations and stepping up measurement of sediment transport in rivers. The supply of bed load to a reservoir determines the depositional process close to the mouth of the inflowing river. The proportion of bed load to total load should therefore be estimated, calculated or measured and be included in the reservoir sedimentation simulations.

(iii) Continuation of regular sediment surveys on Masinga reservoir. Hydrographic surveys should be undertaken once every 3 to 5 years.

(iv) Shear threshold for deposition of clay and silt (STDEP), shear threshold for particle erosion of clay and silt (STMERO), and slope of the erosion rate curve for mass...
erosion (ERMASS) parameters should be determined *in situ* or by laboratory tests to improve on the results of GSTARS 2.1 simulations. The parameters are highly dependent on the local conditions and may vary widely from case to case, always requiring field verification.
CHAPTER SIX

SEDIMENT MANAGEMENT STRATEGIES

The planning, design and implementation of measures to mitigate the undesired effects of reservoir sedimentation are important parts of a water resources management scheme. They consist of measures to reduce the quantity of sediment entering the reservoir, to reduce the deposition of sediment in the reservoir and to recover part of the lost storage. First, general reservoir sediment management strategies are outlined, citing examples where they have been applied. Secondly, sediment management strategies specific for Masinga catchment and reservoir are suggested in sections 6.5 and 6.6, respectively.

6.1 METHODS OF MINIMISING DEPOSITION IN A RESERVOIR

This section gives an evaluation of some available techniques of reducing the entry of sediment into the reservoir, hence, prolong its useful life.

6.1.1 Reduction of Sediment Inflow by Soil Conservation

Soil conservation methods in preventing the movement of soil particles or preventing the transport of sediment to the reservoir include watershed structures and land-treatment measures in the catchment.

Several types of structures may be built in a watershed. These include: sedimentation basins to store sediment permanently for the design life of the reservoir or to store sediment for specific storm runoffs before periodic clean-out, drop inlets and chutes for reduction of gully erosion, stream-bank revetments to reduce stream-bank erosion, and sills or drop structures for stream-bed stabilisation.

Watershed land-treatment measures to reduce sheet erosion include soil improvement, proper tillage methods, strip cropping, terracing, and crop rotations. If the watershed is not very large, the effect of soil conservation can be felt in a short time. But where vast areas with poor natural conditions are involved, the soil conservation works can hardly be effective in a short period of time. Hence, the effectiveness of soil conservation for large catchment areas like Masinga cannot be estimated with accuracy.

Two cases of soil conservation programs designed to reduce sediment transport into the reservoir are cited. The Tungabhadra Reservoir Project (Rajan, 1982) was constructed on the river Tungabhadra in India in 1953. The storage capacity of the reservoir is $3.75 \times 10^9$ m$^3$. The catchment area at the dam site is 28,178 km$^2$, 14.5% of which is forest, 53% of heavy soils and 32.5% of erodible soils including black cotton soil. The project was planned assuming a rate of siltation of $12.8 \times 10^6$ m$^3$ per year. Sediment surveys were taken on the reservoir in 1963, 1972, and 1978. Table 6.1 contains the volume of sediment deposited for various periods and the rate of siltation expressed on a unit area basis.
It can be seen from Table 6.1 that siltation rate was reduced after 1963. The reduction is partly attributed to trapping of sediments in reservoirs on the upstream tributaries and partly due to watershed treatment. Soil conservation methods such as contour bunding and afforestation had been introduced. It was estimated that an area of 4571 km$^2$, consisting of erodible soils present in patches in the catchment area, was the most vulnerable, and required urgent treatment. By 1978, an area of 3075 km$^2$ had been treated, 2085 km$^2$ by soil conservation methods and 233 km$^2$ by afforestation. This led to further drop in the rate of siltation in Tungabhadra reservoir.

Table 6.1: Rate of Siltation in Tungabhadra Reservoir. Extracted from Unesco, Paris (1985).

<table>
<thead>
<tr>
<th>Period</th>
<th>No. of years</th>
<th>Volume deposited ($10^6$ m$^3$)</th>
<th>Rate of siltation ($10^6$ m$^3$/km$^2$ p.a.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1953 – 1963</td>
<td>10</td>
<td>504.72</td>
<td>1794</td>
</tr>
<tr>
<td>1953 – 1972</td>
<td>19</td>
<td>322.79</td>
<td>600</td>
</tr>
<tr>
<td>1953 – 1978</td>
<td>25</td>
<td>418.81</td>
<td>594</td>
</tr>
<tr>
<td>1972 - 1978</td>
<td>6</td>
<td>96.02</td>
<td>566</td>
</tr>
</tbody>
</table>

The Guanting reservoir built in 1956 on the Yongding river in North China is another example to illustrate the effect of soil conservation measures in reducing sediment inflow into the reservoir. The catchment area at the dam site is 43,400 km$^2$ while the average modulus of erosion of the watershed is about 3,000 t/km$^2$ per year with a maximum value of 18,000 t/km$^2$ per year. The storage capacity of the reservoir is 2.29 x $10^9$ m$^3$. The annual runoff at the Guanting station is 1.4 x $10^9$ m$^3$ and the annual sediment load is 81 x $10^6$ tonnes. During the early years (1956-60) of reservoir impounding, sediment deposition amounted to 360 x $10^6$ m$^3$. Since 1958, about 300 reservoirs have been built in the upstream reaches of the Guanting reservoir with total storage capacity of 1.5 x $10^9$ m$^3$. The reservoirs built to conserve soil resulted in a reduction of both runoff and sediment yield (Unesco, 1985). Diversion of silt-laden flow with high content of fertilizer into farmlands for irrigation coupled with soil conservation measures caused a remarkable reduction in the amount of sedimentation and the rate of siltation in Guanting reservoir (Table 6.2).

Table 6.2: Reduction of Rate of Siltation in Guanting Reservoir. Extracted from Unesco, Paris (1985).

<table>
<thead>
<tr>
<th>Period</th>
<th>Total amount of deposition ($10^6$ m$^3$)</th>
<th>Rate of siltation ($10^6$ m$^3$ per year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1956 – 1960</td>
<td>350</td>
<td>70</td>
</tr>
<tr>
<td>1961 – 1970</td>
<td>82</td>
<td>8.2</td>
</tr>
<tr>
<td>1971 - 1980</td>
<td>73</td>
<td>6.6</td>
</tr>
</tbody>
</table>

6.1.2 Trapping and Retention of Sediment by a Vegetative Screen

A vegetative screen is most effective in preventing the sediment from entering the reservoir. Such screens, whether artificial or natural, when situated at the head of the reservoir serve to diffuse the incoming flow, reduce its velocity and cause sediment to
deposit. Thus, a great amount of incoming sediment can be trapped at the head of the reservoir and prevented from penetrating farther into the reservoir basin. The growth of Salt Cedar along the Pecos river above lake McMillan, USA, helped in trapping the sediments, hence, reducing sedimentation in the lake (Stevens, 1936).

However, one ill-effect of increased vegetation cover is the increase of water consumption. According to Maddock (1948), the increased vegetation in the upstream reaches of Elephant Butte reservoir on the Rio Grande led to over $123 \times 10^6$ m$^3$ of water being consumed annually by transpiration. This loss accounted for almost 10% of the yearly water supply (Garde et al., 1978).

6.1.3 Bypassing of Heavily Sediment-laden Flows

The construction of bypassing channels or conduits is one of the principal methods used to control the inflow of sediment to impounding reservoirs. Generally, a great amount of sediment is carried by the river flow during the flood periods, especially in arid and semi-arid regions. Thus, when a large part of the flow with high concentration is bypassed through a channel or tunnel, serious silting in the reservoir may be avoided.

A scheme in Switzerland was constructed with the diversion dam and bypass tunnel above the Amsteg Reservoir on the Reuss river. According to Reed (1981), the Amsteg dam is 32 m high, with a storage capacity of only $197 \times 10^3$ m$^3$. The drainage area above the reservoir is 404 km$^2$, and the flow of the Reuss river ranges from minimum of 1.98 m$^3$/s to flood flow of 396 m$^3$/s. In order to prevent river debris from filling the reservoir, a bypass tunnel was constructed around the reservoir. The flow through the tunnel is regulated by gates, so that the flow into the reservoir is only sufficient to supply the power-plant demand. During the low-flow season, when the full flow is utilised through the plant, little sediment is carried by the stream. During flood periods about one-third of the total flow is spilled over the diversion dam and passes through the reservoir. This is the part of the stream flow which carries only silt and fine sand, while the large part of the flow heavily-laden with sediment is bypassed through the tunnel. Observation made after three years of operation showed that deposition in the reservoir had become insignificant.

The construction of a tunnel around a reservoir is very expensive. The presence of suitable topographic features near a reservoir which permit the installation of a short bypass canal or conduit is invaluable economically. It seems that to prevent silting for power dams which have small reservoir capacity, and for which it is necessary to maintain a constant head, it is often economically feasible to build bypass structures. Other benefits include the reduction of sediment damage to water turbines and other power equipment.

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6.2 METHODS OF MAXIMASING SEDIMENT REMOVAL THROUGH FLOW

6.2.1 Flow Regulation During Floods

The purpose of regulating the flow through a reservoir during the flood season is to release as much sediment as possible from the reservoir by making use of the high silt carrying capacity of the flood. Generally, the regulation of flow is achieved by lowering the water level during the flood season by operating the deep or bottom outlets under controlled or uncontrolled conditions. During the period of rising water level of a flood in detention reservoir, the outflow silt discharge is always smaller than that of the inflow, as a result of the backwater effect, and the consequent decrease in the velocity of the flood waters. Subsequently, during the lowering of the water level at the dam in the absence of a backwater effect, the outflow silt discharge is often greater than the inflow, due to erosion occurring in the reservoir.

Operation of reservoirs by lowering the pool levels during the flood season to sluice out waters with high sediment concentration is normal in many reservoirs in China (Unesco, 1985). Heisonglin reservoir is one example employing this mode of operation (Zhang et al., 1976). Before July 1962, the reservoir was operated as an impounding reservoir, resulting in serious siltation. The average rate of sediment deposition amounted to 540,000 m$^3$ per year. After August 1962, when the mode of operation of the reservoir was changed to involve lowering the pool level during flood season and impounding water outside the flood season, the rate of deposition decreased to 93,000 m$^3$ per year. This mode of operation of storing the clear water during the non-flood season, discharging the muddy water during the flood season, and diverting the muddy water for irrigation and warping not only decreases the rate of siltation, thereby preserving the long-term usable storage capacity of the reservoir, but it also enables both the sediment and flood water to be used in agricultural production.

In a dry detention reservoir, where the dam is only for flood reduction, surges generally evolve. Sediment is deposited as a result of backwater effect and surges occur in the reservoir when the water level is rising. When the water begins to draw down, the velocity of the flow is increased and erosion of the deposits occurs. This kind of erosion in the reservoir is referred to as retrogressive erosion, because erosion always progresses in the upstream direction during the sudden drawing down of the water level. During this process, much of the sediment previously deposited above the dam is eroded or a large proportion of the incoming sediment is carried through and released from the reservoir.

6.2.2 Draw-down Flushing

Drawing down the water level in a reservoir for the sake of reducing the amount of sedimentation, or in order to induce erosion of deposited sediment to recover storage capacity, is a method often used in reservoirs, especially those of hydro-electric power stations. The efficiency of sediment flushing depends on the topographic position of the reservoir, the capacity of the outlet, the outlet elevation, the characteristics of the inflow
sediment, the mode of operation, the time duration of flushing, the flushing discharge, etc.

Draw-down flushing has been practised at the Khashm El Girba dam in Sudan. The dam was completed in 1964, on the Atbara river. Its original storage capacity was $950 \times 10^6$ m$^3$ and the water was used for irrigation, hydro-electric power and for water supply. The capacity was seriously depleted by silting of the reservoir as a result of the average annual sediment inflow of about $84 \times 10^6$ tonnes. According to El Hag (1980) and El Fatih Saad (1980), the sediment outflow during the flushing operation periods was $85 \times 10^6$ tonnes, which is greater than the estimated average annual sediment inflow. This was attributed to the erosion of deposits from the reservoir.

Draw-down flushing has got some setbacks. The quantity which could have been evacuated is limited partly because the fine sediment deposits becoming consolidated, partly because deposition of the bed load occurring in the upper part of the reservoir, and partly due to the high elevation of spillway through which the flushing discharge must pass. This method may not be economical in terms of energy used during the flushing process. If for example, bottom outlets were available, it is certain that more sediment would be sluiced out and more storage capacity recovered.

6.2.3 Density Current Flushing

Gould defines a density current as a gravity flow of turbid water through, under or over water of different density (Fuat, 1994). The density difference being a function of the differences in temperature, salt content or silt content of the two fluids. The venting of density currents has long been considered an effective means of reducing the rate of reservoir silting, especially in impounding reservoirs. Following the recognition of the phenomenon of density currents, the method of density current flushing has been adopted in many reservoirs to reduce sedimentation (Unesco, 1985). In lake Mead, USA, density currents have been vented out through a diversion outlet (Unesco, 1985). Other examples include the Sautet reservoir in France, the Metka and the Groshnitz reservoirs in Yugoslavia, and the Nulek reservoir in Russia (Mihailova et al., 1975; Pyrkin et al., 1978).

6.3 RECOVERY OF STORAGE

6.3.1 Flushing of Deposited Sediments

Reservoir-emptying operations may be used periodically for small reservoirs where the storage capacity could not be maintained for beneficial use after a period of several years of operation. Since a great part of the useful storage capacity in a small reservoir is located near the dam site, the sediment deposits may be removed by flood flow if the outlet gates are left open for a period of time. The channel, thus scoured out in the deposits, becomes a part of the storage capacity. Emptying and flushing operations may be used in reservoirs where a balance between deposition and erosion cannot be obtained by flushing sediment during the flood and storing clearer water during the non-flood

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seasons. An example of a reservoir operated by emptying and flushing is the Hengshan reservoir, a small gorge type reservoir, 1 km in length, with a storage capacity of 13.3 x 10^6 m³. During emptying and flushing, strong retrogressive erosion occurred as a result of lowering the water level in the reservoir. A channel was rapidly formed in the flood plain deposits, and this deepened continuously and extended upstream. On the flood plain surface, mud slid gradually into the channel, lowering the level of flood plain deposits. The deposited fine sediment (D_{50} = 0.02 mm) had a high water content and were capable of sliding into the channel where they easily eroded and released from the reservoir at high concentrations (Unesco, 1985).

How to select the time for emptying and flushing, and how to predict the instant and duration of flushing for evacuating the sediments are most important problems in reservoir operation. Experience in the Hengshan reservoir suggests that greater recovery of capacity could have been achieved if the reservoir were emptied before the arrival of the flood, so that the flood waters could exert their strongest erosive force on the deposits, which had not yet consolidated after emptying of the reservoir. The period of flushing should be restricted to the flood season, so that no further serious deposition would occur after the flushing period.

6.3.2 Dredging

Dredging to remove sediment from the reservoir is undertaken where

(a) flushing is not successful,
(b) the building of bypass is impossible
(c) the draw-down of the pool level for flushing is not allowed for the sake of saving water,
(d) the dam is irreplaceable with no possibility of further raising the dam height, or
(e) the energy consumed in the flushing by lowering pool level or emptying the reservoir is uneconomical for reducing the rate of silting in the reservoir.

Generally speaking, dredging is an expensive means of restoring the storage capacity of a reservoir unless the deposits removed can be used for beneficial purposes. Some coarse sediments dredged may be used as building/construction material. Dredging in reservoirs is performed:

(i) To recover the storage capacity of small-sized reservoirs, small compensation basins, gravel retention basins; or to partially recover the capacity of medium sized reservoirs. For example, in Algeria, dredging is undertaken during the irrigation periods to regain storage capacity (Bellouni, 1980; Belachir, 1980).
(ii) To clear the deposits in the backwater ponds of a chain of power stations to lower the flood level in the river channel, or to maintain a necessary navigation depth in the backwater sedimentation reach of the reservoir. In Austria, gravel deposits along the river Danube are periodically removed by dredging to prevent raising of the flood levels and maintain the clear height of 8 m required for navigation below the railway bridges (Unesco, 1985).
(iii) In the case of deposits that may be used as aggregates for concrete, the dredging method is not costly. There are many reservoirs in Japan where the deposits dredged out from reservoirs are used as concrete aggregates. Three reservoirs; Akiba, Sukuma and Miwa fall into this category (Nose, 1982; Okada et al. 1982; Shiozawa, 1974; Murakami, 1979).

6.3.3 Siphoning

Siphon dredging used for desilting reservoirs differs from ordinary suction dredging in exploiting the hydraulic head difference between the upstream levels of the dam as a source of motive power for the suction dredging. Siphon dredgers have been used in small dams in north and north-west China for restoring storage capacity, e.g., the experimental siphon dredging used in the Tianjiawan reservoir (Unesco, 1985). It had the following advantages:

(a) Low cost of dredging,
(b) There is almost no waste of water during the operation. The water sediment mixture discharged is used for irrigation and serves as fertilizer in warping areas.
(c) The siphon dredger is easily maneuvered, and has high flexibility.

6.4 BOTTOM OUTLET STRUCTURES

Whenever reservoir silting is likely to be a problem, the potentialities of the bottom outlet release of sediment through a dam should be considered in the dam design; and at the same time, the form of reservoir operation should also be taken into consideration. Of all the methods of sediment sluicing, the use of bottom outlets seems to be one of the most effective. Bottom outlets may be used under the following circumstances:

(i) For undersluicing the flood, or draining the reservoir under emergency conditions when lowering the reservoir water level is urgently needed in a short period of time.
(ii) For sluicing sediment by drawing down the water level in the reservoir, to release sediment deposits (sils, sands and gravels), which are eroded by the tractive force of flow. Similarly, density currents may be vented out from an impounding reservoir.
(iii) When bottom outlets are located below the power intake, they may be useful in preventing the silt from entering the power intake, minimising the possibility of wear occurring in turbines.

An example where bottom outlets are used is in the Old Aswan Dam located on the river Nile near Haifa, Egypt (Unesco, 1985). Sediment-laden flow water of floods was allowed to pass through the reservoir basin without appreciable diminution of velocity, hence reducing the rate of siltation.
6.5 Masinga Watershed Suggested Management Strategies

6.5.1 Background

From the results of WEPP model erosion prediction, it is evident that soil erosion in Masinga catchment is alarming. There is therefore an urgent need to step up soil conservation programmes in this area. Apart from reducing the rate of loss of top fertile soil from agricultural areas, soil conservation in the catchment will help reduce the rate of siltation in Masinga reservoir. Over the last 35 years, land use in the area has evolved from controlled grazing to settled agriculture intermixed with extensive grazing. With unreliable rainfall, relatively shallow and infertile soils, and continuing population increase, the pressures on the land are acute. Land and water resources are being increasingly threatened in the face of these rising demands, resulting in increasing erosion problems and the opening up of marginal areas. Today, there is widespread evidence of high rates of soil erosion, degradation and surface runoff. There is need, therefore, for an integrated approach on soil conservation planning and execution.

To ensure success of a soil conservation scheme, key elements of the programme should include:

(i) Integrated planning and implementation of effective physical conservation measures.
(ii) Increased protection of the arable land and encouragement of better agronomic and cultivation techniques to promote increased infiltration of rainfall.
(iii) Reduction of surface runoff from grazing areas.
(iv) Reduction in livestock numbers and greater integration of livestock with arable farming.
(v) Improvements to road design and control of gully erosion on tracks.
(vi) Upgrading of extension services.
(vii) Rehabilitation of severely degraded lands by exclusion of livestock.
(viii) Improved domestic and livestock water supplies

An effective combination of improved physical structures and agronomic measures will be important in the arable areas. This is the only way to attain soil and water conservation during the early stages of the crop growing seasons. Improved agronomic practices would help to contain soil erosion during the later stages of the growing cycle.

There should be strong encouragement for livestock production to be integrated with crop production and to create conditions whereby the quality of cattle becomes more important than the quantity.

Soil conservation measures have been applied in Masinga catchment before but without much success (Atkins, 1984). The following points should be considered in the formulation of soil and water conservation programme objectives to ensure success of the project:

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(i) Provide a strong organisational framework to initiate improved soil conservation works in high priority areas, largely through voluntary communal work effort. This framework should be readily absorbed into the Ministry of Agriculture (M.O.A) activities on project completion and be replicable throughout Masinga catchment.

(ii) Provide the technical support, training supervision, materials and equipment required to achieve realistic work programme targets.

(iii) Integrate closely with existing conservation programmes and policy, with particular emphasis on harnessing the evident goodwill and support of the administration, at all levels, and farmers themselves. Popular support and participation are a prerequisite to project success. Project staff should actively participate in all existing committees concerned with soil conservation.

(iv) Focus on practical in-field construction and maintenance of physical conservation measures as a complement to the improved agronomic practices that have been advocated through the Kenyan television programmes.

(v) Step up provision of supplies of tree seeds and seedlings for agro-forestry and forestry planting to ease pressure on present woodland and reduce the current deficit in fuel wood and pole timber.

(vi) Limit the scale and phasing of the programme to take realistic account of the availability of trained personnel.

(vii) Address the problems of road-related gully erosion and other erosion problems of public concern through creation of specialised units. Co-operation with the Ministry of Transport and Communication would be needed to achieve this objective.

6.5.2 Technical Recommendations

6.5.2.1 Physical Conservation

All physical conservation structures which dispose of runoff water require that the volume of anticipated water be determined and a channel designed to contain this at a non-erosive velocity. This erosive velocity threshold can be increased by improving the lining from bare soil to grass or some other material such as rock, concrete, etc. It is important to apply this design approach to structures which have widely varying characteristics, especially cut-off drains and waterways.

In planning physical layouts, the following points require consideration:

(i) Physical conservation measures must effectively reduce erosion,
(ii) Physical conservation measures must not cause erosion elsewhere,
(iii) Anticipation of future development is necessary to avoid conflict between present and future conservation requirements,
(iv) Water should, as far as possible, be allowed to follow its natural course.

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The following physical control measures are recommended for Masinga catchment:

1. **Cut-off Drains**

In the uplands, attention should be given to the dimensions of cut-off structures in relation to the anticipated runoff. There is also a need to develop design criteria to replace the present ‘trial and error’ practice.

In the lowlands, cut-off drains need to be constructed as part of an integral layout. Suitable waterways are needed into which water from cut-off drains can safely be discharged.

2. **Waterways**

In the uplands, emphasis should be placed on detention of water to minimise the need for waterways. Existing boundary footpaths and waterways should be given increased protection or realigned, where feasible, to reduce risks of gully erosion. Protection measures should include improved grass cover along boundaries and use of bags filled with topsoil to aid establishment of grass in waterways on steep slopes.

In the lowland areas, emphasis should be on protection of existing natural waterways and establishment of artificial waterways. These should be extended through grazing areas to intercept cut-off drain discharge. Protection should be provided to prevent overgrazing of grassed banks by livestock.

3. **Bench Terraces**

Bench terraces in Masinga catchment are virtually exclusive to the coffee zone, where they are of a reasonable standard. Technical advice should be made available so that bench terracing can be applied to other crop zones.

4. **Fanya Juu**

In the uplands, fanya juu should be encouraged on steep slopes under annual crops. In the lowlands, the inter-terrace area requires measures to detain water for the benefit of crops and reduction of run-off. Tied ridging methods should therefore be introduced for this purpose. Conventional graded, narrow channel terraces, combined with inter-terrace tied ridging should also be tested.

5. **Inter-Terrace Practices**

In the uplands, establishment of grass/vegetation contour strips (up to 1.0 m) should be stepped up, especially in areas of annual crops on steep slopes. In the drier areas, grass strips up to 2.0 m in width should be introduced. On slopes up to 10%, inter-terrace ridging should to be encouraged, with a cross-tie every 1 to 2 m.
6.5.2.2 Other Conservation Measures

Emphasis should be laid to measures which are considered to be implementable. As conservation is a dynamic concept, measures which are appropriate now may not be so in ten years time due to changes occurring in the catchment. Apart from physical conservation, other measures that can be applied to Masinga catchment to reduce sediment yield include:

(i) Good agronomic practices to conserve soil, i.e., prevent loss of top fertile soil from arable land.
(ii) Better livestock management to prevent overstocking.
(iii) In the lowlands areas, efforts should be made to improve the carrying capacity of the range lands.
(iv) Expansion of agro-forestry to reduce fuel wood deficits and increase the protection benefits of trees against soil erosion.
(v) Improvement on road design and construction to minimise erosion from roads.

6.6 SUGGESTED MANAGEMENT STRATEGIES FOR MASINGA RESERVOIR

6.6.1 Sedimentation Basins

Sedimentation basins are structures placed within a water course to capture sediments from the flow. Sedimentation basins rely primarily on physical processes to treat the water and are designed to trap sediments upstream of a natural or constructed wetland or reservoir system. The rate at which a sedimentation basin removes sediments from the water is affected by the sediment particle size, the velocity of the water, and detention time.

Basins have enlarged cross-sections that create slow tranquil flow and allow sediment to settle out. If correctly designed, sedimentation basins can provide a comprehensive solution, and can operate successfully where alternative measures for controlling sediments are ineffective, for instance, where either head or flows for flushing are unavailable or where sediments are relatively fine. Correct design is crucial to sedimentation basins operating effectively and should take into consideration the following design criteria:

(i) the trapping efficiency of a basin
(ii) concentrations and composition of the material passing into the reservoir downstream from a basin
(iii) the time taken for a basin to fill with sediments
(iv) the time required to sluice a basin
(v) the size grading of the reservoir bed material downstream from a basin.

Sedimentation basins placed at the mouths of Masinga reservoir (Fig. 6.1) could be the most attractive and reliable measure for preventing excessive sediment loads from entering the reservoir. The purpose of the basin will be to detain the sediment-laden flow.
from the rivers (Tana and Thika) long enough for the majority of the sediment to settle out in the respective basins. This will reduce sediment transport far downstream into the reservoir. Sedimentation basins by no means trap all the sediment that washes into them. Sometimes more than half of the sediment flows through. Therefore, sedimentation basins, as with all sediment controls, should be used in conjunction with erosion control practices as suggested in section 6.5 to reduce the total amount of sediment washing into Masinga reservoir.

Sedimentation basins have relatively good sediment-trapping efficiencies and require little maintenance compared to other practices used to treat sediment-laden runoff. Commonly, trapping efficiency varies between 60% and 80%. If a higher trapping efficiency is desired, a sedimentation basin with larger storage volume and longer detention time should be used. However, trapping efficiency should be optimised within site constraints. Sedimentation basins are considered as the most useful and cost-effective measures for treating sediment-laden flow and are relatively easy to construct. Hence, they are generally recommended as the principal sediment-control practice for flow entering the reservoir.

However, the following are some of the limitations of sedimentation basins:

(i) Sedimentation basins are relatively large, generally requiring a good deal of site area
(ii) It can be difficult to keep a sedimentation basin functioning during the entire period of reservoir operation
(iii) Improper construction and maintenance greatly reduces their effectiveness
(iv) Not particularly effective for fine silts or clay soils, or for intense rainfall events, which can re-suspend sediment within the basin.

Sedimentation basins require routine maintenance to remain effective as sediment traps. When the sediment reaches the maximum level assumed in the design (usually one-third to one-half the basin volume), it must be removed. Head should be provided for in the design to make it possible to sluice out the deposited material instead of using mechanical means. Mechanical removal of sediment makes operation of sedimentation basins more expensive. Excavated sediments must be placed in a location where it will not easily be eroded again. In addition to sediment clean-out, sedimentation basins should be inspected after large storms to determine whether the embankment or spillways sustained any damage that requires repair. If the outlet becomes clogged with sediment, it should be cleaned to restore its flow capacity.

**6.6.2 Dredging at Specific Locations of Masinga Reservoir**

Dredging is the removal of sediments by earth moving equipment or suction dredge. It is a short-term remedial measure to alleviate the problem of sedimentation in the reservoir and do not provide a long-term solution to the problem. This method can restore storage to its maximum because it can remove bank deposits which flushing cannot handle.

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From the results of sedimentation modelling in Masinga reservoir and the pattern of sedimentation predicted for 2000, it can be seen that deposition occurred in specific areas. Principally, deposition occurred in the old river channel and very little on the reservoir terraces. However, modelling revealed that there is preferential sedimentation at the two mouths of the reservoir, at the confluence of the two reservoir arms and near the dam wall. These are areas that should be targeted for dredging (Fig. 6.1). The delta, which have been formed at the mouths of the reservoir could be cleared by dredging, thereby recovering part of the storage and also eliminating the possibility of flooding.

Dredging may not be economically viable unless the dredged material is used for economic benefits. The fine grained cohesive silt deposits at the confluence and near the dam wall could be used for making clay bricks for construction works. The dredged material from the mouths of the reservoir could be used for industrial or landfill purposes.

![Diagram showing suggested management strategies for Masinga reservoir]

**Fig. 6.1: Summary of suggested management strategies for Masinga reservoir**

**6.6.3 Periodic Operation of Bottom Outlet Gates**

This is a remedial measure which makes use of the bottom outlet gates of the dam. These gates are part of the permanent hydraulic structure of the dam. In this case, the removal of sediments is through reservoir operation. This method makes use of the hydraulics of flow and mechanical means to remove sediments that have accumulated in the reservoir. Water is released through these low-level outlets leading to large flow velocities in the approach channel, providing a local concentration of flow that washes out the sediments downstream. This method could be the most appropriate for Masinga reservoir because it is cost-effective.

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CHAPTER SEVEN

SUMMARY, CONCLUSIONS AND PROPOSALS FOR FURTHER RESEARCH

7.1 Summary and Conclusions

Masinga reservoir impounded on Tana river for generation of hydro-electric power and for regulation of flow to downstream reservoirs is imperilled by rapid sedimentation caused by intense soil erosion in Masinga catchment. A systematic and rational integrated approach to determine the rate of soil erosion, sediment yield from Masinga catchment and modelling of sedimentation in Masinga reservoir was done. This was accomplished through application of an erosion prediction model, WEPP, on the catchment and a sedimentation model, GSTARS 2.1, to model the sedimentation process in the reservoir. The simulation results from the sedimentation model were processed using a two-dimensional BOSS SMS model for visualisation purposes. Where possible, the results from the erosion model were compared with the results from the sedimentation model with respect to the volume of sediment yielded from the catchment and the volume of sediment deposited in the reservoir, respectively. Some comparison was also made with the work of other researchers. The main conclusions arrived at are summarised here.

1. The WEPP hillslope profile model is designed to simulate runoff, evapotranspiration, total soil water, rill erosion, interrill erosion, sediment yield at the end of the hillslope, etc. The model uses many algorithms from the SWRRB model. The WEPP model was run using data from Masinga catchment in Kenya for the period from 1981 to 2000. Comparison of model-simulated runoff and runoff obtained from separation of hydrograph components indicates that the model estimates are representative of the Watershed. This is further supported by a good agreement between the model-simulated and field computed potential evapotranspiration for Thika sub-catchment. In addition, it indicates that the model is able to simulate antecedent soil water content which is used in infiltration component of WEPP. The model is therefore desirable for use in modelling sediment input into Masinga reservoir.

2. The simulations obtained from WEPP hillslope profile version demonstrate that the undisturbed forested areas typically yield 3.3 – 13.2 t/ha of sediment per year. Cultivated lands have a high variable sediment yield which depends upon the amount of rainfall, slope, soil type, land use and management practice applied. Values documented for the cultivated areas of Thika sub-basin range from 0.64 – 45.05 t/ha of sediment per year. In the Tana sub-basin, however, the sediment yields of the cultivated areas range from 0.02 – 302.5 t/ha of sediment per year. Highest contributions of sediment, according to the WEPP model, are from Mathioya and its neighbouring areas draining the Aberdares with sediment yield contributions of over 300 t/ha per year. Of these contributions, bed load was assumed to constitute 10%. Most of the sediment produced is silt and fine sand because of the intense chemical weathering typical of humid tropical regions.

3. The results from WEPP watershed simulations are generally representative of suspended load at the inlet of Masinga reservoir. The water balance for runoff demonstrates that the program is able to model runoff peaks satisfactorily. However, the model is less desirable for simulating suspended load when no rainfall occurs. The procedure cannot be applied to areas having permanent channels such as classical gullies and perennial streams. As a whole, the WEPP output provides a potentially powerful tool for conservation planning. It is easy to view and interpret the results since the model estimates where and when soil loss problems occur on a given hillslope for a given management system. The model also provides an inexpensive and rapid method of evaluating various soil conservation options.

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4. The results of suspended load from WEPP watershed model compare well with measured values. However, there is high fluctuation in simulated sediment load with maximum being about 200 mg/l for Thika and about 160 mg/l for Tana. Mean values of suspended load for Thika and Tana are 45.2 mg/l and 23.7 mg/l, respectively.

5. In this study, an attempt was made to derive sediment rating curves for both Tana and Thika sub-catchments. These curves were then used as input into the sedimentation model for describing inflow suspended load as a function of water discharge. The rating curves exhibit a strong correlation between suspended load and water discharge. However, they should be taken as approximations only.

6. GSTARS 2.1 was developed due to the need for a generalised water and sediment-routing computer model that could be used to solve complex river/reservoir engineering problems for which limited data and resources are available, as the case is, for Masinga catchment/reservoir. The model is based on stream tube concept and is capable of simulating channel geometry changes in depth and width simultaneously using total stream power minimisation theory. GSTARS 2.1 was applied to a 45-km reach of Masinga reservoir on river Tana, which was represented by 50 cross sections. The model was calibrated using field data and was then used for predicting reservoir bed changes over a 20-year period, from 1981 to 2000. The prognosis simulation suggests that future reservoir bed-evolution will depend strongly on the amount of inflow sediment and its grain size distribution.

7. The results of calibration/Prediction simulations show that GSTARS 2.1 can simulate longitudinal reservoir bed-evolution with reasonable accuracy, implying that the model can describe selective fractional deposition and erosion processes within the reservoir under unsteady flow conditions fairly well. There was good agreement between model simulated and measured cross-sections for most stations. However, the model seemed to overestimated sediment deposition for areas near the dam wall and at the confluence of the two arms (Tana and Thika). The programme simulated deposition on flood plains reasonably well but failed to simulate deposition in deeper parts of the channels.

8. GSTARS 2.1 predicted a reduction in reservoir storage capacity of 13.7% for the period between 1981 and 2000 while WEPP model predicted a value of 10.1% storage capacity reduction for the same period.

9. Visual pictures processed with BOSS SMS to show sediment distribution in the reservoir has led to the conclusion that all the dead storage has already been filled up by sediment deposits and part of the active storage has also been filled up. More deposition occurred in the old river channel, at the mouths of the reservoir, at the confluence and the area near the dam wall. These are areas where dredging should be carried out.

10. Of the available reservoir sediment management strategies, watershed management is the best method to reduce the yield of sediment and its entry into the reservoir. Vegetative screens at the upstream end of the reservoirs may withhold a significant part of the entering sediment. Construction of sedimentation basins at the mouths of the reservoir may be the most feasible solution to sedimentation problems in Masinga reservoir. Periodic sluicing of sediments through operation of bottom outlet gates may be another approach of sediment management in this reservoir.

7.2 Proposals for Further Research

Based on the findings of this research, further studies of a more general nature are recommended as a contribution to a greater understanding of soil erosion, and the soil and water conservation problems in Masinga catchment, as well as the degree of siltation in Masinga reservoir.

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It would be important to mount a project for refining of the estimates of the short and long-term runoff characteristics of the basin, so that the water resources may be developed and used in an optimum way.

There is need to formulate a programme for determination of the proportions of runoff and sediment originating in various sub-basins and zones. Hence, determination of the relative amounts of soil lost from agricultural land, road drainage, footpath, gully and stream bank erosion (especially in the uplands) will be necessary and appropriate.

In addition to measurements of sediment transport within river channels, there is a need to study the distribution and nature of sediment sources within the catchment as they relate to the geomorphic and hydrologic processes. Only when the major sediment sources and erosive processes are identified, can rational decisions be made about the kinds of soil conservation measures that will be effective. The kinds of soil conservation measures required to minimise erosion from roads, for example, are different from those needed for cultivated fields.

Some effort should be put in monitoring the effects of soil and water conservation measures on river and sediment yields and the reductions in the rates of soil losses, including the verification of those zones in which the greatest benefits can be achieved.

A well-funded research programme should be initiated in order to assess the erosion problems within Mathioya and neighbouring areas that have been documented as the major sediment contributing regions within the arable areas. In the upper reaches of Masinga catchment, significant areas of the forests are being removed from extremely steep slopes. This tendency is likely to continue because of land pressure and therefore its effects on water resources and sedimentation problems need to be addressed. The resulting accelerated soil erosion speeds up siltation of the reservoir. Therefore, there is a need for some further geomorphic and hydrologic research to quantify the effects of deforestation of the steep slopes on water resources and sedimentation in Masinga reservoir. In this dissertation, sedimentation process was studied assuming non-transient land-use and climate conditions which is not true in nature.

Research should be instituted into finding the most appropriate soil and water conservation methods suitable for the local areas within Masinga catchment which is characterised by variability of topography, land use, rainfall, soils and other cultural and social conditions. In this research, recommendations of conservation measures were given considering erosion problems at a macroscopic scale. It should be more appropriate and effective to tackle erosion problems based on small and localised areas with unique topography, land use and soils. Hence, determination of the rates of soil loss in various landform units under different land uses to identify cropping/soil/slope combinations with a high erosion hazard is desirable.

With limited additional support, it would be possible to extend sediment measurements to incorporate the sampling of bed load. This will be useful for defining the bed load contribution to total sediment rates. No research has so far been documented on study of bed load transport in Tana river, yet it contributes significantly to the total sediment rates, especially in the upper reaches of the river. It would also be interesting to carry out research for defining bed load transport at critically high flows in the catchment.

A research on determination of the soil tolerance in relation to soil losses would help to assess the relative degree of soil degradation.
It is very likely that the economic benefits from Masinga reservoir has dropped because of reduction in its useful storage capacity due to siltation. There is a need, therefore, to do an evaluation of the economic impact of sedimentation on this reservoir.

A thorough evaluation of sedimentation problems in Masinga reservoir need to be adequately assessed based on extensive investigation of hydrologic and geomorphic processes as they relate to present and probable future land-use changes.

As another means of defining total sediment input into Masinga reservoir, a hydrographic survey should be carried out as soon as possible and should be repeated at 3-year to 5-year intervals. The last survey done on this reservoir was in 1988.

The application of the sedimentation model for non-uniform sediment show that the model is suitable for simulating morphological changes in the reservoir. However, the model can be improved to increase the accuracy of predicted results through the following work:

- Sedimentation on flood plains results from sediment exchange due to convection and dispersion at the interface between the main channel and the flood plains. It will be necessary to incorporate the sediment dispersion-diffusion formula in the source code. With this formula, experiments can be performed to determine dispersion coefficient between main channel and flood plain in order to quantify the exchange of sediment at the main channel and flood plain interface.
- Frictional resistance will vary with composition changes in bed material during sorting procedure. Resistance to flow influences the calculated water surface level which in turn influences the sediment transport simulation. It is essential that provision is made in the model to have a variety of friction resistance values used where the bed material is being sorted, coarsened or fined.
- Studies should be carried out to find the possibilities of modelling the phenomena of secondary current, diffusion, and turbidity. Incorporation of these phenomena in the model would greatly improve the prediction results of sedimentation in the reservoir. Secondary and eddy currents play an important role in the sedimentation process at confluence of two arms of reservoir and near the dam wall. Diffusion and turbidity phenomena have great influence on sedimentation in the reservoir as a whole, especially when fine sediments are involved.

The model results could possibly be improved by using measured bed load transport data. In this research, only suspended sediment was considered. However, in natural rivers like Tana, bed load component is significant and selective transport has an extreme effect on non-uniform sediment transport process and changes in reservoir morphology.

The present work has probably made a useful contribution to theoretical integrated approach on soil erosion from Masinga catchment, sediment transport in channels, and sedimentation in Masinga reservoir. It is necessary to extend this study to other integral catchment-river-reservoir units in other parts of the country with different topographic, soil, climatic, and geologic characteristics.
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