MATHEMATICAL MODELS FOR SANDBAG CHECKDAM

BY

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CERTIFICATION

This Thesis has been read and approved as meeting the requirements of the Postgraduate School, Federal University of Technology, Owerri, for the award of Doctor of Philosophy (Ph.D) Degree in Civil Engineering (Water Resources Engineering) to Osuagwu, Joachim Chinonyerem (Reg No: 20064552168)

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DEDICATION

Dedicated to my wife, Mrs. Pauline Osuagwu

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ABSTRACT

The study was aimed at developing mathematical models on the hydraulic performance of Sandbags as Checkdam for erosion control. Pilot studies were carried out at two sites. Site 'A' was a natural gully at a location in Ezinihitte-Mbaise, Imo State while site 'B' was an artificial channel excavated on the slope of Otamiri River at Federal University of Technology, Owerri. Artificial runoff was simulated into site 'B' by pumping water from Otamiri River. Sandbags of various patterns and heights were placed across the gullies and hydraulic effects upstream recorded. Two mathematical models were developed based on material balance and regression analysis respectively. Model 1 relates sediment concentration (C) with flow rate (Q) and rate of storage (S) while Model 2 relates height of sandbag (h_s) with a dimensionless parameter S_f (storage factor). Model verification indicated a high level of correlation between measured and predicted values of the variables. The coefficients of correlation (R^2) were computed as 0.964 and 0.966 for models 1 and 2 respectively. Similarly standard errors of estimates (Se) were 0.00384 and 0.00196 respectively. Comparative studies on patterns of placement of sandbags showed that transverse arrangement yielded better results compared to longitudinal pattern. Based on capital costs, sandbags are comparatively cheaper than conventional concrete check dams (average cost ratio =1:12). However, a major limitation lies in its poor durability leading to increased maintenance costs for long term application. The

mathematical and graphical relationships obtained in this study will be useful in the design and sustainable use of sandbags for erosion control.

TABLE OF CONTENTS

CHAPTER FIVE: CONCLUSION AND RECOMMENDATIONS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

Erosion is a three phase process consisting of detachment of individual soil particles from the soil mass and their transportation by erosion agents (example, wind and water) with subsequent deposition of the related sediments into land depressions, as influenced by natural (geologic soil erosion) or human (accelerated soil erosion) activities (Hundson, 1981). Soil erosion is a major environmental threat to sustainability and productive capacity of Agriculture. During the last 40years, nearly one-third of the Worlds arable land has been lost to erosion and continues to be lost at a rate more than 10million ha/year. With the addition of a quarter of a million people each day, the World is increasing at a time when per capita food production is beginning to decline. (Penitel and others, 1975).

Erosion is a major ecological problem in various parts of the World. In United States, Western Iowa region has a reputation for big sediment loads in streams and severe gully erosion problems. Estimates indicate that 5,000 to 10,000 acres of potential crop land are lost or removed from production annually as a result of gully growth (Bettis, 1983).Large amounts of time and money are spent on maintaining drainage ditches and stream channels which become choked with sediment eroded from gullies. Bridge failures resulting from gully widening are also common in Western Iowa. Numerous other problems directly or indirectly associated with the growth of gullies plague residents in this region.

Erosion also poses a serious threat in different parts of Nigeria especially in the South East region due to high rainfall intensity. The presence of gully sites in the area are some of the hazard features that characterize this zone as well as other zones that adjoin them (Ofomata, 1985). Asiabaka and Boers (1988) had estimated that over 1970 gully sites exist in Imo and Abia states. The rapid retreat of gully head scraps (at an average rate of 40 m per year), and extensive slumping/sliding on the walls of existing gullies is gradually destroying the physical base of many rural and urban communities in the region. (Olarewaju, 2000). In many states of the country, erosion has resulted in several environmental hazards such as disruptions of drains, and roads (Eze-Uzoamaka, 1991). Loss of Agricultural productivity, siltation and washing away of pollutants into river courses. It was estimated that 3,000,000 tons of sediments are retained annually in Kainji reservoir (Oyebande and Martins, 1986). Some states in North East Nigeria also experience problem of erosion. A Study in Biu, Borno state revealed that over 1,000 tonnes of soil per hectare were lost to gully erosion alone (Ekwue and Aliyu, 1990), while estimated loss by gully erosion of 31,000 tonnes soil per hectare in Sade town, Bauchi state (Rattenbury and others, 1988). Mubi and its environs in Adamawa state is one of the most affected areas in the North Eastern Nigeria where large farmlands have been lost to soil erosion (Tekwa and Usman, 2006).

It is disturbing that despite the danger posed by erosion, the problem is not being highlighted as other problems like global warming. This has led to lack of coordinated long term approaches in solving the problem especially in less developed countries. For instance, in Nigeria, erosion control stud ies and design are undertaken independently by various agencies at different levels of government.

This sectional and uncoordinated approach has not helped in addressing the problem. Emphasis has been placed on control of gully expansion and reclamation with little or no attention to prevention. There is absence of a strategic and integrated proactive approach towards erosion control (Osuagwu, 2006). Emphasis has been wrongly placed on control rather than on prevention. Each year, the Nigerian Government spends billions of Naira on provision of structures for Gully control, (Ijioma, 2009). Conventional concrete works used in most erosion control works have failed to yield optimum results due to poor design and construction coupled with inadequate maintenance. The use of heavy concrete structures for erosion control is very expensive.

There is therefore the need to adopt alternative methods that are cost effective. Some of these non conventional and traditional methods control development of gullies at the early stage. The methods require little skill and technicality to adopt as compared to heavy concrete structures. Among these methods include the use of sand bags, which are basically units of bags filled with sand and placed across flow directions in natural channels to reduce runoff velocity and encourage siltation upstream. Sandbags are randomly adopted by rural inhabitants in Nigeria to reduce the erosive power of runoff. There is however absence of standards and specifications for its optimal application. It is therefore necessary to develop analytical concepts based on principles of hydrodynamics that would optimize the use of these materials.

1.2 STATEMENT OF PROBLEM

Erosion menace poses a serious problem to mankind. As the World is battling with other problems like pollution resulting in global warming, ocean surge and desert encroachment, the problem of erosion also deserves attention. Human activities have continuously increased the scope of the problem. Huge sums of money that would be used for social and infrastructural development are being channeled to erosion control works. Conventional methods of controlling erosion being adopted have not achieved the desired results. The current urge is to import erosion control materials being advertised by western firms. However, considering the availability of natural materials that could be employed for erosion control, the question arises as to the desirability of spending scarce foreign exchange on importation of these materials.

The major limitation of the application of some of these traditional and unconventional methods like sandbags is that in most cases they are applied randomly without specifications .There is therefore a problem of formulating analytic concepts based on principles of Mathematics and Hydraulic Engineering on the optimal use of sandbags for erosion control.

1.2 0BJECTIVES OF STUDY

The objectives of the study include;

(i) To conduct pilot studies on the hydraulic performance of sandbags as erosion control Chekdam.

(ii) To develop mathematical models that relate flow parameters with sediment accumulation rate and concentration of sediments upstream of sandbag location in a gully.

(iii) To calibrate and verify the solution to the mathematical models using field data.

(iv) To develop a design approach and specifications for application of Sandbag Chckdams in gully erosion control.

1.4 JUSTIFICATION OF STUDY:

The Social and economic consequences of Erosion are enormous. The study is geared towards providing a cost effective approach to addressing the problem at the early stage of development of the gullies. The problem of climatic changes resulting from global warming would increase rainfall intensities and worsen the problem of erosion in the years ahead. This study would therefore complement other efforts being made to combat the adverse effects of global climatic changes.

The results shall equip stakeholders in erosion control including Engineers and Environmentalists with analytical concepts for effective application of sandbags. The various government agencies involved in erosion control are expected to apply the results of this study in their erosion control campaigns. Non Governmental Organisations (NGOs) working with global agencies on environmental protection will also find the results of this study significant since it offers a community based approach for combating gully erosion.

1.5 SCOPE OF STUDY:

The study focused on the use of sandbags as checkdam for gully erosion control. Gullies at early stages of development were considered. The general causes and nature of development of erosion gullies were overviewed in order to have a clear understanding of the mode of development of gullies. The hydraulic performance of the sandbags and their limitations were investigated through pilot studies. The investigations focused on pattern and rate of settlement of sediments upstream. The effects of the hydraulic parameters including flow rate, sediment concentration, and height of sandbags, gully cross section and bed slopes were analyzed leading to formulation of models.

Field Studies were carried out at two sites (‘A’ and ‘B’) in Imo State, South Eastern Nigeria (an erosion prone area of Nigeria). The area is located between latitudes 40^0 45′N and 70^0 15′N and longitudes 60^0 50′E and 70^0 25′E. See Location Map (Appendix 1a.)

1.6 LIMITATIONS

The major constraint of this study was absence of basic equipment and materials for some of the experiments. Much time and resources were spent in sourcing for the equipment and materials for field and laboratory investigations. Unavailability of research grants was also a major handicap that limited the scope of data collection. A broader spectrum of data would no doubt have given a better set of results. Inadequate Power supply for computer work was another problem we battled with in plotting the graphs and preparing the report.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Several studies have been carried out on types, causes and effects of erosion. Many research works have also focused on control measures including development of models for prediction of development of erosion gullies. Some studies have been carried out on conventional control structures. Efforts have also been made in producing synthetic erosion control materials. Literatures on these products are available from the manufacturers. Although, there have been some reports on the use of natural materials for erosion control, much work has not been done with regard to optimization of the use of these materials through engineering design. This literature review focuses on types, causes, effects of erosion and control measures. It also discusses optimization and modeling techniques, and their application in erosion control.

2.2 EROSION

The word “Erosion” is derived from the Latin word “erosion” meaning ‘to gnaw’ away. In general terms, Soil Erosion implies the physical removal of topsoil by various agents, including falling raindrops, water flow over and through the soil profile, wind and gravitational force. It is defined as the wearing away of land surfaces by running water, wind, ice or other geological agents including such processes as gravitational creep (SCSA,1982).

According to Wikipedia Encyclopedia (2009), erosion is the displacement of solids, soils, rocks and other particles usually by the agents of current such as wind or ice by downward or down slope movement in response to gravity or by living organisms. It is a three-phase process consisting of detachment of individual soil particles from the soil mass and their transportation by erosion agents. (e.g. wind and water) with subsequent deposition of the eroded sediments into depressions as influenced by natural (geologic soil erosion) or human (accelerated soil erosion) activities. With water erosion, the progressive concentration of surface runoff starts with sheet erosion (the washing of the surface soil from grabble lands) then rill erosion as the water concentrates into small rivulets in the fields, then gully erosion, which occurs when eroded channels are larger (Hudson 1981).

Erosion is distinguished from weathering, which is the process of chemical or physical breakdown of the minerals in the rock, although the two processes may be concurrent. It is an important natural process resulting in the redistribution of the products of geological weathering and is part of both soil formation and soil loss.

In some places, erosion occurs so rapidly that anyone can see it happen. In most places, however erosion is more subtle. In this case, it is a creeping disaster that occurs in small increments, the damage often resulting being more in terms of remedial actions needed and impact to operational activities than in implementation of appropriate control measures.

2.3 TYPES OF EROSION

The two main types of erosion are geological erosion and erosion from human or animal activities. Geological erosion includes soil forming as well as soil eroding process that maintain the soil in a favourable balance suitable for the growth of most plants. Human or animal induced erosion includes a breakdown of soil aggregate and accelerated removal of organic and mineral particles resulting from tillage and removal of natural vegetation.

Water erosion is the detachment and transport of soil from the land by water including run off from the melted snows and ice. In the SOTER methodology (FAO, 1993), the following types of Water erosion are recognized:

i) Sheet,

ii) Rill,

iii) Gully and

IV) Tunnel Erosion.

Fig 2.1 illustrates the different types of erosion based on the causative agents according to Ratan (1990). Three main classes of erosion are identified as follows:

i) Erosion caused by Wind,

ii) Erosion caused by fluids; including water and glaciated erosion,

iii) Erosion caused by gravitational force; including Falls, Slides, debris flow and creep.

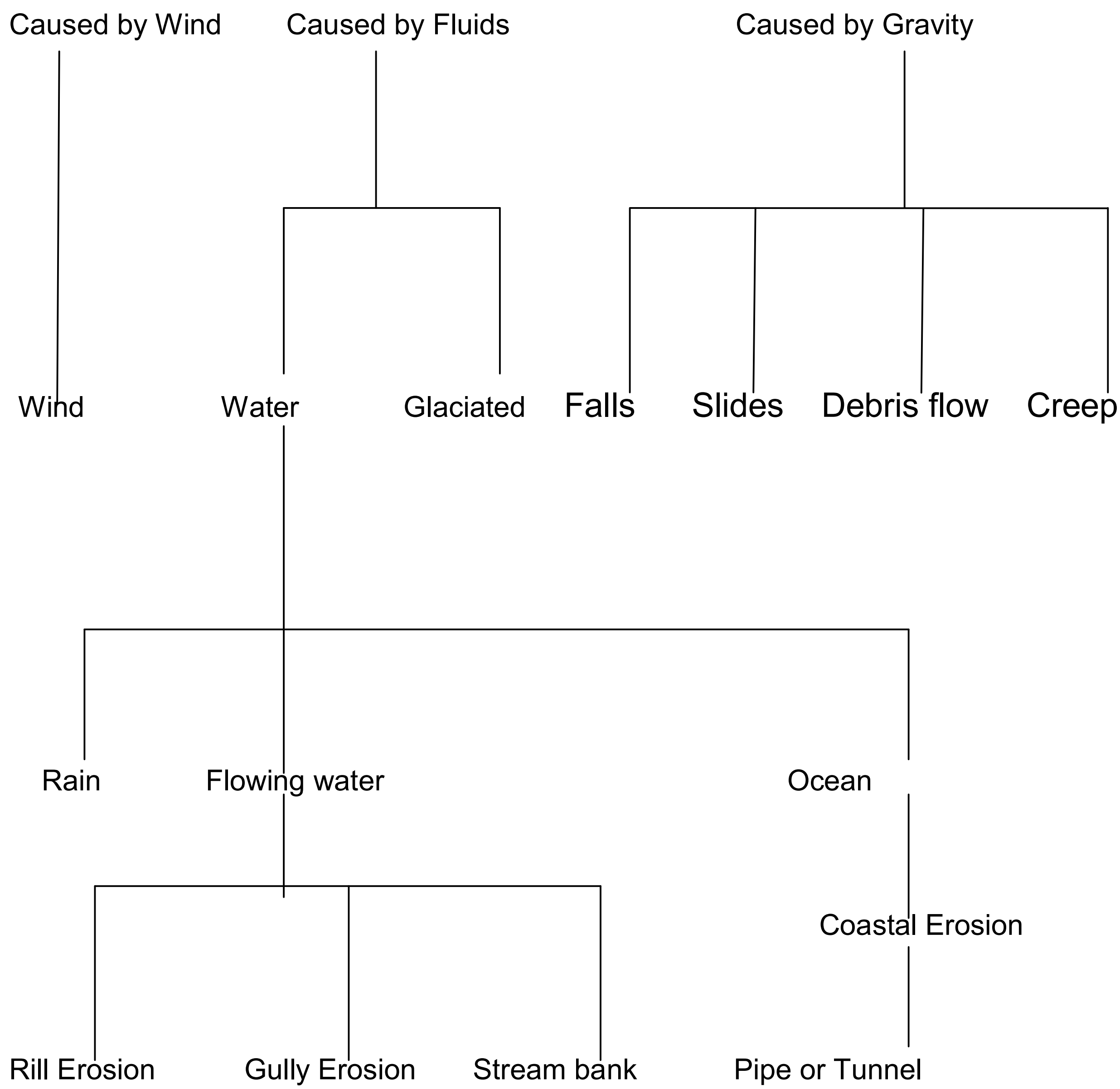

Caused by Wind
Caused by Fluids
Caused by Gravity
Wind
Water
Glaciated
Falls
Slides
Debris flow
Creep
Rain
Flowing water
Ocean
Coastal Erosion
Rill Erosion
Gully Erosion
Stream bank
Pipe or Tunnel

Fig.2.1. Types of Erosion based on Causative Agents (Ratan, 1990)

2.3.1 Raindrop Erosion

Raindrop erosion is the detachment and transportation of soil resulting from the impact of water drops on soil particles or on thin water surfaces. Although, the impact of raindrops on shallow streams may not splash soil, it does increase turbulence, providing a greater sediment carrying capacity. Tremendous qualities of soil are splashed into the air, most particles more than once. The amount of soil splashed into the air as indicated by the splash losses from elevated pairs was found to be 50 to 90 times greater than runoff losses. On bare soil, it is estimated that as much as 200mg/ha is splashed into the air by heavy rains. The relationship among erosion, rainfall momentum and energy is determined by the raindrop mass size distribution, shape, velocity and direction.

Foster and others (1981) had found the relationship between rainfall intensity and energy to be,

$$E = 0.119 + 0.0873 log_{10} i \qquad 2.1$$

Where E = kinetic energy in MJ/ha – mm

i = intensity of rainfall in mm/h

The factors affecting the direction and distance of soil splash include slope, wind, surface condition and impediments to splash as vegetative cover and mulches. On sloping land, the splash move further downhill than uphill, not only become the soil particles travel further downhill than uphill but also become the angel of impact causes the splash reaction to be in a downhill direction. Component of wind velocity up or down the slope have an important effect on the soil movement by splash. Surface roughness and impediments to splash tend to counteract the effects of slope and winds. Contour furrows and ridges break up the slope and cause more of the soil to be splashed uphill. If raindrops fall on crop residue or growing plants, the energy is absorbed and thus soil splashed is reduced. Raindrop impact on bare soil not only causes splash also decreases aggregation and causes deterioration of soil structure.

Okereke and Osuji (1990) carried out a study on the effect of raindrop size and its impact energy on soil splash. The result showed that there is linear relationship between amount of soil and drop size for the three soil types. The greatest splash was observed in sandy soil followed by sandy loam. The rain impact energy increases with increase in rainfall intensity accordingly relates to soil splash exponentially with actual amount of soil splash being dependent on soil type. The results of their studies with regard to the effects of drop size, rainfall impact energy, moisture content variation and runoff/soil loss are presented in Table 2.1

Table 2.1 Effect of Raindrop size on Soil Splash:

Mm/hr	Mean drop size(mm)	Splashed Soil (gms)		
		SS	SCL	SL
25	3.48	18.75	9.42	12.90
50	3.74	57.96	20.10	46.57
75	3.95	87.86	45.27	73.68
100	4.00	119.73	70.12	102.63

SS = Sandy Soil, SCl = Sandy Clay, Sl =Sandy loam.

Source: Okereke and Osuji (1990)

2.3.2 Sheet Erosion

Sheet Erosion is the erosion of the surface of the soil layers over a large area (Strohbach, 2000).The idealized concept of sheet erosion was the uniform removal of thin layers from sloping land, resulting from sheet or overland flow. Fundamental studies of the mechanism of erosion, in which both time-lapse and high speed photographic techniques have been used, indicate that thin idealized form of erosion rarely occurs. Minute rilling takes place almost simultaneously with the first detachment and movement of soil particles. The constant meander and change of position of these microscopic rills obscure their presence from normal observation, hence establishing a false concept of sheet erosion.

The beating action of raindrops combined with surface flow causes initial microscopic riling. Raindrops detach the soil particles, and the detached sediment can reduce the infiltration rate by sealing the soil pores. The eroding and transporting power of sheet flow is a function of rainfall intensity, infiltration rate, and field slope for a given size, shape and density of soil particle aggregate.

According to Haagsma (1992), sheet erosion which is often referred to as erosion on agricultural land does not lead to serious forms of rill and gully erosion at the field level to any sizeable extent. Klerk (1988) measured the sediment yield of two catchment areas in the northern part of Imo State. The results showed values for the sediment yields corresponding with values for average erosion losses per ha per year at about 1 ton.

2.3.3 Rill Erosion

Rill Erosion is the removal of Soil by concentrated water running through little streamlets or headcuts. It is the detachment of earth materials from the surface. Once detached, agents like water or wind transport the materials to new locations where they are loacted (Ritter, 2003). Rills are small enough to be removed by normal tillage operations. Rill erosion is the predominant form of erosion in most cases. It is most serious where intense storms occur on soils with high runoff producing characteristics and highly erodible topsoil.

Tayfur and Kaavas (1994) observed that many land surfaces, on which shallow flows occur, contain not only irregular micro topography but also rills. Flows over such surface occur in both rill and interrill areas (see Fig 2.2). Runoff over hill slopes or agricultural watersheds initially starts as sheet flow, and then it concentrates into series of small channels. The flow concentrations are due to other topographic irregularities or differences in soil erodibility. Such erosion formed micro channels are called rills.

The importance of rills on flow dynamic and sediment transport have been well observed experimentally in field and laboratory studies. Tayfur and Kaavas (1994) also reported results of studies by Meyer on the influence of riling in determining the source of eroded soil in agricultural plots. They observed that there was a significant increase in sediment loss due to presence of rills and also documented that the transport capacity of the rill flow is much greater than that of sheet flow over interrill areas (that is, soil loss increases three to five times when rill develops on a surface.

Fig 2.2 Overland Flow Domain Over Hillslope.

Madubuike and Obiechefu (2006) in a study on the role of undercutting of banks in the collapse and evolution of small channels, observed that undercutting of banks develop in rill channels by reverse roller effect of plunging water jet into scour holes, cutting the underside of head cut wall. The extent of undercutting and the rate of draw down of water into the rill influenced the collapse of the rill wall. Results of the studies showed that slumping was the predominant mode of failure of rill banks.

2.3.4 Gully Erosion

According to Bettis (1983), a gully is a relatively deep, vertical walled channel, recently formed within a valley where no well-defined channel previously existed. Gully erosion produces channels larger than rills. These channels carry water during and immediately after rains, and as clearly distinguished from rills, gullies cannot be obliterated by tillage. The amount of sediment from gully erosion is usually less than that from upland area, but the nuisance of having fields divided by large gullies has been the major problem. In tropical areas, gully growth following deforestation and cultivation has led to severe problems from soil loss and danger to buildings, roads and airports (Aneke, 1985). Some gullies are several kilometers long while other are as short as 30m and have nearly vertical walls. Gullies in large valleys contain streams which usually flow year round but streams in most gullies are dry during portions of the year.

Gullies develop because of decrease in the erosional resistance of the land surface or an increase in the erosional force acting on the land surface. What causes gullies to form, when and where they do is poorly understood. Field and laboratory studies indicate that certain reaches of a valley are more prone to gully development than other. However, the timing of the initial down cutting and which of the “most probable” reaches develops into a gully cannot be predicted with certainty. Once a gully has formed, the processes whereby it lengthens and widens are much better understood. The upper end of a gully is marked by a headwall, a vertical scalp separating the ungullied portion of the valley floor from the gully below. Water flows over the headwall during runoff and falls into a plunge pool at the base of the headwall. The water then erodes the base and sides of the pool, under cutting the headwall. When under cutting reaches an advanced stage, the headwall fails and topples into the gully, thereby lengthening the trench. This process is repeated many times as a gully advances up the drainage way. When first formed, most gullies are quite narrow and have vertical sidewalls. Increased pore pressure from groundwater moving toward the gully, coupled with some under cutting of the side walls causes deep enough water is flowing through the gully to carry away the slumped material, additional slumping can occur. This causes the gully to widen widening also occurs when upper portions of gully walls separate and topple into the gully (Bettis, 1983).

According Bradford and others (1973), the rate of gully erosion depends primarily on the runoff-producing characteristics of the watershed; the drainage area, soil characteristics, the alignment, size and shape of the gully and the slope of the channel. They` stated that a gully develops by processes that take place either simultaneously or during different periods of growth. The processes are:

(i) Waterfall erosion or head cutting

(ii) Erosion caused by water flowing through the gully or by the raindrop splash on exposed gully sides

(iii) Alternate freezing of the exposed banks

(iv) Slides or mass movement of soil into the gully.

Evaluation and prediction of gully development are difficult because the factors are not well defined and field records of gullying are inadequate.

From aerial photographs and field topographic surveys, Beer and Johnson (1963) developed a prediction equation for the deep losses region runoff western Iowa region based on watershed runoff characteristics and soil properties. Gully formation was found to depend on soil shear strength, infiltration, and depth of water table (Bradford and others, 1973). In many cases, an impending layer resulted in saturated soil conditions at the floor of the gully. The saturated soils tended to be weak, leading to under cutting and side sloughing. Runoff from subsequent storms would then remove loose soil from the gully floor.

A report by Haagsma (1992) on study of eleven erosion sites in Akwa Ibom, South- South Nigeria, revealed that all the gullies could be ascribed to road construction activities. Inadequate drainage provision (culverts and drain outlets) and the poor implementation of designs concentrating runoff on road surfaces and compounds to single outlet points has caused and still causes enormous gullies. Gully erosion is caused when run-off concentrates and flows at a velocity sufficient to detach and transport soil particles. A waterfall may form, with run-off picking up energy as it plunges over the gully head. Splashback at the base of the gully head erodes the subsoil and the gully eats its way up the slope.

According to Carrey (2006), gullies may develop in water courses or other places where run-off concentrates. In cultivation or pastures, advanced rill erosion can develop into gully erosion if no protective measures are taken. Cattle pads can be a starting point for a small rill that can develop into a large gully.

A watercourse is ordinarily in a state of balance where its size, shape and gradient are suitable for the flows it carries. If the balance is disturbed, for example by larger than normal flows, gully formation may begin. Gullies generally create for more capacity than they need to accommodate the run-off they are likely to carry. Widening of the gully sides may occur by slumping and mass movement especially on the outside curve of meanders. Scouring of the toe slope can lead to mass failure of the side of the gully under gravity. This soil is then washed away by subsequent flows.

Active gully sides are usually vertical but may adopt an oblique shape once they start to stabilize. This process may occur naturally but can be hastened by the adoption of various gully treatment measures.

Run-off may enter a gully from the sides, causing in a 'badlands' effect. The gully floor may be subject to further down-cutting as secondary gullies advance up the channel. Sediment deposition below gully heads results in a "steps and stairs" pattern. While peak flows from intense rainfall causes considerable gully erosion, the prolonged low flows resulting from an extended wet period can also create problems. Constant trickle flows through a drainage line can saturate the soil in the trickle zone making it structurally weak and very susceptible to erosion. The constant wet conditions may also weaken the vegetation which then provides less resistance to erosion (Carey, 2006).

Gully depth is often limited by the depth to the underlying rock.

Carey (2006) observed that gully development may be triggered by:

(i) Cultivation or grazing on soil susceptible to gully erosion

(ii) Increased run-off from land use changes such as tree clearing in a catchment or construction of new residential areas

(iii) Run-off concentration caused by furrows, contour banks, waterways, dam bywashes, stock pads, fences, tracks or roads.

(iv) Improper design, construction or maintenance of waterways in cropping areas.

(v) Poor vegetative cover, example from overgrazing, Fires or salinity problems

(vi) Low flows or seepage flows over a long period.

(vii) 'Down-cutting' in a creek (causes gullies to advance up the drainage lines flowing into it).

.(viii) Diversion of a drainage line to an area of high risk to erosion example, a steep creek bank or highly erodible soils.

Soil with dispersible subsoils are very vulnerable to gully erosion when the top soil is disturbed. When saturated by seepage flows or subjected to splash back, these subsoil will slump, leaving the topsoil unsupported. The topsoil then collapses and the process is repeated.

Sub surface flow in dispersible soils can cause the saturation of gully sides leading to slumping of gully walls and the expansion of the gully. Under these circumstance gully head advancement can occur with little or no surface flow. The slumping process is comparable to the way a hole dug to the depth of the water table in the sand at the beach expands as the sides slump away.

2.3.5.Tunnelling

Tunneling is an important mechanism for headward and lateral gully expansion in dispersible soils. When dispersible subsoils become exposed, the gradient for water flow through cracks in the soil is increased causing more rapid seepage water flow and crack enlargement by tunnel erosion. The enlarged cracks develop into tunnels which carry a suspension of soil and water. The tunnels soon collapse causing rapid progression on the gully head (Carey,2006).

2.4 FACTORS CONTRIBUTING TO EROSION

Water erosion occurs when raindrops hit the ground and dislodge soil particles from the soil, and then these dislodged soil particles wash away and in the process dislodge and remove further soil particles.

The amount of erosion is thus a function of the following four factors: the rainfall energy, the vegetative cover, the length and steepness of the slope and the type of soil (Stocking, 1987).

Ratan (1990) presented a detailed illustration of the factors affecting Soil erosion including active and passive factors (Fig 2.3). According to him, the active factors are related to land use while the passive factors are related to climate, soil type, hydrology and land forms.

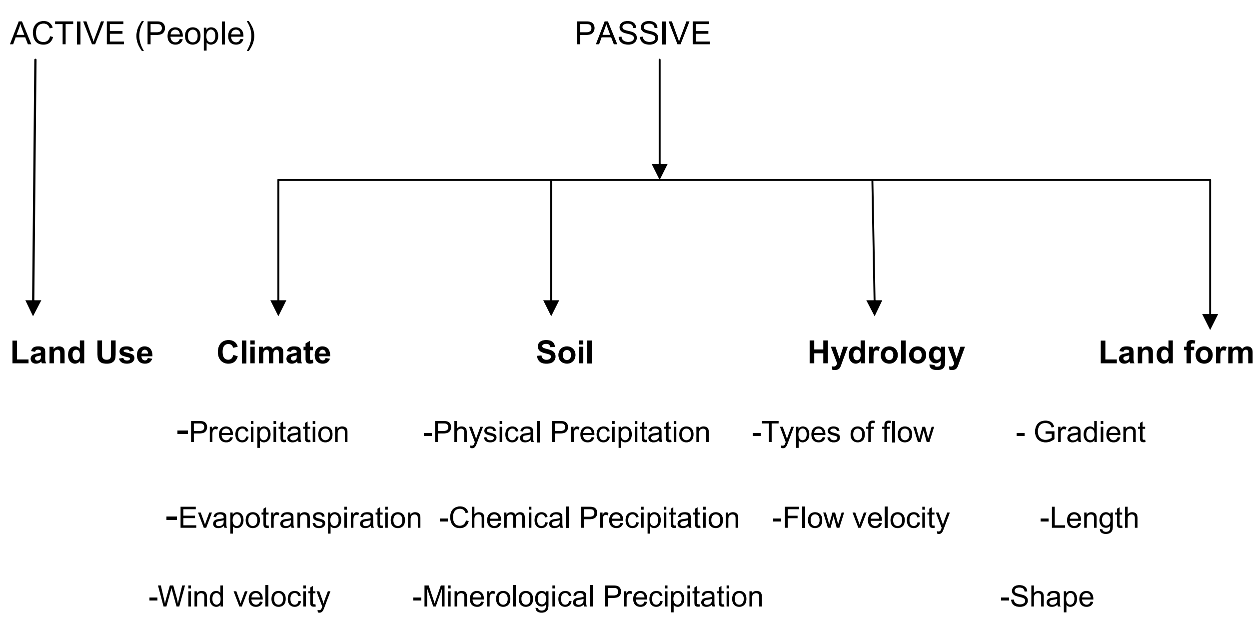


Fig.2.3 Factors Affecting Soil Erosion (Ratan,1990)

2.4.1 Rainfall energy:

The rainfall energy is the energy which falling rain drops have when impacting with the soil (Strohbach, 2000). This energy is a product of the mass (that is, the size) of the drop as well as the speed at impact. The higher the origin of the drop the higher its impact speed. The bigger the drops and thus the “harder” the rainfall event, the more energy is released to the soil (Stocking, 1987).

Erosivity is the potential ability of rain to cause erosion. It is a function of the physical characteristics of rainfall. Wischeimer and Smith (1978), found the best correlation of soil erosion with rainfall data when using EI_{30}, the product of kinetic energy of a storm and its intensity. The kinetic energy (E), in foot-ponds acre reflects the greater effect of larger drops, which impact with a higher terminal velocity. The maximum 30minutes intensity (I_{30}) is the intensity in inches/hr for the 3ominutes period of greatest intensity as obtained from recording rain gauges.

Outside the USA, this index has been found to give poor correlation, particularly in the tropics. In southern Africa, Hundson (1971) found that a simpler index E = 1, calculated by summing the kinetic energy of rain falling at an intensity > 1 in h^{-1} for a storm gave a better correlation. The main difficulty in formulating an effective erosivity index is the paucity of rainfall intensity records, let alone data on drop size / velocity and hence kinetic energy, However, Roose (1977) found that a good correlation held for West Africa with an erosivity index derived only from rainfall amounts, and a recent technique for estimating R from daily rainfall amounts in the USA is given by Richardson and others (1983).

2.4.2 Slope length and Steepness:

After dislodging the soil particles from the soil surface, these particles have to be transported in order for erosion to take place.(Strohbach, 2000). Gravity is the driving force. The steeper the slope, the faster the water can move. The faster water moves, the more soil particles it can take along and the more additional soil particles can be dislodged. Obstructions along the slope will impede the flow of water and reduce the amount of soil particles the flowing water takes along. The longer the (uninterrupted) slope, the more the erosion caused by flowing water.

The effect of slope gradient is more pronounced in the tropics (Hundson, 1989). Studies by Igbokwe and others (2008) revealed that gully developments are more pronounced in areas with high terrain undulation. In these areas, the slopes of the ground are steep and vary. This inevitably results in increase in the speed and volume of overland flow and subsequently the rate of detachment and transportation of soil particles.

2.4.3 Soil Type

Physical properties of soil affect the infiltration capacity and the extent to which particles can be detached and transported. The corresponding soil characteristic that describe the ease with which soil particles may be eroded are soil detachability increase as the size of soil particles or aggregate size, that is, clay particles are more difficult to detach their sand, but clay is more easily transported. The properties that influence erosion include soil structure texture, organic matter, water content, clay mineralogy and density or compactness, as well as chemical and biological characteristic of the soil.

The soil type and especially the chemical properties of the soil type, determine the ease with which soils are renewed from the soil body (Strohbach, 2000). A major factor is the ease with which water is absorbed into the soil,as well as the bonding of the soil particles.

Soil erodibility factor (k) is an inherent property of the soil, usually applied to a soil series and is independent of the effects of management (Landon,1996). Wischeimer and others (1971) developed nomographs for determining k value from known values of :

(a) Percentage silt and very fine sand (0.002-0.01mm)

(b) Percentage sand (0.1-2.0mm)

(c) Organic matter content

(d) Topsoil structure grade

(e) Permeability grade

The factor quantifies the susceptibility of soil to erosion as affected by infiltration capacity and structural stability.

2.5 EFFECTS OF EROSION

Pineo and Barton (2009) outlined the effects of erosion as follows:

(i) Disturbance of Fragile Aquatic and Wetland Eco-systems.

Sediment clouds the water and reduces the ability of underwater plants and animals to get the light they require for survival. The introduction of high nutrient levels from runoff fertilizer causes dense algae growth which removes oxygen from the water and exacerbates the problem of inadequate sunlight.

(ii) Degradation of Water Quality

Erosion washes away the topsoil, the nutrient-rich surface layer that supports all plants, beneficial organisms, and human populations. National geographic reports that the world's six billion people depend on only 11% of world's land for all its food need and only 3% of the planets soil are still fertile making erosion control essential.

(iii) Increase in Maintenance Costs of Stormwater Management

Sediments clog storm sewers, making them less effective and prone to overflowing. Increased maintenance translates to increased costs.

(iv) Sedimentation of Waterways

Sedimentation (settling of eroded particles) makes waterways shallower. Collapsed banks undercut by erosion make them narrower. Shallow and narrow waterways disrupt water traffic reducing economic and recreational opportunities. Dredging to ease this problem wreaks havoc on aquatic and animal communities,

(v) Pollution of Water bodies

As the soil washes away, it carries with it a myriad of pollutants ranging from fertilizers and pesticides to oils, heavy metals, chemicals and animal wastes. This mixture ends up in our natural water bodies with great costs to the environment as well as to public health.

2.6 PREVENTIVE MEASURES

Carey (2006) identified a range of measures to prevent the development of gullies as follows:

(i) Property development

(a) Manage catchments to ensure run-off is not increased.

(b) Assess the land's capability to ensure it is suitable for the proposed use.

(c) Locate and construct roads, fences and laneways so that they cause minimal concentration and diversion of run-off.

(ii) Grazing management

(a) Maintain adequate pasture cover by better stock management.

(b) Be prepared to fence off and exclude stock from land vulnerable to gully erosion

(c) Locate watering points, stockyards, shade areas and gates away from gully-prone areas.

Contour banks are normally not necessary on well managed pasture land. Poorly maintained banks in such situations may lead to rills and gullies. An option is to level them or create gaps in the banks to safely disperse the run-off.

(iii) Cropping management

(a) Control erosion on sloping, cultivated land by stubble retention and the construction and maintenance of contour banks and waterways.

(b) Construct waterways to appropriate specifications and stabilize and maintain them.

(c) Ensure that contour banks discharge into waterways at state locations.

(d) Spread flood flows on cultivated floodplains and avoid practices that concentrate flood flows.

(i) **Urban development and management**

(a) Avoid the development of bare, compacted areas that may occur in school grounds or other heavily trafficked areas.

(b) Avoid developing steep sites and drainage lines.

(c) Minimise soil disturbance, stockpile and respread topsoil and re-vegetate affected areas

(d) Construct flood detention systems below high run-off areas

2.7 Erosion Control:

According to Haagsma (1992), integrated gully reclamation requires a combination of various measures to be taken in different zones. He recommended the following criteria;

(i) Upstream Area: Canalization of runoff (interceptor ditch or ponding system and increase of infiltration (reforestation and improved cultivation methods).

(ii) Erosion zone: check dams, sills, embankment to avoid bank erosion, gabions.

(iii) On the slope: Terracing /shaping, planting of vegetation, wickerwork fences.

Carey (2006) listed the factors to consider when controlling a gully as follows:

(i) The cause.

(ii) The effect – What will happen if no action is taken?

(iii) Catchment size – The larger the catchment the more complex the problem.

(iv) Soil type – Gullies in fertile soils are easier to control than those in poor soils that are highly erodible

(v) Gully components - Which parts of the gully are most actively eroding? Is it the gully head, the floor or the sides? Does the gully have branches? What is the height of the gully head?

(vi) Potential for diversion – Is there an option to divert runoff flowing into the eroding gully to a safe disposal area?

According to Goldman and others (1991), preparing an erosion and sediment control plan is a four step process as follows:

i. Collect data so as to have information on site conditions.

ii. Analyze data so as to interpret the data collected in step 1.

iii. Develop the site plan in such a way that erosion and sediment control are considered along with such traditional planning criteria as economics, utility areas, and traffic patterns.

iv. Develop erosion and sediment control plan by applying the principles of erosion and sediment control.

2.8 Erosion Control Structures:

2.8.1 Drains

Drains are structures that discharge excess water from land safely to collecting streams. Drains may be classified into municipal drainage, land drainage and highway drainage (Agunwamba, 2001). They may also be classified as natural or manmade. The manmade drains could be subdivided according to materials into concrete and earth (clay lined) channels. According to condition of flow, drainage channels are classified as open or closed. Open channels in which the water/liquid surface is open to atmosphere are most commonly used. Flows in open channels may be in a form whereby the depths and velocity distribution remain constant from section to section or non uniform whereby depth and velocity distribution vary from section to section. Flows could also be steady in which velocity and depth at a given section do not vary with time (Raghunath, 1986).

2.8.1.1 Design of drains

The design of drains involves application of data from hydrological analyses to determine the type, shape and size of channels that will effectively convey the runoff to discharging streams. The design of drainage channels is based on the principle of open channel flow; in which case water with a free surface. As much as possible drains are designed to flow by gravity following the natural topography.

Agunwamba (2001) suggested the use of Chezy formula for calculating velocity of flow:

$$V = C\sqrt{RS_O} \qquad 2.2$$

where,

Hydraulic radius, $R \quad = \quad {}^{A}/_{P}$ 2.3

S_O = channel slope, A = area, P= wetted perimeter.

C is Chezy coefficient and is related to the friction coefficient f in the formula:

$$Friction\ head, H \quad = \frac{flv^2}{2gR} \qquad 2.4$$

$$c^2 \quad = \frac{2g}{f} \qquad 2.5$$

C depends on the nature of the channel and on the hydraulic mean depth. To some extent it also depends upon the mean velocity of the stream. One of the often used formulas for uniform flow in open channels is that published by an Irish Engineer, Robert Manning (Finnemore and Frazini, 2002). Manning had found from many tests that the value of C in the Chezy formula varied approximately as $R^{1/6}$, and also observed that the proportionality factor was very close to the reciprocal of n, the coefficient of roughness in the previously used but complicated and inaccurate Kutter formula. The Manning's formula is as follows;

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad 2.6$$

$$Q = AV \qquad 2.7$$

$$Q = \frac{A}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad 2.8$$

Where, n = manning constant (see Table 2.2), S = channel slope

Table 2.2 Values of n in Mannings formula for open channels.

Nature of surface	Maximum

Lucite	0.010
Glass	0.013
Neat cement surface	0.013
Wood – stave pipe	0.013
Plank flumes, planed	0.014
Vitrified sewer pipe	0.017
Concrete, precast	0.013
Metal flumes, smooth	0.015
Cement mortar surfaces	0.015
Planks flumes, unplanned	0.015
Common – clay drainage tile	0.017
Concrete, monolithic	0.016
Brick with cement mortar	0.017
Cast iron, new	0.017
Riveted steel	0.020
Cement rubble surfaces	0.030
Canals and ditches, smooth earth	0.025
Corrugated metal pipe	0.030

Sourcce: Finnermore and Franzini (2002)

2.8.1.2 Runoff Estimation:

According to Raghunath (1986), runoff is that balance of rainwater, which runs over the natural ground surface after losses by evaporation, interception and infiltration. It is that portion of precipitation that make its way towards stream, canals, lake or oceans as surface flow. The design of channels or structures to handle storm runoffs involves the determination of peak rate of runoff volume, and the time distribution of runoff rates.The factors affecting runoff may be divided into factors associated with precipitation and factors associated with the watershed (catchment area). Precipitation factors include rainfall intensity (i) and distribution of rainfall over an area. On the other hand, watershed factors include size, shape and orientation of the watershed, topography and geology of the watershed area. The runoff from rainfall may be estimated by the following methods:

(i) Rational method

(ii) Empirical formulae, curves and tables

(iii) Infiltration method

(iv) Overland flow hydrograph

(v) Unit hydrograph method

The Rational method is the most commonly used method-predicting runoff. It was first published in Ireland in1851. It is also known as Lloyd – Davies equation (Agunwamba 2001).

The formula is given by:

$$Q = 0.278CiA \qquad 2.9$$

Where Q is the quantity of runoff ($m^3/_s$)

C is the coefficient of runoff (Table 2.3)

i is the rainfall intensity (mm/hr)

A is the catchment area (km^2)

The method is an attempt to obtain the yield of a catchment assuming a suitable runoff coefficient. It assumes a uniform rainfall over the entire watershed. The rainfall duration should be at least equal to the time of concentration.

2.8.1.3 Rainfall Intensity and Duration:

The design rainfall is characterized by its intensity and duration. The rainfall intensity is often read from rainfall intensity duration curves if the time concentration (t_c) and storm return period (T) are known. Rainfall intensity duration area based on rainfall records (1938-1974) for Apapa, Ikeja and Kano as plotted in the Federal Ministry of Works and Housing Highway Manual (1972) are presented in Appendix 1-3. Generally, storms of high intensity last for fairly short periods and cover small areas . Storms covering large are seldom of high intensity but may last for several days. The combinations of relatively high intensity and duration occur infrequently, but when it does occur a large amount of runoff results. In Nigeria especially the South East region, these infrequent storms have resulted in severe gully erosion.

For a given frequency occurrences, rainfall intensity decreases with duration of storm while depth increases with duration. Empirical relationship between rainfall duration and intensity as reported by Agunwamba (2001) indicated that,

$$i = \frac{a}{t + b} \qquad 2.10$$

where a and b are locality constants;

a = 1846, b = 3.6 for Eastern part of Nigeria

t = time of concentration in minutes.

for $t > 12$ mins

Also,

$$i = \frac{C_0}{t^s} \qquad 2.11$$

Typical values of s = 1/2, and C_O = 446

Oyebande and Longe (1990) obtained another empirical formula using Gumbel Extreme value distribution:

$$i = KT^m t^{{n_1}^{-1}} \qquad 2.12$$

where k, m and n_1 are parameters dependent on the regime. T is the return period (yrs); and t is the rainfall duration in hrs.

If t < 1, n_1 is replaced by n_2.

2.8.1.4 Time of Concentration (T_C):

Time of Concentration,(T_C) is the time required for water to flow from the remotest part of the watershed to the outlet. It is the sum of inlet time (t_i) and the flow time (t_f). Many empirical equations have been proposed for quick estimate of time of concentration which is taken as rainfall duration. The most widely used formula is (Agunwamba, 2001):

$$T_c = \frac{0.01947(L)^{0.77}}{S^{0.5}} \qquad 2.13$$

Where

T_C = time of concentration in mins

L = flow path distances in m

S = surface slope

Also, Izzard (1946) developed a formula for computing rainfall intensity:

$$T_C = \frac{526.423bL^{1/3}}{(Ci)^{2/3}} \quad 2.14$$

where,

$$b = \frac{2.8 \times 10^{-5}i + C_r}{s^{1/3}} \quad 2.15$$

C_r = retardance coefficient with values 0.007 for smooth asphalt surface, 0.012 for concrete pavement, 0.017 for tar and gravel pavement and .06 for dense blue grass turf.

2.8.1.5 Runoff coefficient

The coefficient of runoff (C) is a function of the vegetative cover, slope and other factors that affect the rate of infiltration. Values of C for different vegetative covers and slopes as given by the Federal Republic of Nigeria, Ministry of Works Highway Manual (1972) are presented in Table 2.3

Table 2.3. Values of Runoff Coefficient C.

Description	Flat	Rolling 2-10%	Hilly Over 10%
Pavement and Roofs	0.9	0.9	0.9
Earth Shoulders	0.5	0.5	0.5
Drives and Walks	0.75	0.8	0.85
Gravel Pavement	0.5	0.55	0.6
City Business Area	0.8	0.85	0.85
Apartment Dwelling Areas	0.5	0.6	0.7
Suburban, Normal Residential	0.45	0.5	0.55
Dense Residential Sections	0.5	0.65	0.7
Lawns, Sandy Soil	0.1	0.15	0.2
Lawns, Heavy Soil	0.2	0.25	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Turf	0.3	0.3	0.3
Median Areas, Turf	0.25	0.3	0.3
Cultivated Land, Sand and Gravel	0.25	0.3	0.35
Industrial Areas, Heavy	0.6	0.8	0.9
Parks and Cemeteries	0.1	0.15	0.25
Playgrounds	0.2	0.25	0.3
Woodland and Forests	0.1	0.15	0.2

Source: Federal Republic of Nigeria, Federal Ministry of Works Highway Manual (1972)

2.8.2 Checkdams:

Checkdam is a small barrier constructed of rock, gravel layer, sandbags, fiber rolls or reusable products, placed across a constructed swale or drainage ditch (CASQA, 2003). Checkdams reduce the effective slope of channels thereby reducing the velocity of flowing water, allowing sediment to settle and reducing velocity, The flood water which is trapped by the dam is allowed to infiltrate into the soil. The dam also traps soil particles, silt and other sediments and after a period they gully is expected to fill up to the new level set by the dam height. Common types of check dam include concrete sills, wickerwork and embankments. Sandbags though unconventional also function as check dams.

2.8.3 Stilling Basins

Stilling basins are energy dissipaters provided to dissipate excess Kinetic energy in sheep channels. The structure creates a hydraulic jump and connects flow from supercritical to subcritical condition. Most of the water energy is dissipated in hydraulic jump assisted by appurtenances (example, steps, baffle blocks).Basically, the hydraulic design of a stilling basin must ensure a safe dissipation of the flow kinetic energy, to maximize the rate of energy dissipation and to minimize the size (and cost) of the structure (Chanson, 1999). Several standard designs of stilling basins were developed in the 1950's and 1960's. These basins were tested in models and prototypes over a considerable range of operating flow conditions. The prototype conditions are well known and can be selected without further model studies.

2.8.4 Rip–rap:

Rip–rap is a commonly used material for the protection of canals against flow–induced erosion (Christensen, 1995). In some cases, it is hand packed, especially on side slopes which are expected to remain as placed without settlement, but with increasing rise of mechanical equipment, it is more often placed in a random manner. This is also preferable in locations where it is expected to settle or move down due to scour. On the slide slopes, the thickness of rip-rap should be sufficient to accommodate the biggest stones without large gaps – at least .5 times median stone diameter – and an under layer or filter of smaller stone is generally required to prevent the base material from being washed out by wave action. In the case of bed protection surfaces not subject to scour, this may be treated in the same way. But at transitions from stilling basins and in general where the channel bed may scour beneath the apron level, the volume of rip-rap should be sufficient to protect a slope at the angle of repose of the rip-rap on the bed material extending from the apron level to the level of anticipated deepest scour. The rip-rap may be laid on a prepared slope or it may be laid in a horizontal apron which it is assumed will settle to a slope when scour occurs. The assumed thickness of the stone on the slope is that sufficient to prevent removal of the fine materials beneath, under the particular flow conditions, and in estimation of the volume required. A margin should be included to allow for stone carried away in the flow. In some cases, a filter layer of smaller material is provided, but its availability and location after settlement of the apron is uncertain (Blake, 1975).

When available in sufficient size, rock rip-rap is usually the most economical material for bank protection. Rock rip rap has many other advantages over other types of protection. A rip-rap blanket is flexible and is neither impaired nor weakened by slight movement of the bank resulting from settlement. The cost effectiveness of locally available rip-rap provides a viable alternative to many other types of bank protection (Agurrie and Fuentes, 1995).

2.8.5 Gabions:

Gabions are forms of revetment that involve the use of crates of wire mesh filled with stone. They are commonly of 2m x 1m x 1m sizes but longer lengths up to 6m are used (McLeod, 1975). They are boxes subdivided by diaphragms and cells are filled by carefully packing the stone round the faces and filling the centre in a more random fashion. The gabions can be obtained prefabricated in hexagonal or square mesh or can be made up on site from ordinary reinforcement fabric (75mm mesh, 5 guage). The square mesh is of heavier gauge and is easier to fill without distortion. The resultant gabions are less flexible. The gabions are built up like bricks to the required height, each course being stepped back 0.1 – 0.5m behind the course below. The bottom course should stand on a mattress 0.3 or 0.5m thick and extending to form an apron 3 to 4m wide.

Gabion weirs offer advantage in that the structure can be changed in height and size simply by building up or removing courses of gabions on the existing structure. This can be very convenient when control works are required urgently in channels on which the collected hydrological information is meager. After a period of operation, the shape of the structure can be adjusted according to requirements and progressive adjustments can be made (Agostini and others, 1985).

2.8.6 Vegetation

Vegetation refers to plants found in an environment. It comprises grasses, shrubs and other trees. Vegetation engineering makes use of plants to help solve environmental problems associated with flood and erosion. The use of vegetation in civil engineering is known as soil bio-engineering. The term covers a range of applications including surface soil protection slope stabilization, water course and shoreline protection wind breaks, vegetation barriers including noise barriers and visual screens and the ecological enhancement of an area (Coppin and Richards, 1990).

Vegetation was for many years, an unknown quantity in stability analysis and the benefits from strength to roots and increase in root-soil reinforcement was ignored (Greenwood and Norris, 2004). However, in recent times, soil bio-engineering or using vegetation as an engineering structure has become an established practice in many parts of Europe. It is considered a practical alternative to traditional methods of soil stabilization such as soil nailing or geosynthetic reinforcement. In Nigeria and most tropical countries, little information of relevance to Engineers and Environmentalists is known about the 'below the ground' function and properties of various types of vegetation.

2.8.6.1 Engineering Influences of vegetation:

Vegetation performs an important engineering function as it has a direct influence on the soil at both the surface and at depth. The influences are tabulated in Table 2.4. Vegetation provides soil stability and protects the ground from soil erosion.

Table 2.4: Influences of vegetation on the soil.

Surface	**Depth**
Protection against wind erosion	Increased water infiltration
Protection against foot traffic	Water uptake by roots
Protection against raindrop impact	Reinforcement of soil roots
Reduction of surface water runoff	Anchoring and buttressing by taproots
Interception of rainfall	
Protection against erosion by surface water flow	

Source: Coppin and Richards (1990).

Soil stabilization and reinforcement on slopping ground are significant influences that the roots of woody shrubs and trees have on the soil. According to Greenwood and Norris (2004), the main engineering influences are;

(i) Additional effective cohesion due to vegetation.

(ii) Increase in weight of slice due to vegetation.

(iii) Tensile reinforcement force by the roots present on the base of each slice .

(iv) Wind force.

(v) Changes in undrained soil strength due to moisture removal by the vegetation

(vi) Changes in pore water pressure.

Forests and grasses are the best natural soil protective agencies known. Greenwood and Norris (2004) outlined the major effects of vegetation in reducing erosion as follows:

i. Interception of rainfall by absorbing the energy of raindrops and thus reducing surface sealing runoff.

ii. Reservation of soil erosion by decreased surface velocity.

iii. Physical restraint of soil movement

iv. Improvement of aggregation and porosity of the soil by plant roots and plant residue

v. Increased biological activity in the soil.

vi. Transpiration, which decreases soil water, resulting in increased storage capacity and less runoff. These vegetative influences vary with the season, crop degree of maturity of the vegetation, soil and climate, as well as with the kind of vegetative material, namely, roots, plants tops, and plant residues.

2.8.7 Sandbags:

Although, sandbags are most closely associated with flood control, they actually have a number of applications from environmental remediation to forts because they are easy to handle cheap and are economical. A sandbag (flood sack) is a sack made of burlap polypropylene or their materials that is filled with sand or soil and used for such purposes as flood control, military fortification, shielding glass in war and ballast (Wikipedia,2009). Advantages are that burlap and sand are inexpensive and that the bags can be brought empty and filled with local sand or soil.

A sandbag barrier is a series of sand filled bags placed on a level contour to interrupt sheet flows (CASQA, 2003). Sandbag barriers pond sheet flow runoff, allowing sediment to settle out. The carlifornia storm water BMP handbook outlines areas of application as follows:

(a) As a linear sediment control measure:

i. Below the top of slopes and erodible slopes.

ii. As sediment traps at culvert/pipe outlets.

iii. Below other small cleaned areas.

iv. Along the perimeter of a site.

v. Down the slope of exposed soil areas.

vi. Around temporary stockpiles and spoil areas.

vii. Parallel to a roadway to keep sediment off paved areas.

(b) As linear erosion control measure:

i. Along the face and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff round as sheet flow.

ii. At the top of slopes to divert runoff away from disturbed slopes.

iii. At the top of slopes to direct runoff away from disturbed slopes.

iv. As check dams across mildly sloped construction roads.

Fig 2.4 Illustrates a typical application of sandbag barriers as recommended in the California stormwater BMP Handbook (CASQA, 2003).

Fig 2.4. Application of Sandbag Barriers.

Source: CASQA (2003).

2.8.7.1. Design and Layout of Sandbags (CASQA, 2003)

(i) Locate sandbag barriers on a level contour.

-Slopes between 20:1 and 2:1 (H:V): Sandbags should be placed at a maximum interval of 50ft (a closer spacing is more effective), with the first row near the slope toe.

-Slopes 2:1(H:V) or steeper: Sandbags should be placed at a maximum interval of 25ft (a closer spacing is more effective), with the first row near the slope toe

(ii) Turn the ends of the sandbag barrier up slope to prevent runoff from going round the barrier.

(ii) Allow sufficient space up slope from the barrier to allow ponding, and to provide room from sediment storage

(iii) For installation near the toe of the , consider moving the barrier away from the slope toe cleaning. To prevent flow behind the barrier, sandbags can be placed perpendicular to the barrier to serve as cross barriers.

(iv) Drainage area should not exceed 5 acres.

2.9 GULLY RESHAPING AND FILLING

The practicability of shaping a gully depends on its size and the amount of fill needed to restore the gully to its desired shape (Carey, 2006). Steep gully sides can be reshaped. Topsoil should be stockpiled and respread over exposed areas to ensure the rapid establishment of vegetation. Annual crops such as millet (summer), oats or barley (winter) can be used to provide a quick cover. It may be possible to temporarily divert water from the battered gully while grass is generating.

Haagsma (1992) observed that though current engineering approach favours filling of gullies, the results are not very encouraging. Technically, this method may be effective for smaller gullies, if proper compaction is done. For bigger gullies, this method is clearly inappropriate, as proper compaction cannot be achieved and ground water movements are likely to endanger the result, with more serious impact as ever before. Gullies in cultivation can be filled when constructing contour banks. The banks must have sufficient capacity where they cross old gully lines as this is a common site for contour bank failure.

2.10 SEDIMENT PROPERTIES:

The most important property of sediment particles is the characteristic size. It is termed the diameter or sediment size and denoted as d_s. In practice, natural sediment particles are not spherical but exhibit irregular shapes. Several definitions of sediment sizes are available:

i. The sieve diameter

ii. The sediment diameter and the nominal diameter.

The sieve diameter is the size of particle that passes through a square mesh sieve of given size but not through the next sediment sieve, example, $1mm < d_s < 2mm$ (Chanson, 1999).

2.10.1 Sediment Transport:

Sediment transport is the general term used for the transport of material (e.g. silt, sand, gravel, boulders), in rivers and streams (Chanson,1999). The transported material is called the sediment load. Distinction is made between the bed load and the suspended load. The bed load characterizes grains rolling along the bed while suspended load refers to grains maintained in suspension by turbulence.

The sedimentation diameter is the size of a quartz sphere that settles down (in the same fluid) with the same settling velocity as the real sediment particle. The nominal diameter is the size of a sphere of same density and same mass as to actual particle. The sediment size may also be expressed as a function of the sedimentological parameter that is defined as:

$$d_s = 2^{-\emptyset} \qquad 2.15$$

$$\emptyset = \frac{ln(d_s)}{ln(2)} \qquad 2.16$$

where d_s is diameter in mm. A typical sediment size classification by is presented in Table 2.5.

Table 2.5.Sediment size classification.

Class	Size range	Phi-scale ϕ	Remarks
Clay	$d_s < 0.002\ to\ 0.004mm$	$+8\ to + 9 < \phi$	
Silt	$0.002\ to\ .004 < d_s < 0.06mm$	$+4 < \phi < +8\ to + 9$	
Sand	$0.060.06 < d_s < 2.0mm$	-1< $\phi < +4$	Silica
Gravel	2.0< $d_s < 64mm$	$-6 < \phi < -1$	Rock fragments
Cobble	64< $d_s < 256mm$	$-8 < \phi < -6$	Original rocks
Boulder	256< d_s	$\phi < -8$	Original rocks

Source: Chanson (1999

2.10.2 Sediment Concentration:

For suspended sediment, the sediment concentration may be expressed in kg/m^3 and is calculated as the ratio of dry sediment mass to the volume of water – sediment mixture. It can also be expressed as a volume concentration (dimensionless). Another unit, parts per million (ppm) is sometimes used. It is defined as the ratio of the weight of sediment to the weight of the water-sediment mixture times one million (Chanson, 1999).

The conversion relationship is:

Mass concentration $= \rho_s C_s$ 2.17

Where,

ρ_s = sediment density (most natural materials have densities similar to that of quartz with $specific\ gravity\ s =$ 2650kg/m^3).

C_s = sediment concentration in kg/m^3.

Suspended sediment concentrations may be measured from representative samples of the sediment – laden flow. The sampling techniques may be instantaneous sampling-point- sampling or depth – integrated sampling.

2.11 OPTIMIZATION:

Optimization is the act of obtaining the best result under given circumstances (Rao, 1996). It is the process of finding the conditions that give maximum or minimum value of a function. Several optimization methods have been developed for solving different types of optimization problems. This includes:

(i) Mathematical programming techniques useful in finding the minimum of a function of several variables under prescribed set of constants.

(ii) Stochastic process techniques useful in analyzing problems described by a set of random variables having known probability distribution.

(iii) Statistical methods that enable one to analyze the experimental data and build empirical models to obtain the most accurate representation of the physical situation.

A detailed list of different techniques under the above categories is presented in Table 2.6 according to Rao (1996).

Table 2.6: Optimization techniques.

Mathematical Programming Techniques	**Stochastic Process Techniques**	**Statistical Method**
- Calculus Methods - Calculus Variation - Non-linear - Geometric Planning -Quadratic Programming - Linear Programming -Dynamic Programming - Integer Programming -Stochastic Programming -Separable Programming - Network Methods, CPM and PERT. - Game Theory - Simulated Annealing - Genetic Algorithms	-Statistical Decision - Theory - Marker Process - Overing Theory -Simulation Methods - Reliability Theory	- Regression Analysis - Cluster Analysis - Pattern Reoperation - Design of Experiments - Discriminate Analysis - Factor Analysis

Source: Rao (1996)

According to Chu and Forward (1978), the best set of input parameter values for a mathematical model in optimization techniques is usually obtained by minimizing the objective function, that is, the sum of the difference between the observed and the calculated hydrographs.

Mathematically, the problem can be restated as:

$$\sum(Q_O - Q_i)^2 \qquad 2.18$$

with constraints:

lower limit $\leq x \leq$ upper limit for all i.

where, Q_o is the observed hydrograph

Q_i is the calculated hydrograph

x is the input parameter

Since the calculated hydrograph is generated by a mathematical model of highly non-linear nature, the objective function has non-linear characteristics. Therefore, the problem is to minimize a non-linear objective function subject to linear inequality constants. Two types of approaches are now being used in hydrologic engineering field, the 'gradient method' and the 'direct search method'. In either approach, two separate operations are needed in optimization method, determining the direction of the step and the length.

Camp and Pezechk (1988) also developed a Genetic Algorithm (GA) for discrete optimization of composite structures. The procedure conforms to the load and resistance factor design (LRFD) method. The objective function is considered as the cost of the structure. The objectives function is minimized subject to serviceability and strength requirements.

Osuagwu and Agunwamba (2011) derived an optimum cost function for rectangular channels using classical optimization techniques:

where :

$$C_{min} = 2h^2 \propto_4 + (10 \propto_1 + 4k \propto_2 + 2 \propto_3)h - 6kC_O \propto_2 \qquad 2.19$$

$$\alpha_1 = 10^{-6} C_c$$

$$\alpha_2 = 6165x10^{-9} C_r$$

$$\alpha_3 = 10^{-3} C_f$$

$$\alpha_4 = 10^{-6} Ce$$

C_f = unit cost of formwork, ₦/m^2

C_c = unit cost of concrete, ₦/m^3

C_e = unit cost of excavation, ₦/m^3

C_r = unit cost of reinforcement, ₦/ton

C_O = concrete cover, mm

$$k = \frac{M}{f_{cu}bd^2} \qquad 2.19b$$

(M=moment in N/m, b= unit breadth in mm, f_{cu} = concrete cube strength in N/mm^2)

Kinori and Mevorach (1984) plotted a simple optimization procedure for flood control (See Fig. 2.5). Curve (1) represents the annual cost of the project; curve (2), the average annual damage cost; curve (3), indicates the minimum annual cost of the profit and the corresponding design discharge.

Fig 2.5 Optimization Procedure for flood control

Source: Kinori and Mevorach (1984)

2.12 Mathematical Models:

A Mathematical model is a simplified representative of certain aspects of a real system. It is created using mathematical concepts such as functions, graphs, diagrams and equations to solve problems in the real world (Edwards and Hamson, 1989). Neumaier (2004) defined mathematical modeling as the art of translating physical problems into tractable mathematical formulations whose understanding of the real life phenomenon and solution to the problem. Modeling involves identifying and selecting relevant features of a real-world situation, representing those features symbolically, analysing and reasoning about the model and characteristics of the situation, and considering the accuracy and limitations of the model (Aris, 1994). Agunwamba (2007) listed the importance of mathematical modeling as follows:

i. Assists in improving the quality of product.
ii. Improves the design operation and efficiencies of engineering systems.
iii. Predicts the occurrence of some hazards so that effective measures can be taken.
iv. Helps us to understand the modeled phenomenon better.

v. Models often allow quick and cheap evaluation of alternatives leading to optimal solutions which are not otherwise obvious.

vi. Models represent mathematical core of a situation without extraneous information.

A mathematical model can be formulated either through intuitive reasoning about the phenomenon or from physical law based on evidence from experiment. It is usually constructed in the language of mathematics, logic and computer following the algebraic rules of syntax. A mathematical model often takes the form of differential equation or a system of differential equations. The objective is usually to find specific solutions by imposing some initial boundary conditions on the equations. Neumaier (2004) illustrated the steps followed in developing a Mathematical model (Fig 3.7).

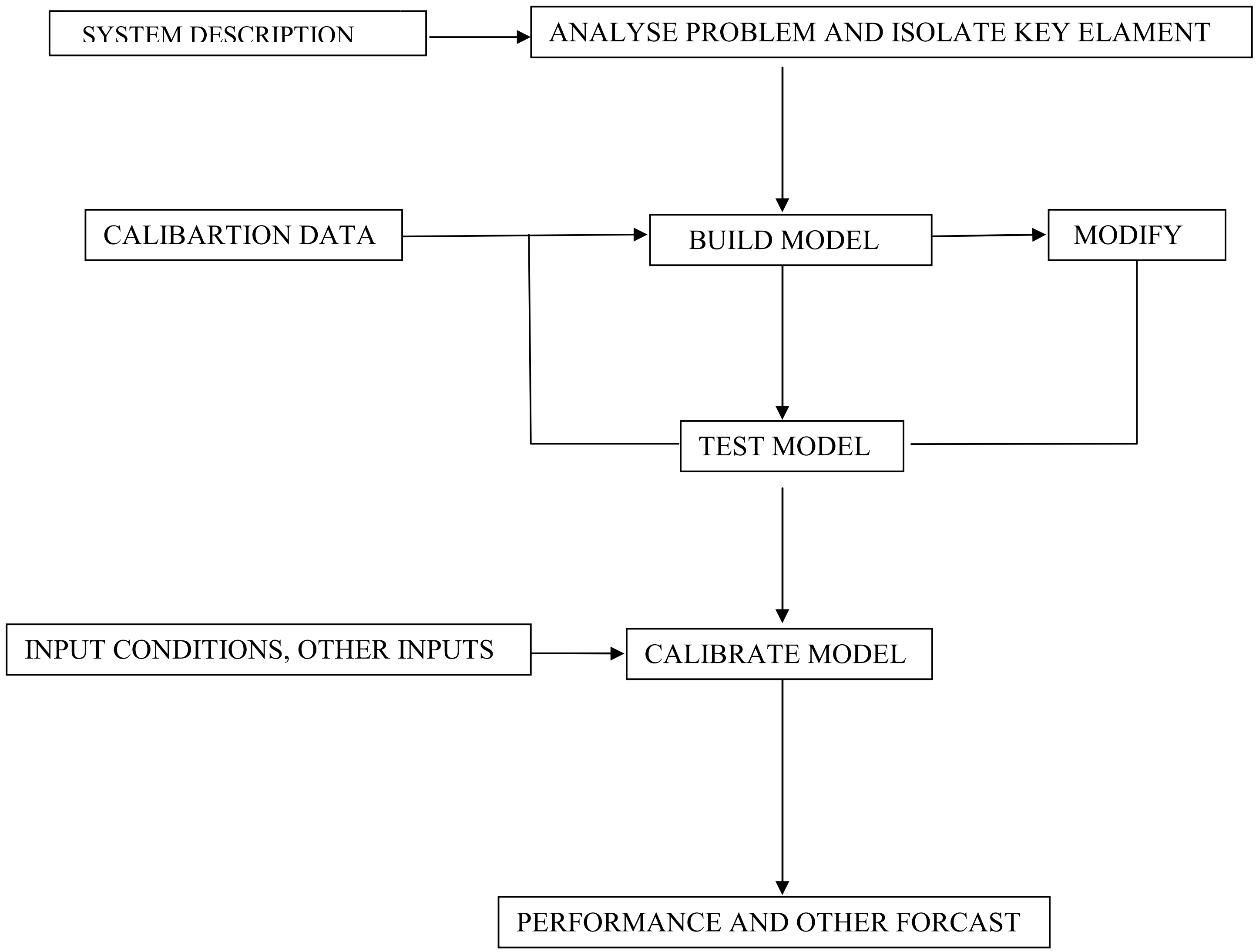


Fig 2.6 Steps in Developing a Mathematical Model

2.12.1 Erosion Models

Despite the importance of erosion studies, only a few models exist for the computation of soil loss due to erosion (Agunwamba , 2010). The universal soil loss equation developed in the US in the 1950's by Wischeimer and Smith from trials conducted from the late 1920's is very popular (Landon, 1996). It has been modified for use in various parts of the world. The equation is a quantitative method of predicting soil erosion losses from rainfall, soil and other factors. The equation is presented thus:

$$A = RKLSCP \qquad 2.23$$

Where,

A = Soil loss in short tons (2000Ib) per acre per year

R = Rainfall erosivity factor

K = Soil erodibility factor

L = Slope length factor

S = Slope gradient factor

C = Crop management factor

P = Conservation particle factor

For a particular region, A is determined empirically for a range of representative soils on experimental plots by standardizing factors L, S, C and P.

The soil erodibility factor is considered to be an inherent property of the soil series, and is independent of the effects of management which are covered by the factors P, C and L (Landon, 1996). Rainfall was either accurately measured or simulated, and the eroded soil was collected at the base of the slope for weighing. The universal soil loss equation (USLE) is difficult to use because it requires the evaluation of several factors which vary significantly from place to place.

Ume and Ojiako (1989) developed an exponential mathematical model for the rate of soil loss in gullies. It has not been tried out in practice. Describing the model to be more appropriate to sheet erosion, Eze Uzoamaka (1991) pointed out that it did not incorporate the soil shear strength. An attempt was made to in this by Eze Uzoamaka (1979) but it has not been developed to yield the rate of soil loss. The limitations of laboratory model notwithstanding, it has not yet been verified using independent experimental data. Ogbonna (1990) compared measured and predicted values of erosion for gullies, using equation developed by Koruma (1976). It predicted the soil losses well based on pilot-scale experiments.

Agunwamba (1997) compared Ume and Ojiako (1989) and Koruma (1976) models using field data. The results revealed some disparity between measured and predicted values. The predictive capacity of the laboratory models, an exponential model proposed by Ume and Ojiako was improved considerably when A, (detachment capacity of flow) was expressed as a function of slope and soil density and the parameter (a) expressed in terms of slope. On the other hand, koruma's model (1976) showed evidence of good predictive capacity when verified with field data. However, this was not obtained at low rainfall intensity (<2mm/hr) because of the assumption of constant momentum coefficient (β) which rather increases with decreasing rainfall intensity and the omission of interill detachment term which will usually be predominant for low flows.

Tayfur and Kaavas (1994) developed spatially averaged conservation equations for interacting rill interrill area overland flows. They developed a model that can simulate the flow over a hill slope containing both rills and interril areas. The flow in rills was conceptualized as two dimensional (2D) and having the dynamics of sheet flows (see equation 2.20). It was derived as the locally averaged equation of interrill- area sheet flows:

$$\frac{\partial h_o}{\partial t} + \frac{\partial}{\partial x}\left(C_x {h_o}^{5/3}\right) = q_t - \left(\frac{\pi}{2}\right)^{5/3} \frac{C_y}{l} {h_o}^{5/3} \qquad 2.20$$

$h_o \quad = \quad sheet\ flow\ depth$

$q_t \quad = \quad lateral\ flow$

$l \quad = \quad interrill\ area\ width$

C_x *and* C_y *are model parameters* (x *and* y *directions*)

In which,

$$C_x = \frac{\sqrt{S_{ox}}}{n\left[1+\left({S_{oy}}/{S_{ox}}\right)^2\right]^{1/4}} \qquad 2.21$$

$$C_y = \frac{\sqrt{S_{oy}}}{n\left[1+\left({S_{ox}}/{S_{oy}}\right)^2\right]^{1/4}} \qquad 2.22$$

S_{ox} and S_{oy} are bed slopes in the x and y – directions respectively.

The model proposed by Komura (1976) for sheet erosion with rill is expressed as:

$$E = C_A C_E {q^*}^{15/8} L^{3/8} {S_o}^{3/8} (N/D) \qquad 2.23$$

In which, E is the rate of soil loss (m^3/m^2 sec) and

$$q^* = 2.778 f I \qquad 2.24$$

Where, f is the runoff coefficient; I is the rainfall intensity (mm/hr); S_0 is the slope; L is the slope length (m); C_A is the bare soil area ratio to total slope area; C_E is the erodibility coefficient; D is the sediment size (mm) and N is a whole number at a given temperature of 26°c as

$$\frac{N}{D} = \frac{3000(1+2p)a_s v^{b/12}(k+0.012)^{b/3}}{(7b+6)(8g)^{b/3}\left[\frac{\rho_s{}^{-1}}{\rho}\right]^{P_1-1}}\left(\frac{2\beta}{(s-1)s}\right)^{1/2} \qquad 2.24$$

In which, D is expressed in m; v= kinematic viscosity ($1x10^6 m^2/s$);

$\rho \; and \; \rho_s$ are densities of water and sediments respectively; k = Darcy Weisbarch friction factor constant without rainfall; s= ratio of natural slope gradient to mean friction slope; g is acceleration due to gravity; β is momentum coefficient; a_s and b are constants; and p_1 is a dimensionless parameter.The exponential equation which relates the rate of soil loss to time velocity of flow distance in the direction of flow and mean particle size by Ume and Ojiako (1989) is as flows:

$$E = Ae^{\propto(ut-x_0)} \qquad 2.25$$

Where, E is the soil loss; A= detachment capacity of flow; u= flow velocity; x_o=distance in the direction of flow; t= time of flow, and a is ∝ constant.

Ijioma (2009) presented results of recent studies on erosion and sediment transport by McCaleb and McLaughlia (2008). The need to develop physically based soil erosion models and to describe the distributed processes of detachment, transport and deposition has stimulated researchers to establish the best equations for estimating overload flow transport capacity (Zhang and others, 2008). The Yalin equation according to Nearing and others (1989) has been widely used in soil erosion models. The equation which is an expression for bed-load transport is as follows

$$\frac{T_C}{sd\sqrt{\rho\tau}} = 0.635\delta\left[1 - \frac{1}{\beta(1+\beta)}\right] \qquad 2.26$$

$$\tau = \rho g R S \qquad 2.27$$

$$\delta = \frac{Y}{Y_c} - 1(when\ Y < Y_c\ \ , then\ \delta = 0 \qquad 2.28$$

$$\beta = 2.45{S_g}^{0.4}{Y_c}^{0.5} \qquad 2.29$$

$$Y = \frac{RS}{\left(S_g - 1\right)gh} \qquad 2.30$$

Where, Tc = Sediment transport capacity (Kg m^{-1} s^{-1})

S_g = Particle specific gravity

d = Particle diameter (m)

p = Water density (kgm^{-3})

τ = Sheer stress of flow (Pa)

R = Hydraulic radius (m)

h = flow depth (m)

S = slope gradient (mm^{-1})

Y = dimensionless shear stress

Y_C = dimensionless critical shear stress.

The equation was further simplified by Flagman (2007) and Zhang (2008) thus:

$$T_C = K_t \tau^{3/2} \qquad 2.25$$

Where, K_t is the sediment transport coefficient:

$$k_t = \frac{T_{co}}{{\tau_0}^{3/2}} \qquad 2.27$$

Where, T_{co} is the transport capacity computed by the Yalin equation using τ_o as representative slope identified from a particular shape of the profile.

2.13 Gully Erosion Problem in South Eastern Nigeria:

According to the report by the short mission to Erosion Center FUTO (Haagsma,1992), south eastern Nigeria is a densely populated area with an extensive road infrastructure and a high percentage of urban settlements, which contributes to high rates of runoff, due to the large area of relatively impervious surfaces. The report stated that dense compound surfaces in the semi-urban areas with low infiltration rates also yield high runoff volumes. This situation contributes to the wide spread of gully erosion.

The presence of gully sites is one of the hazard features that characterize this zone as well as other states that adjoin them (Ofomata, 1985). Asiabaka and Boers (1988) had estimated that over 1970 gully sites occur in Imo and Abia States. A conservative assessment shows the distribution of known gully sites in different stages of development as follows Abia (300), Anambra (700), Ebonyi (250), Enugu (600), Imo (450). (Igbokwe and others, 2003; Egboka, 2004). (See Table 2.7).

Table 2.7: Distribution of Gully sites in South Eastern Nigeria (at different stages of development.

S/N	**State**	**No of gully sites**	**State**	**Control measure**
1	Anambra	700	Mostly Active	Not Successful
2	Abia	300	Some active/some dominant	Not successful
3	Ebonyi	250	Mostly minor gully sites	No records
4	Enugu	600	Some active some dominant	Non
5	Imo	450	Some active some dominant	Not successful

Source: Igbokwe and others (2008), Egboka (2004).

A pictorial view of some of the active sites in the region is presented in Plate 2.1

.

Plate 2.1. Typical Erosion Sites in South Eastern Nigeria.

Table 2.8 presents the amounts of soil loss per annum from the states of Abia, Anambra, Ebonyi Enugu and Imo.

Table 2.8 Soil Losses in States of South Eastern Nigeria.

s/no	State	Soil loss(Tons/ha/Yr.)	
		Min (low areas)	Max (high areas)
1	Abia	9.20	10.16
2	Anambra	9.11	10.03
3	Ebonyi	8.71	9.60
4	Enugu	9.46	10.54
5	Imo	9.23	9.93

Source: Igbokwe and others (2008)

Haagsma (1992) reported that erosion in South Eastern Nigeria draws the attention of every visitor who sees the impressive gully system, which make huge scars in the landscape, threatening schools, villages, building, roads and fields. Within this context, erosion is often attributed to physical and climatic condition when the combination of highly erodible soils, undulating topography and high intensity of the frequent tropical thunder storms cause due to shifting cultivation system of agriculture that is no longer suitable under the condition of high population densities. Generally, erosion is a rather extraordinary phenomenon under the prevailing climatic conditions of south Eastern Nigeria.

2.14 Soil Erosion and Climate Change:

The consensus of atmospheric scientists is that climate change is occurring, both in term of global air temperature and precipitation patterns. (Wikipedia, 2009). Warmer temperatures associated with greenhouse warming are expected to lead to a more vigorous hydrological cycle including more extreme rainfall events. The natural greenhouse effect is a warming process whereby the greenhouse gases in the atmosphere trap the infra-red radiation that is trying to escape back into space. The increase in temperature of the globe causes the sea to rise and will change the amount and pattern of precipitation, probably including expansion of subtropical deserts (Knight, 2008). Studies on soil erosion suggest that increased rainfall amount and intensities will lead to greater rates of erosion. Thus, if rainfall intensities increase in many parts of the world as expected, erosion will also increase, unless amelioration measures are taken. Soil erosion rates are expected to increase in response to changes in climate.

CHAPTER THREE

METHODOLOGY

3.1 INTRODUCTION

Data for analysis were collected through field and laboratory investigations. These include hydraulic, hydrological, topographical, and geotechnical parameters. Specifically, the following data were collected:

i. Depth of flow (m)

ii. Rainfall amounts (mm) and duration (hrs)

iii. Flow rate of runoff (m^3/s)

iv. Velocity of flow (m/s)

v. Rate of accumulation of sediments (m^3/s)

vi. Relative concentration of silt (mg/mg)

vii. Gully profiles (horizontal and vertical alignments)

3.2 FIELD INVESTIGATIONS

Pilot studies were carried out at two different sites. Site ‘A’ was a natural gully developing at a location in Okpofe Ezinihitte – Mbaise, Imo state. The gully was at an early stage of development.

On the other hand, Site 'B' was an experimental channel formed on the slope of Otamiri River at Federal University of Technology, Owerri. Due to the high slope of the anal, it is prone to erosion. The channel has a cross section of 1.2 × 0.6m measuring a length of 15m. An artificial runoff simulated by pumping water from Otamiri river was directed into the channel.

3.3 Experimental Set- up:

3.3.1 **Artificial Runoff Simulation**

(a.) **Materials:**

These include:

A 5hp Yamaha Hydro pump, and 50mm diameter delivery hose, 100m long.

(b) **Method:**

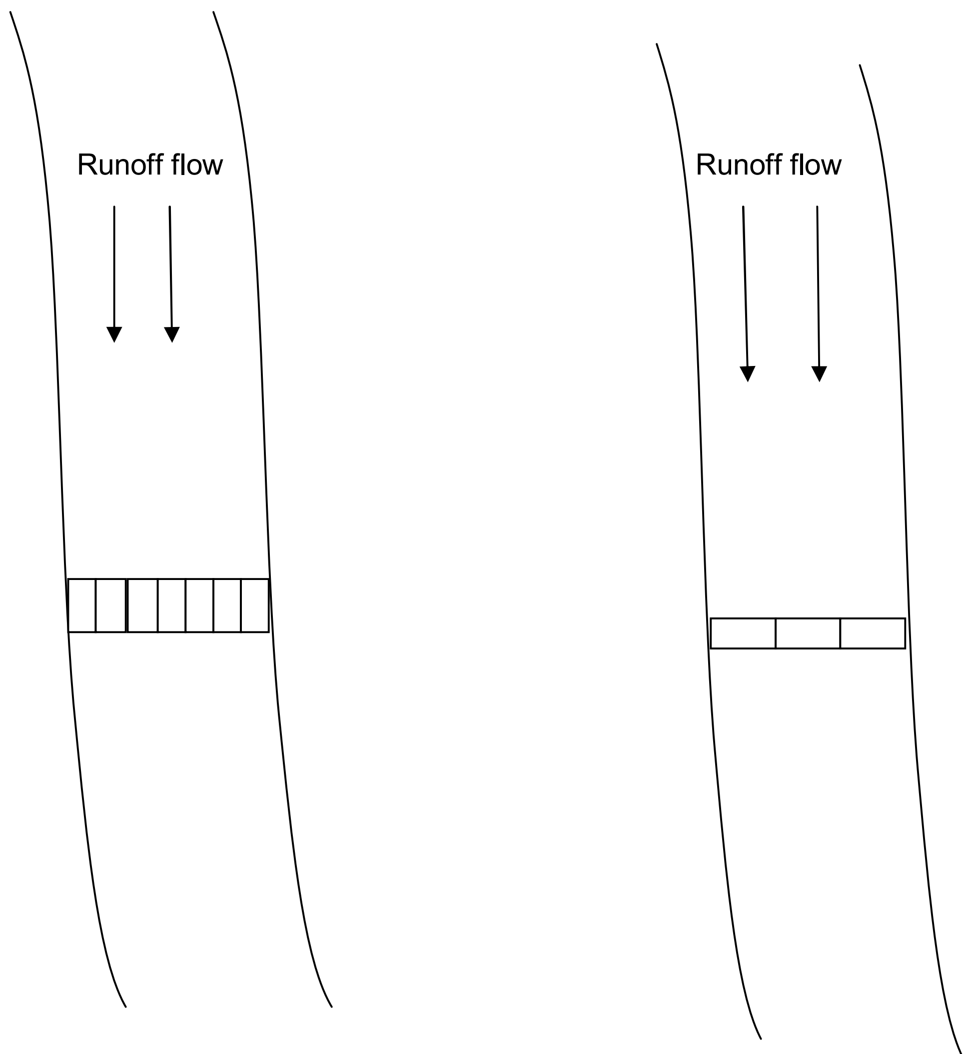
The pump was positioned at a suitable point on the bank of the river. The suction and delivery arms were connected. A strainer fixed to ensure that the suction mouth was protected from debris and suspended organic matter. The delivery mouth outlet was also firmly secured to avoid displacement. A control valve was fixed at the end of the delivery hose.

Water was pumped into the excavated channel as runoff flow (see Fig.3.1)

Fig.3.1 Experimental Setup for Artificial Runoff Simulation.

3.3.2 Placement of Sandbags:

Polypropylene bags of 0.9m length and 0.45m width were filled with sand to an average thickness of 0.25m and placed across the gullies. The heights and patterns of placement (transverse and longitudinal) were varied for different flow conditions. Fig. 3.2 shows the patterns of placement while Plates 3.1 and 3.2 show the sandbags being systematically placed for field observations.



(a) Longitudinal Placement (b) Transverse Placement

Fig 3.2. Patterns of sandbag placement

Plate 3.1. Sandbags systematically being placed for field observations 1

Plate 3.2 Sandbags systematically being placed for field observations 2

3.4 **Field Measurements:**

3.4.1 Precipitation Measurements:

(a) Materials:

i. **Rain gauge:** The study used the United States National weather services standard 20cm gauge, consisting of a rainfall receiver, a measuring cylinder, calibrated measuring stick, an overflow can, on a wooden or metal support. The top portion of the rainfall receiver has a 20cm inside diameter and a funnel shaped bottom that conducts the rainwater caught in the receiver into the tall cylindrical measurement tube.

(b) Method:

The Rain gauge was stationed in an open area within the vicinity of the study sites.

When 1” (2.54cm) rain falls into the receiver and is funneled into the measuring tube, the measuring tube is filled to a depth of 10” (25.4cm).

Accordingly, the scale of the measuring stick used was expanded 10times. The actual depth of water in the tube was read directly to hundredths from the stick. The measuring stick was 20” (50.8cm) full and could measure up to 2” (5.08cm) of rainfall. Additional rainfall spilled over into the overflow can. Since the rainfall depth in the overflow can was not increased 10 times, the measuring stick was not suitable for measuring directly in the overflow can. Instead, water from the can was poured into a measuring cylinder for direct measurement with the stick.

3.4.2 **Topographical Survey**

(a) Materials:

The materials used included:

30m steel tape, automatic Level fitted with a pendulum compensator. (Kern Gk IA model) and Kern Theodolite.

Kern Theodolite (KIA)

3.4.2.1. **Horizontal profile:**

The horizontal alignments of the study gullies were determined by measuring the deflection angle of the intersection points of the centre line of the gullies using the theodolite. The lengths were measured and the top/bottom width recorded at 10m intervals. The theodolite was mounted on a firm tripod and centered over a fixed point on the ground.

3.4.2.2. **Vertical Profile:**

The vertical profile of the gullies were obtained by taking levels at 10m intervals on the bed and top edge of the gullies using the automatic level instrument. The instrument was maintained on a tripod by means of a spherical seating and clamping screw. A small circular bubble was used for initial setting up. A datum level of 100m was first established which served as a temporary benchmark T.B.M for the survey.

3.4.3 Measurement of flow Velocity:

(a) Materials:

Materials for flow velocity measurement included:

Current Meter, Stop watch, floating paper and Measuring Tape.

(b) Method:

A current meter was to be used to measure the velocity of flow inside the gully. However due to limited flow depth, the instrument could not be utilized effectively. A conservative estimate of average velocity of flow was therefore got by the float method. A flow material was introduced at a point and the time it took to travel known distance was recorded for which the velocity was computed.

3.4.4 Determination of flow rate Q:

The quantity of water flowing per unit time across a given section of the gully Q (m^3/s) was computed from the formula:

$$Q = A.V. \qquad 3.1$$

Where, A = Cross-sectional area

V = Velocity of flow

The average cross section dimensions of the gullies were recorded.

3.4.5 Determination of Rate of Siltation:

The rate of accumulation of sediments in (m^3/s) was determined by measurement of the average heights of deposits left behind sandbags over given time intervals. The depths were multiplied with the area or deposits to get the volume of silts accumulated.

$$Rate\ of\ Siltation = \frac{Volume\ of\ silts\ (m^3)}{time\ \ (s)} \qquad 3.2$$

3.4.6 Collection of Samples:

Samples of runoff of given volumes were collected at specific time intervals and properly labeled for laboratory determination of weight of sediments per unit volume of runoff water expressed in (mg/l).

3.5 Laboratory Tests.

3.5.1 Determination of Sediment Concentration:

The volume of water samples collected was measured in a measuring cylinder. The weights were also recorded. Thereafter the water was evaporated by drying in an oven leaving the sediments. The remaining sediments were scooped and weighed in a weighing balance.

$$Concentration\ \left(\frac{mg}{l}\right) = \frac{m_s}{v} \qquad 3.3a$$

Similarly,

$$Relative\ Concentration\ \left(\frac{mg}{mg}\right) = \frac{m_s}{m_r} \qquad 3.3b$$

$$m_s = mass\ of\ sediments,$$

$$v = volume\ of\ runoff\ with\ sediments,$$

$$m_r = mass\ of\ runoff\ with\ sediments.$$

3.6 **Methods of data Presentation:**

Data recorded from the above studies were presented in tables and graphs. The graphs (XY- Scatter) were presented in Microsoft Excel format. The charts compared pairs of values that represent separate measurements. The option with smooth lines and markers were adopted.

3.7 METHODS OF ANALYSIS

3.7.1 Modeling

Two approaches; material balance principle and regression analysis were employed in the modeling. The following variables were considered

i. Flow rate, Q m^3/s

ii. Volume sediments, V m^3

iii. Rate of sediment accumulation (storage), S m^3/s

iv. Initial relative concentration, C_0

v. Relative concentration at given time, C_1

vi. Time, t secs.

vii. Height of Sandbags h_S m

3.7.2 Model Calibration

The models were calibrated by determining, checking and adjusting the coefficients. This was done with another set of experimental data.

3.7.3 Model Verification

A set of measurements from the experimental set up used in formulating each model was used to evaluate the performance. Predicted values of the variables were plotted against the measured values. Subsequently, R^2 values and standard errors were determined.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Introduction:

Relevant data collected from field and laboratory investigations are presented and analyzed in this chapter. Mathematical models are formulated, calibrated and verified using different sets of field data. Basic relationships between relevant variables are established and illustrated graphically. Results of cost analysis and stability analysis are presented. The results and findings of studies are discussed in this chapter.

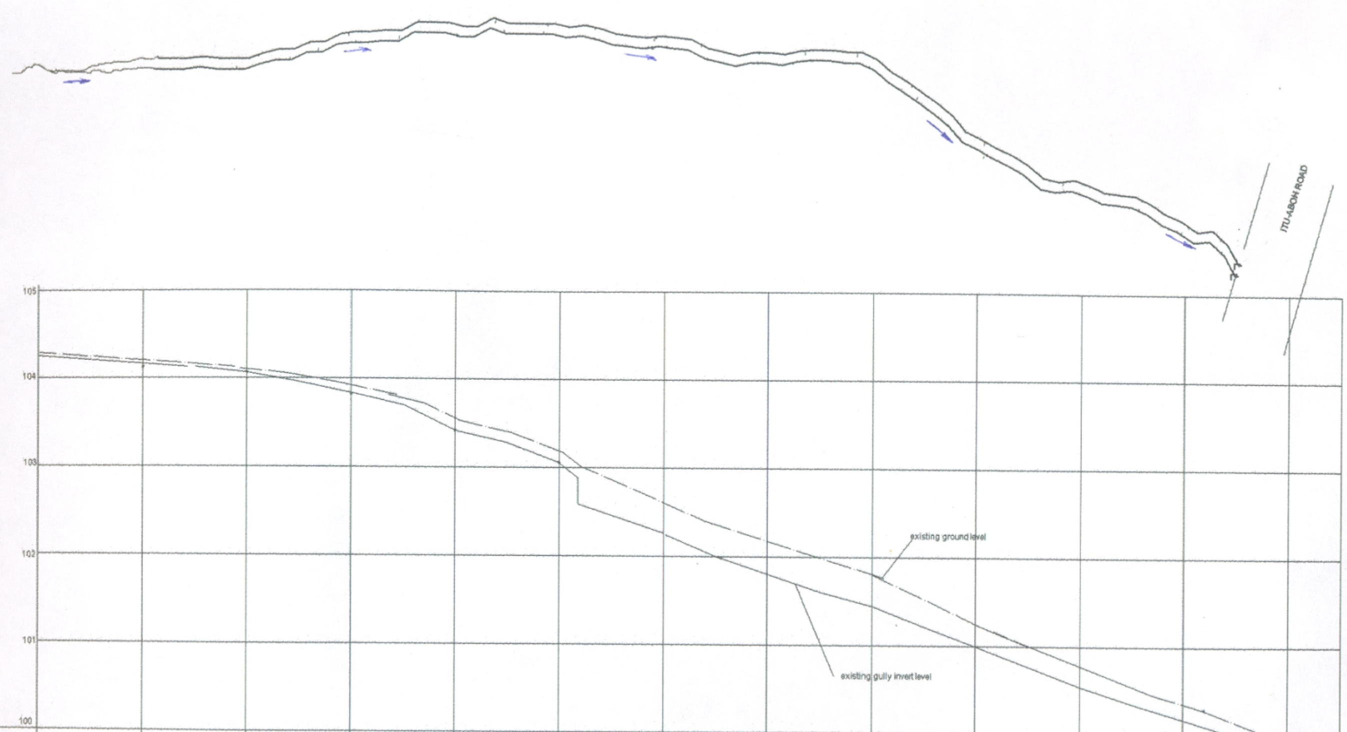
4.2 Data Presentation:

4.2.1 Topographical Data:

The results of topographical survey carried out to obtain the profile of the gullies are presented. Fig.4.1 represents the gully profile prior to placement of the sandbags. The maximum depth of the natural gully at ch 0+062 is 0.65m. Fig4.2 illustrates the new profile after placing the sandbags for one month (June 2008). It could be noticed that siltation flushed horizontally with the top of the bags. (See Plate 4.1).

The total volume of sediment accumulated was $45m^3$ which is 46.5% of the total capacity of the gully.

Plate 4.2 shows the artificial channel (site 'B') with sandbags placed across to reduce the velocity of flow and encourage siltation upstream. The elevation at the river bank (pumping point) is 98.597m while the elevation at the inflow point into the gully is 94.937mm. The level difference is 3.624m over a length of 57m giving a slope s which is 0.6376.



DATUM 100														
EXISTING GROUND LEVEL	104.386	104.31	104.201	103.89	103.664	103.256	102.646	102.315	101.712	101.203	100.822	100.314	99.818	99.675
GULLY INVERT LEVEL	104.216	104.200	104.068	103.768	103.407	103.001	102.314	101.786	101.432	101.011	100.568	100.117	99.631	100.214
CHAINAGE (m)	0+000	0+010	0+020	0+030	0+040	0+050	0+060	0+070	0+080	0+090	0+100	0+110	0+120	0+125

Fig 4.1. INITIAL GULLY PROFILE (SITE A)

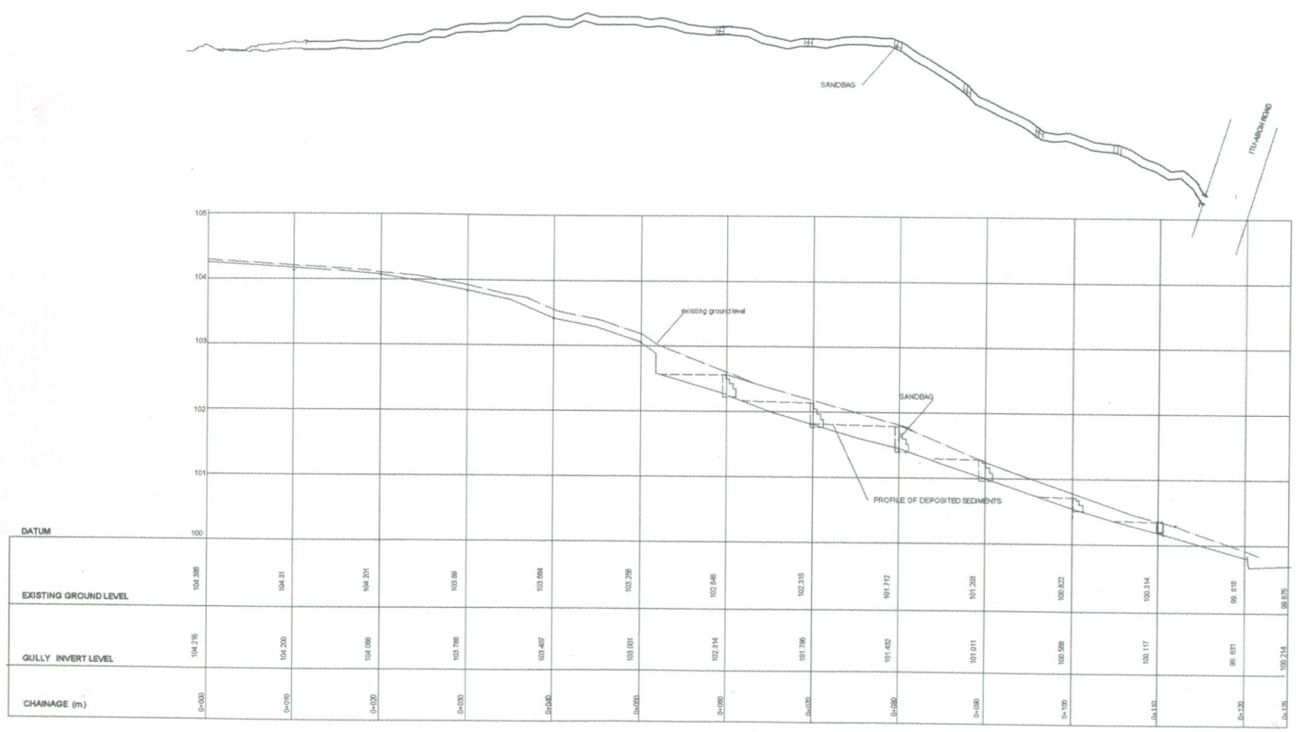


Fig 4 .2 NEW PROFILE OF GULLY .

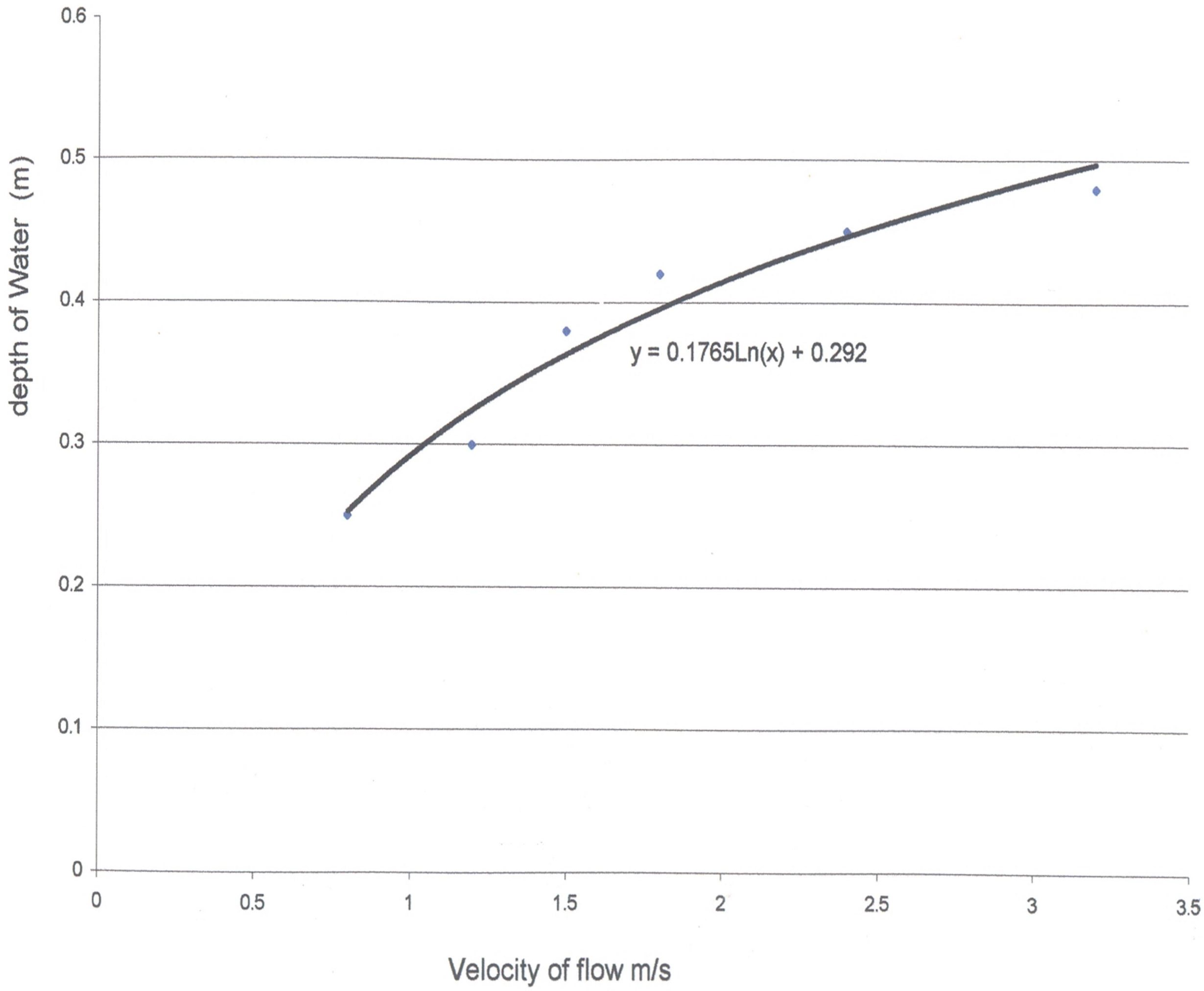


Fig 4.3 VARIATION OF FLOW DEPTH IN A GULLY WITH VELOCITY

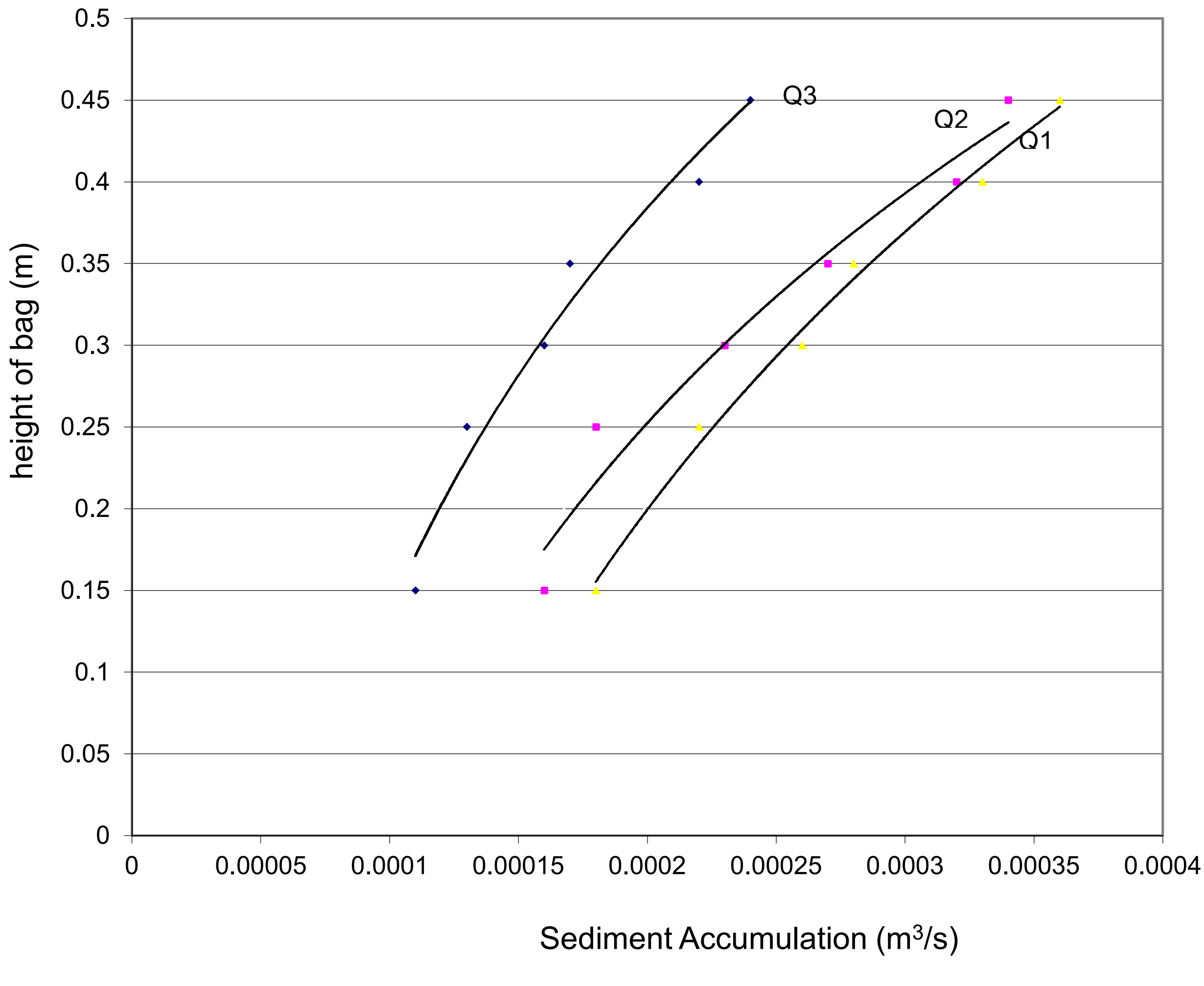


Fig 4.5. Variation of sediment Accumulation Rates with Height of Sand bags for various values of Q

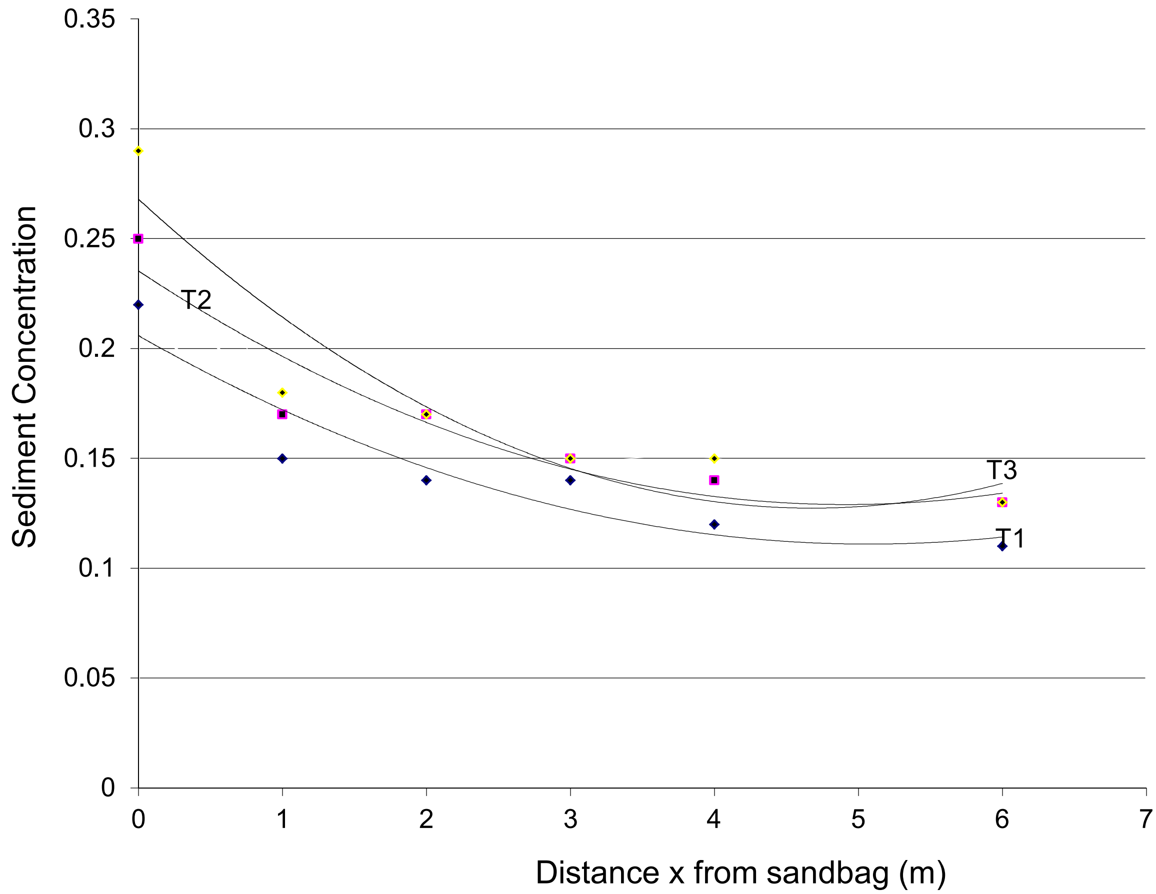


Fig 4.6 Profile Of Sediment Concentration Upstream

Plate 4.1. Sandbags placed in the natural gully (Site A)

Plate 4.2. Sandbags placed in artificial Gully (Site B).

4.2.2 Hydrological data:

(i) Depth – Velocity Relationship:

A plot of the variation of water depths with velocity for the gully used for the pilot study is presented in Fig. 4.3. The graph shows that velocity increases with depth of flow in the form.

$$Y = a_0 + b\ln(x) \qquad 4.1$$

The solution of equation 4.1 gives a_o = 0.292, b = 0.1765. Expressing the equation in terms of velocity of flow (v) m/s and depth (h) m:

$$v = e^{(5.666h - 1.654)} \qquad 4.2$$

Equation 4.2 could be used to predict velocity of flow for any given depths of water in the gully.

(ii) Rainfall measurements:

Daily rainfall records at site 'B' (FUTO) for the period June – November 2009 are presented in Appendix 5. The maximum rainfall of 43.7m was recorded on 10/6/09. From the records, the maximum rainfall intensity is 20.3mm/hr.

(iii) **Rate of Accumulation of Sediments:**

Variation of height of sandbags with sediment accumulation rate for different flow rates Q1, Q2, Q3 is presented in Fig 4.5. (Q1 = 0.12m^3/s, Q2=0.1m^3/s, Q3=0.75 m^3/s). The result showed that for a given flow rate, the rate of accumulation of sediments increases with the height of sandbags in the form of a quadratic curve. For the same height of sandbags, higher values of Q resulted in larger sediment accumulation over a given time.

(iv) **Sediment Concentration:**

Variation of sediment concentration of runoff with distance x from the sandbags for different durations T1, T2, and T3. (T1 = 30mins, T2 = 60mins and T3 = 120mins) is presented in Fig 4.6. The curves indicate that concentrations are higher as x approaches zero, and gradually reduces further upstream to an approximate constant value. Variations of sediment concentration with time for transverse (Tp) and longitudinal placement of sandbags (Lp) is presented in Fig. 4.7. The result showed that sediment concentration at a given time is higher for transverse arrangement than for longitudinal placement. This implies that transverse arrangement is more efficient.

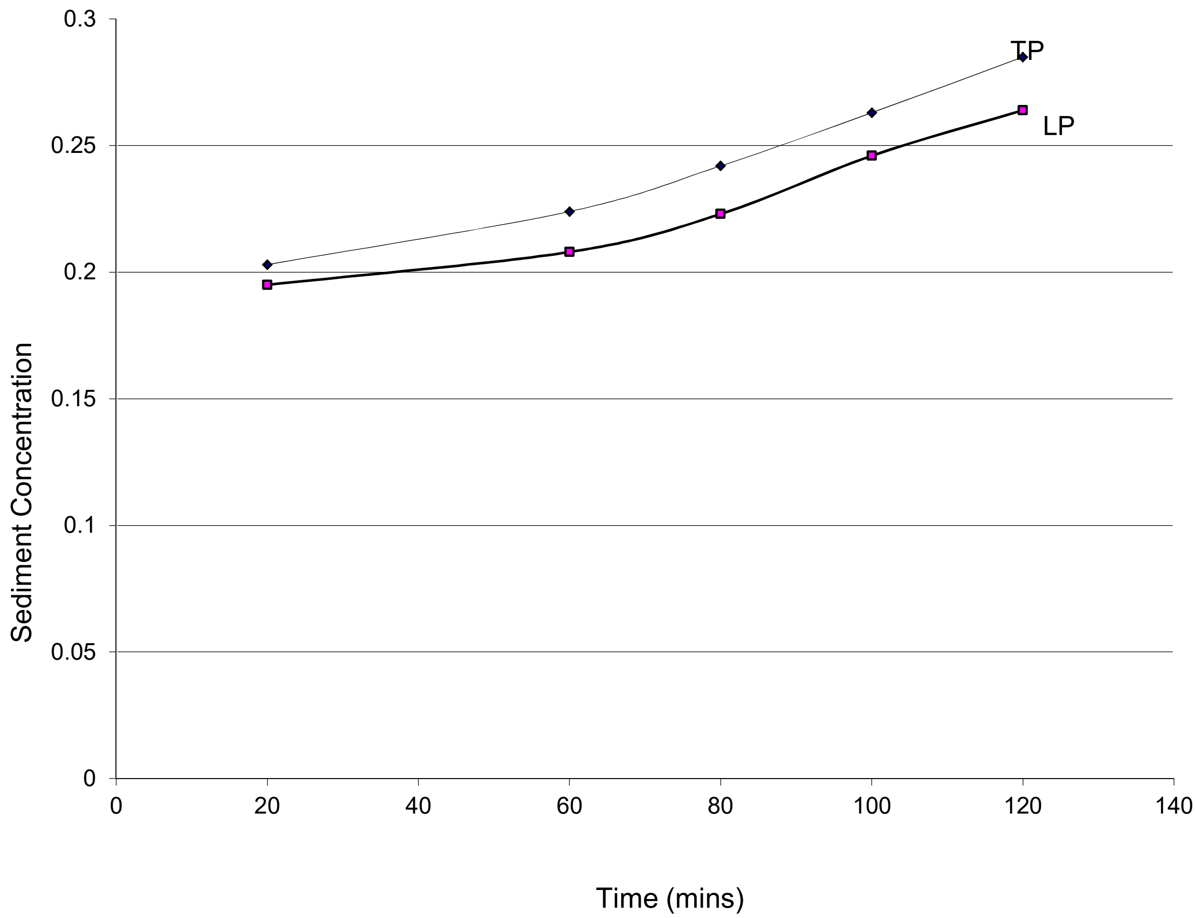


Fig 4.7 Variation Of Sediment Concentration with time for Transverse And Longitudinal Placement Of Sandbags

4.3 MASS BALANCE EQUATION FOR SANDBAG CHECKDAM

In a control volume (Fig 4.8)

$$\text{Inflow} - \text{outflow} = \text{storage} \qquad 4.3a$$

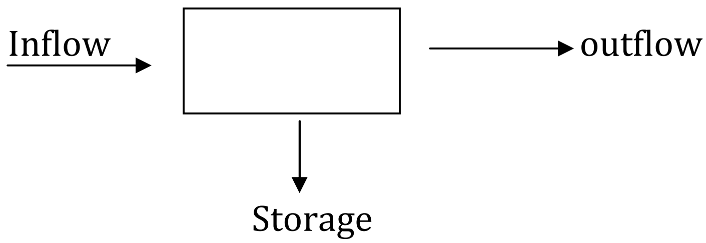


Fig. 4.8 Flow diagram for Principle of Material Balance

In this case, the storage is rate of accumulation of sediments upstream of the sandbags. The following variables and parameters were considered:

viii. Flow rate, Q

ix. Volume sediments, V

x. Rate of sediment accumulation (storage), S

xi. Relative sediment concentration, C

xii. Initial relative concentration, C_0

xiii. Relative concentration at given time, C_t

xiv. Time, t.

A set of differential equations that relate the above parameters was derived.

4.3.1 Objective

The objective was to develop a mathematical relationship among the following parameters based on the principle of material balance.

(i) Flow rate, Q

(ii) Sediment concentration, C.

(iii) Rate of Accumulation of sediments (storage), S.

(iv) Volume of sediments deposited upstream of sandbags, V

4.3.2 Assumptions

The following basic Assumptions were made;

(i) One dimensional flow.

(ii) That the Principle of Material balance is applicable.

(iii) That the rate of sediment Accumulation is constant.

4.3.3 Initial Conditions

At $t=0$, $C=C_{0,}$ For all values of $x>0$

$C_1(0) = C$.

C_o, indicates relative concentration of the runoff before being discharged into the gully. Thereafter C increases with time due to erosion of the bed by the runoff.

4.3.4 Derivation of Equations:

Based on the Principle of Material balance;

Rate of accumulation of sediments = inflow- outflow . 4.3

The following differential equations are derived:

$$\frac{\partial(\mathrm{VC})}{\partial \mathrm{t}} = \mathrm{QC_0} - \mathrm{QC} - \mathrm{k_0}\, u \frac{\partial \mathrm{C}}{\partial \mathrm{x}} \qquad 4.4$$

$$\frac{\mathrm{V}\,\partial \mathrm{C}}{\partial \mathrm{t}} = \mathrm{Q}(\mathrm{C_0} - \mathrm{C}) - \mathrm{k_0} u \frac{\partial \mathrm{C}}{\partial \mathrm{x}} \qquad 4.5$$

$$\frac{\mathrm{VdC_1}}{\mathrm{dt}} = \mathrm{Q}(\mathrm{C_0} - \mathrm{C_1}) - \mathrm{SC_1} \qquad 4.6$$

where :

Q = flow rate, Q m^3/s

u = velocity of flow, m/s

C = Relative Sediment Concentration of runoff.

C_0 =Initial Sediment Concentration

C_1 = Concentration at time, t (sec)

S = Rate of Accumulation of sediments (storage), m^3/s

V= Volume of sediments deposited upstream of sandbags, m^3

k_0 is a dimensional coefficient in L^3, while $\frac{\partial C}{\partial x}$ and $\frac{\partial C}{\partial t}$ refer to change in concentration per length and change in concentration with time respectively.

4.3.5 Solution of Equation by Integrating Factor Method

Exact solutions are not possible for first order linear equations of the form:

$$\frac{dy}{dx} + Py = Q \qquad 4.7$$

$$where, P \text{ and } Q \text{ are constants or functions of } x$$

The method of Integrating factor was therefore adopted. This mehod enabled us to obtain exact solutions. Both sides of the equation were multiplied by an Integrating factor (Stroud and Booth, 2001).

$$I.F = e^{\int PdX} \qquad 4.8$$

A Mathematical model relating sediment concentration with runoff flow rate and sediment accumulation rate was developed from the solution.

From Equation 4.5,

$$\frac{dC_1}{dt} + \left(\frac{Q+S}{V}\right)C_1 = \frac{QC_0}{V} \qquad 4.9$$

$$\text{Integrating factor} = e^{\int \left({(Q+S)}/{V} \right) dt} \qquad 4.10$$

$$\mathrm{I.F} \quad = \quad e^{\left((Q+S)/V\right)t} \qquad 4.11$$

$$V.e^{\left(\frac{Q+S}{V}\right)t}\frac{dC_1}{dt} + e^{\left(\frac{Q+S}{V}\right)t}(Q+S)C_1 = QC_0 e^{\left(\frac{Q+S}{V}\right)t} \qquad 4.12$$

$$\left[C_1 e^{\left(\frac{Q+S}{V}\right)t}\right]' \quad = e^{\left(\frac{Q+S}{V}\right)t}\frac{QC_0}{V} \qquad 4.13$$

$$C_1 \quad = \quad \frac{QC_0}{Q+S} + ke^{-\left((Q+S)/V\right)t} \qquad 4.14$$

$$e^{\left((Q+S)/V\right)t} \quad = \quad \frac{QC_0}{V}V\left(\frac{e^{\left(\frac{Q+S}{V}\right)t}}{Q+S}\right) + k \qquad 4.15$$

$$C_1 - \frac{QC_0}{Q+S} \quad = ke^{-\left(\frac{Q+S}{V}\right)t} \qquad 4.16$$

$$\ln\left[C_1 - \frac{QC_0}{Q+S}\right] \quad = \ln k - \left(\frac{Q+S}{V}\right)t \qquad 4.17$$

Let,

$$\left(C_1 - \frac{QC_0}{Q+S}\right)C_1 = A \qquad 4.18$$

Therefore, equation 4.13 becomes

$$\ln A = \ln k - \left(\frac{Q+S}{V}\right)t \qquad 4.19$$

Equation 4.19 is illustrated graphically as shown in Fig 4.8a. A plot of ln A against t would give a straight line with ln k as the intercept and $(Q+S)/V$ as the slope.

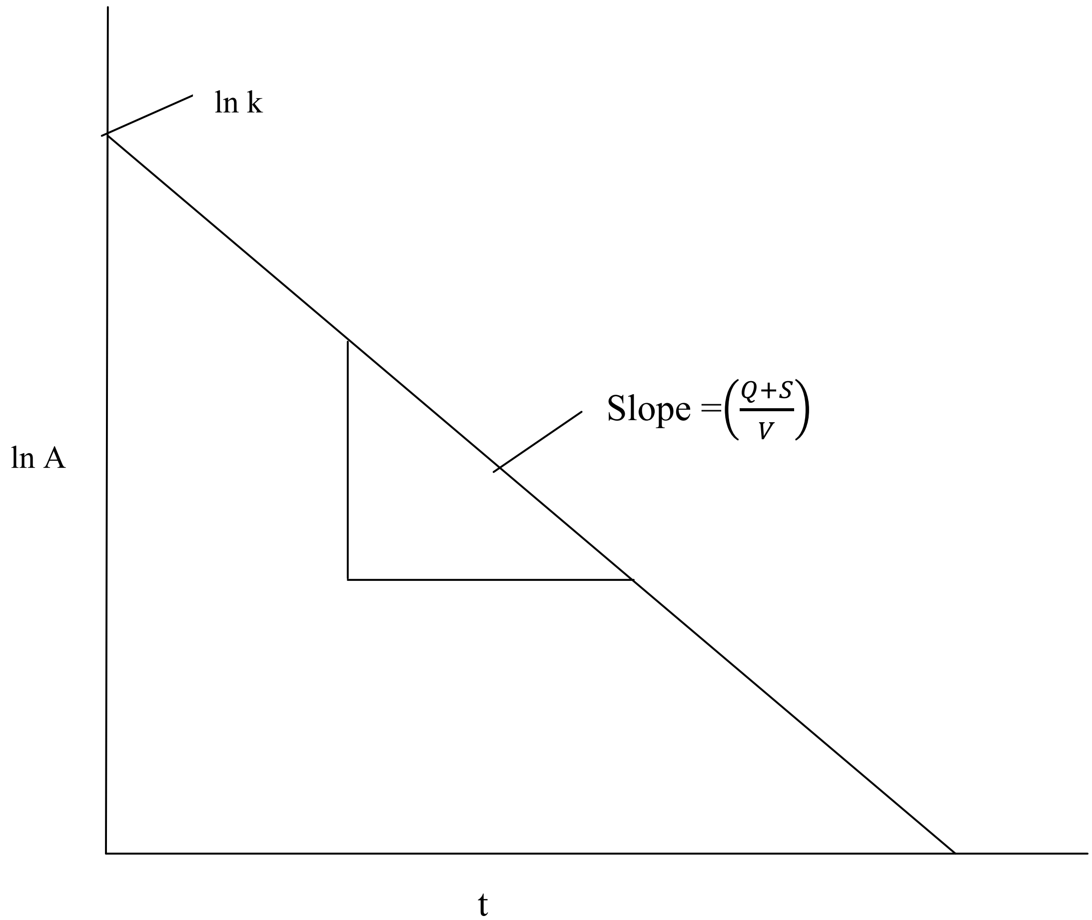


Fig 4.8a Graphical Illustration of Equation 4.19.

For various values of Q, C_O, and S, C_1 values measured at different time intervals are presented in Appendix 6. From the data, values of In(A) for different time intervals were computed for each 3 sets of data. (See Tables 4.1- 4.4). The plots of In(A) against time are presented in Figs 4.9a – 4.9d with the resultant fitting line for each set of data labeled '1' '2' and '3' accordingly. In each case, the k value for the line that has highest correlation coefficient (R^2) was chosen as optimum. Results from the graphs indicate that values of k vary from 0.048 -014.

A summary of optimum k values and C_O for each case is presented in Table 4.3. From the Table, it could be observed that k increases with C_O. The relationship is presented graphically in Fig 4.10. Thus, k could be estimated easily from C_O from the equation:

$$k = 7.38C_0^{1.608} \tag{4.20}$$

A range of values of k and C_0 are computed and presented in Appendix 7.

Substituting for k in equation 4.13, we obtain the equation below:

$$\ln\left[C_1 - \frac{QC_0}{Q+S}\right] = \ln\left(7.38C_0^{1.608}\right) - \left(\frac{Q+S}{V}\right)t \tag{4.21}$$

TABLE 4.2A Values of C_1 and In (A), (Case 1:Q = 0.12m^3/s, C_o = 0.02, S = 0.00022 m^3/s)

T =	10 mins	20mins	30 mins	40 mins
C_1	0.05	0.1	0.15	0.2
In (A)	-3.38	-2.4	-2.00	-1.69
C_1	0.04	0.05	0.06	0.07
In(A)	-3.73	-3.38	-3.12	-2.92
C1	0.03	0.035	0.045	0.06
In(A)	-4.27	-3.96	-3.54	-3.123

TABLE 4.2B Values of C_1 and In (A), (Case 1: Q = 0.12m^3/s, C_o = 0.036, S = 0.0002 m^3/s)

T	10mins	20mins	30mins	40mins
C_1 In (A)	0.09 -3.00	0.23 -1.66	0.38 -1.079	0.52 -0.73
C_1 In (A)	0.1 -2.813	0.28 -1.427	0.41 -0.994	0.56 -0.653
C_1 In (A)	0.06 -3.912	0.18 -1.966	0.37 -1.109	0.42 0.968

TABLE 4.2c Values of C_1 and ln(A), (Case 3: Q = 012m^3/s, C_o = 0.036, S = 0.00024 m^3/s)

T	10mins	20mins	30mins	40mins
C_1 ln(A)	0.04	0.19 -1.858	0.27 -1.444	0.34 -1.184
In (A)	0.06 -3.650	0.21 -1.737	0.32 -1.252	0.34 -1.184
In (A)	0.1 -2.718	0.21 -1.737	0.29 -1.363	0.33 -1.217

TABLE 4.2d Values of C_1 and ln(A), (Case 4: Q = 012m^3/s, Co = 0.041, S = 0.00019 m^3/s)

T	10mins	20mins	30mins	40mins
C_1 In (A)	0.12 -2.538	0.25 -1.565	0.39 -1.053	0.44 -0.919
C_1 In (A)	0.16 -2,128	0.28 -1.431	0.38 -1.082	0.51 -0.757
C_1 In (A)	0.11 -2.674	0.156 -2.155	0.299 -1.355	0.431 0.942

Table 4.3. Optimum Values of K

C_O	K
0.041	0.048
0.039	0.041
0.036	0.03
0.02	0.014

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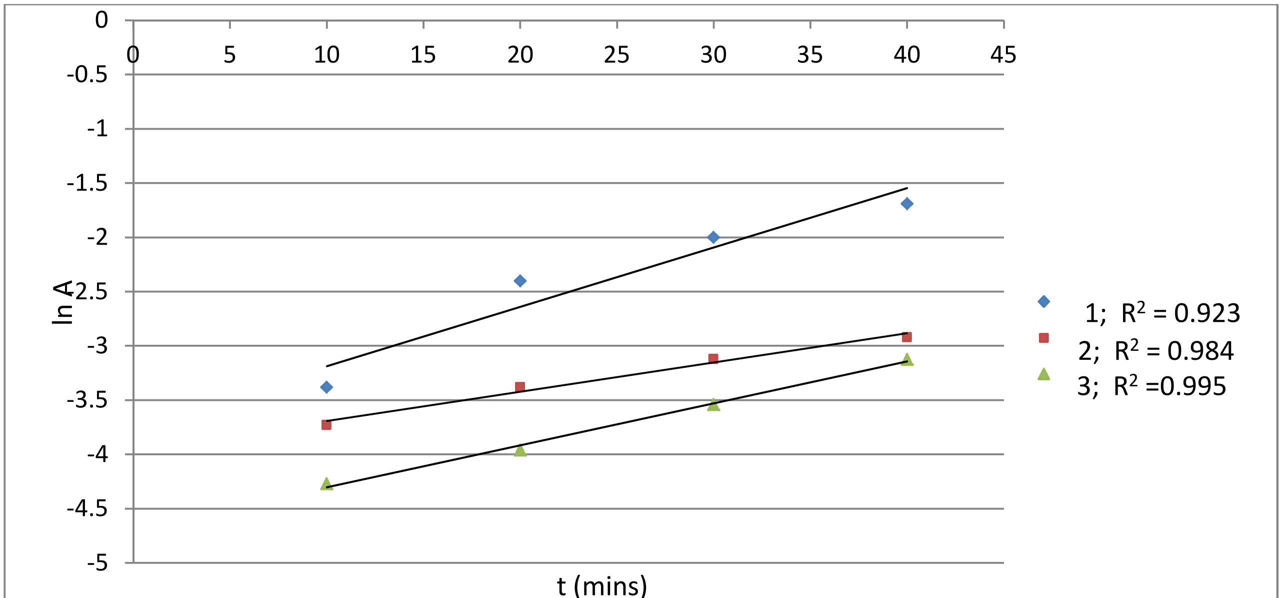


Fig. 4.9a Plot of ln (A) against Time (CASE 1)

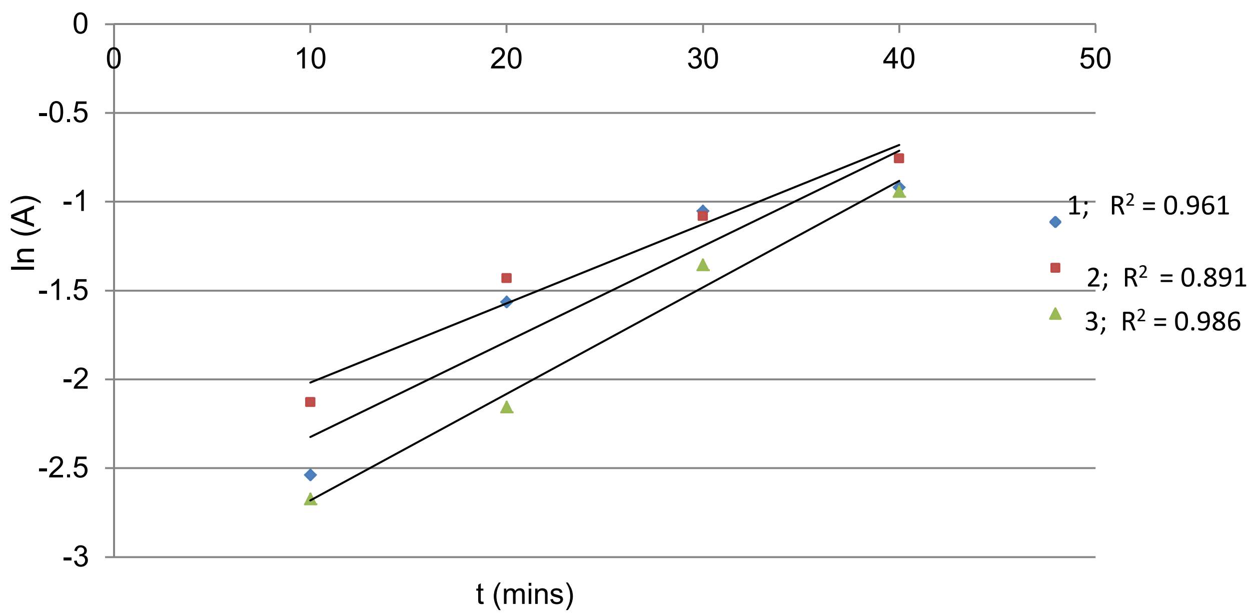


Fig. 4.9b Plot of ln(A) against Time (Case 2)

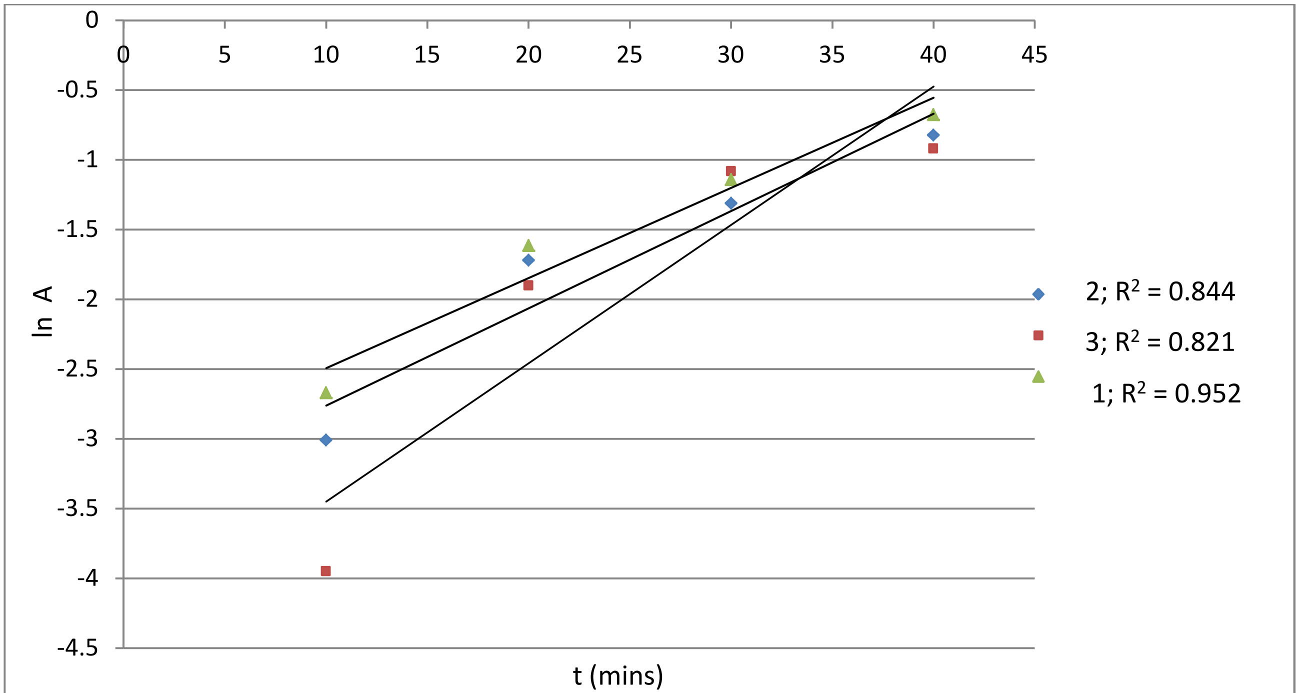


Fig 4.9c Plot of ln (A) against Time (CASE 3)

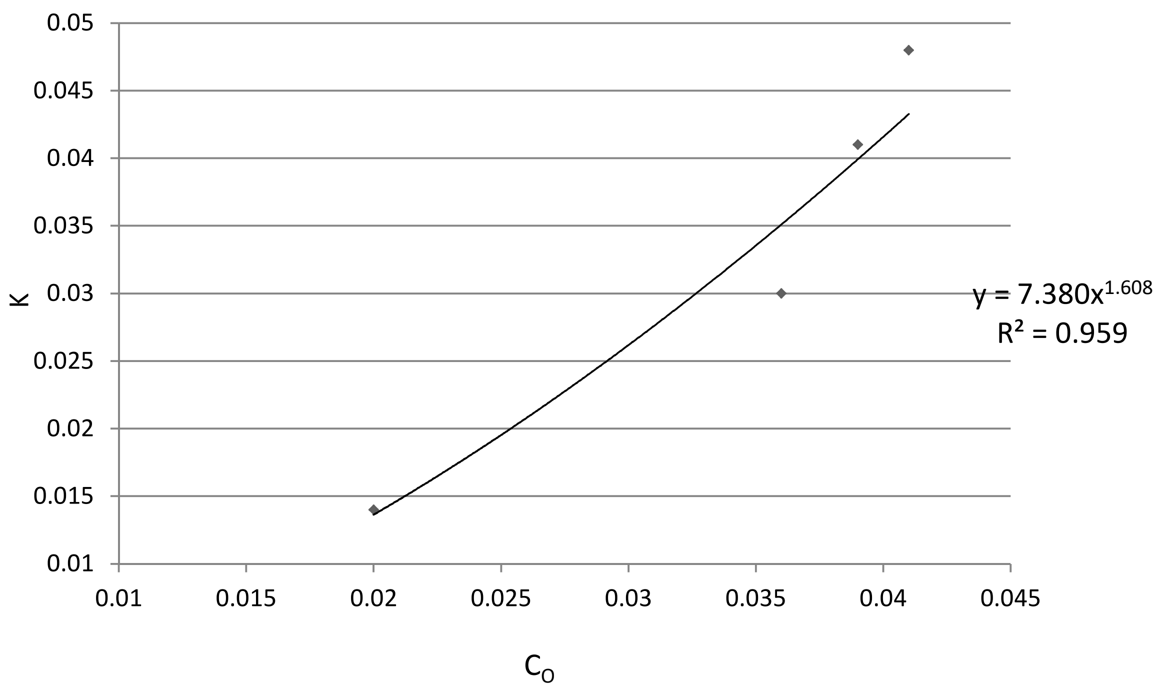


Fig. 4.10 Variation of k with C_o

4.3.6 Model Calibration:

The next step was to check and adjust the model using another set of experimental data.

Data: $Q=0.12m^3/s$, $C_o = 0.08$, $S = 0.00012\ m^3/s$, $t = 1200secs$, $V = 0.144m^3$

C_1 (Measured) = 0.21

$$C_1 = \frac{0.12 \times 0.08}{0.12 + 0.00012} + (7.38 \times 0.08^{1.608})e^{-\left(^{(0.12+\ .00012)}/_{0.144}\right)1200}$$

$$0.21 = 0.127 + 0.1036e^{-145}$$

In other to have a feasible solution, a calibration factor α, was introduced;

$$0.21 = 0.127 + 0.1036e^{-\alpha(145)}$$

$$\alpha = 1.53 \times 10^{-3}$$

Equation 4.16 is thus adjusted as

$$\ln\left[C_1 - \frac{QC_0}{Q+S}\right] = \ln\left(7.38{C_0}^{1.608}\right) - \alpha\left(\frac{Q+S}{V}\right)t \qquad 4.18$$

Similarly, equation 4.10 becomes:

$$C_1 = \frac{QC_0}{Q+S} + (7.38 \times {C_0}^{1.608})e^{-1.53\times10^{-3}\left(^{(Q+S)}/_{V}\right)t} \qquad 4.19$$

4.3.7 Model Verification:

The set of data presented in Table 4.4 was used for model verification. Column ‘6’ shows measured values of C_1 at given time intervals for tests ‘1A – 1D’.

Table 4.4 Experimental Data for Model Verification (Case 1)

	Q (m^3/s)	C_0	S (m^3/s)	t (mins)	C_1
A	0.12	0.024	0.00022	50	0.032
B	0.12	0.042	0.00028	60	0.065
C	0.12	0.033	0.00026	45	0.047
D	0.12	0.02	0.00019	40	0.025

Alternatively, Equation 4.19 was used to predict values of C_1 using data from Table 4.4. The computed values (y_{est}) are tabulated side by side with the measured (y_1) values in Table 4. To compute the standard error of estimates, it was also necessary to tabulate values of $(y_1\text{-}y_{est})^2$ as shown in Table 4.5.

Table 4.5 Observed and Estimated Values of C_1

	Y	y_{est}	$(y-y_{est})^2$
A	0.029	0.032	$9x10^{-6}$
B	0.062	0.065	$9x10^{-6}$
C	0.042	0.047	$2.5x10^{-5}$
D	0.021	0.025	$1.6x10^{-5}$

A plot of measured values against predicted values is presented in Fig. 4.10. The coefficient of Correlation R^2 is 0.964

From equation 3.12,

$$\text{Standard error} = \sqrt{\frac{\sum(y_1 - y_{est})^2}{n}}$$

$$\sum(y_1 - y_{est})^2 = 5.9 \times 10^{-5}$$

$$n = 4$$

$\therefore$ Standard error = 3.841×10^{-3}

The values of R^2 and Standard error indicate closeness of the predicted values with the observed values thus confirming the validity of the model.

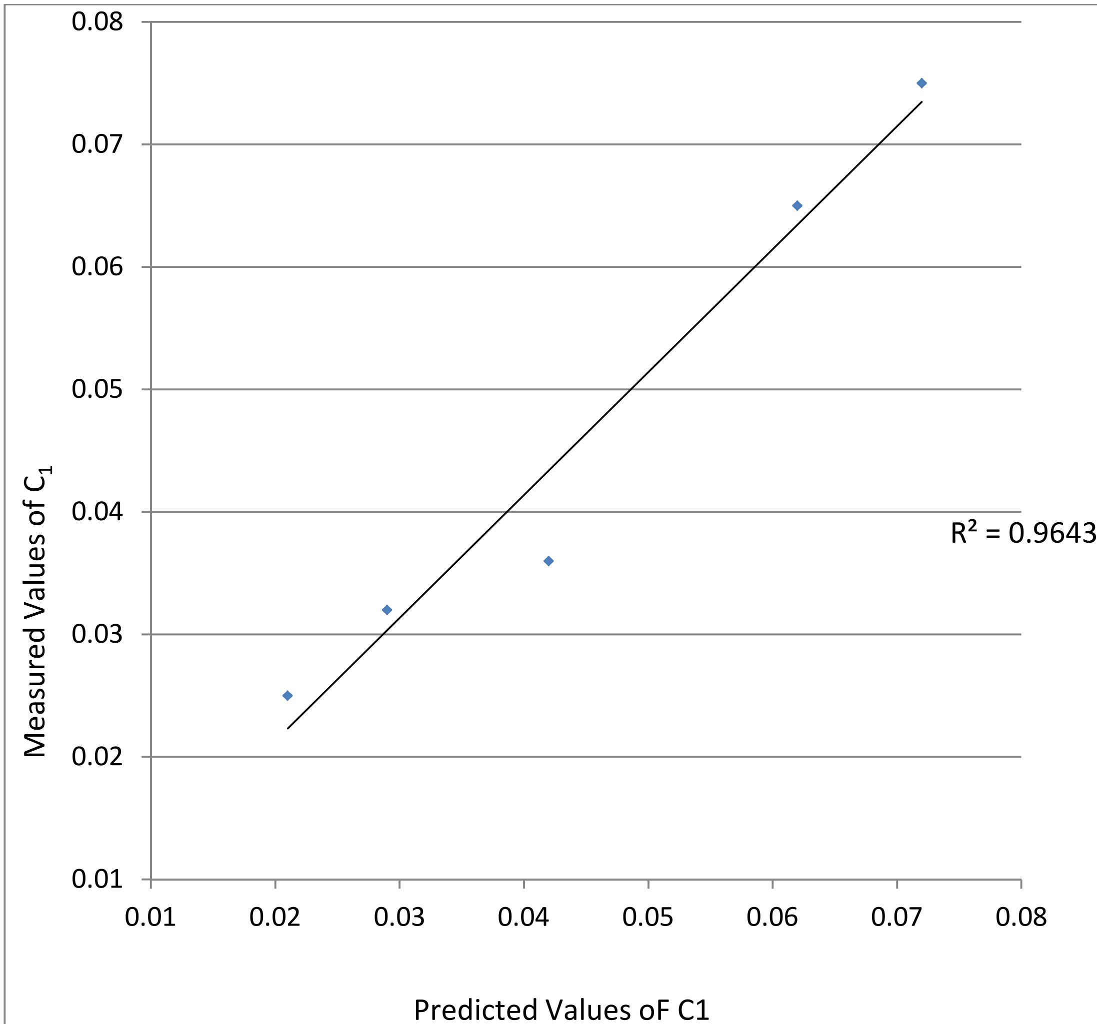


Fig. 4.10 Plot of Mesured values of C_1 against Predicted Values.

4.4 STATISTICAL MODELING OF THE STORAGE FACTOR OF SANDBAGS

An empirical model that relates the major hydraulic variables that affect sediment accumulation with height of sandbags with a dimensionless parameter termed storage factor(S_f).

$$s_f = \frac{S}{QC_0} \qquad 4.20$$

Where C_0 = Initial relative sediment concentration (mg/mg)

S= Storage rate (Rate of sediment accumulation) m^3/s

Q= flow rate (m^3/s)

4.4.1 Objective

The objective was to develope an empirical relationship among the following variables:

(i) Height of sandbags, (h_S),

(ii) Rate of accumulation of sediments (S),

(iii) Flow rate ,(Q), and

(iv) Initial sediment concentration, (C_o)

4.4.2 Assumptions.

(i) One dimensional flow.

(ii) Constant flow Rate.

(iii) Constant Rate of Accumulation of sediments.

4.4.3 Derivation of Equation

Regression analysis was done to study the nature of relationship between the variables. The method of least squares was used to estimate the regression constants. The method is such that the sum of squares of the derivatives of points from the estimated line is minimum (Agunwamba, 2006)

$$min \sum e_1{}^2 = \sum(y_i - a - bx_i)^2 \quad 4.21$$

for $\sum e_1{}^2$ to be minimum:

$$\frac{\partial \sum e_i{}^2}{\partial b} = 0 \quad 4.22$$

$$\frac{\partial \sum e_i{}^2}{\partial a} = 0 \quad 4.23$$

$$a = \frac{\sum y - \sum x \sum xy}{n \sum x^2 - (\sum x)^2} \quad 4.24$$

$$b = \frac{n \sum xy - \sum x \sum y}{n \sum x^2 - (\sum x)^2} \quad 4.25$$

A measure of the degree of the relationship between y and x, called coefficient of correlation r is given by:

$$r \quad = \quad \sqrt{\frac{(n\sum xy - \sum x \sum y)^2}{[n\sum x^2 - (\sum x^2)][n\sum y^2 - (\sum y)^2]}} \qquad 4.26$$

Also, a standard error of estimate (S_{yx}) of y on x which is taken as a measure of the scatter about the regression line of y on x, which is given as

$$S_{yx} \ = \ \sqrt{\frac{\sum(y_1 - y_{est})^2}{n}} \qquad 4.27$$

The relationship between the variables was represented by an exponential function;

$$y = ae^{bx} \qquad 4.28$$

From experimental data, values of S_f and height of sandbags, h_s are tabulated in Table 4.6. A plot of h_s against S_f is presented in Fig 4.10b.The fitting curve equation is:

$$y = 0.058e^{47.28x} \qquad 4.29$$

Thus the relationship becomes;

$$h_s = 0.058e^{47.28S_f} \qquad 4.30$$

The coefficient of correlation, R^2 is 0.935

Table 4.6 Values of Storage factor S_f ($Q = 0.12 m^3/s$, $C_0 = 0.04$)

h_s(m)	0.2	0.25	0.3	0.35
S(m^3/s)	0.00011	0.00013	0.00016	0.00018
s_f	0.0256	0.0325	0.0333	0.0375

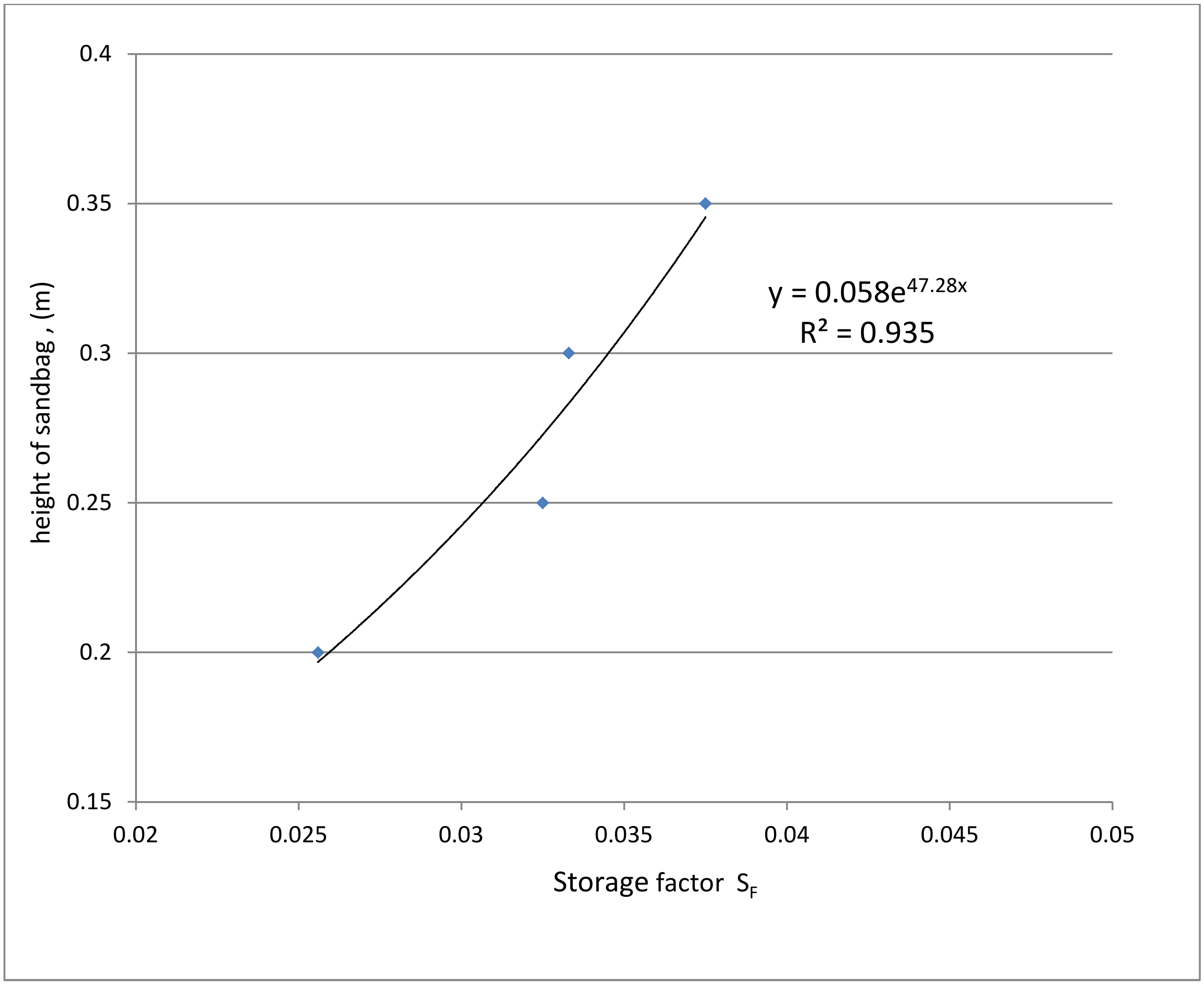


Fig 4.10b Variation of height of sandbags with storage factor

4.4.4 Model Calibration

The data for calibration are as follows;

Q = 0.12m^3/s, C_0 = 0.038, h_s = 0.25, S = 0.00015m^3/s

$$S_f = \frac{0.00015}{0.12 \times 0.042}$$

$$= 0.0298$$

From equation 4.23;

$$h_s = 0.058e^{47.28\ X\ 0.0298}$$

$$h_s = 0.2373$$

Equation 4.21 is adjusted by introducing a calibration parameter β;

$$h_s = 0.058e^{\beta 4\ \ .28\ \ f} \qquad 4.31$$

$$0.25 = 0.058e^{\beta 47.28 \times 0.0298}$$

$$\beta = 1.037$$

Substituting for β in equation 4.24,

$$h_s = \quad 0.058e^{1.037X\ 47.28\ S_f}$$

$$h_s = \quad 0.058e^{49.029S_f} \qquad 4.32$$

Also,

$$S_f = ln\left(\frac{h_s}{0.058}\right)\frac{1}{49.029} \qquad 4.33$$

Equation 4.33 could be used to predict the storage factor for a given height of sandbags. Subsequently, the accumulation rate is computed for given flow rate, Q and initial relative concentration, C_o using equation 4.20.

4.4.5 Model Verification

The set of data for verifying the regression model is presented in Table 4.7. Values of S_f are tabulated alongside measured values of

Q, C_0, and h_s .

Table 4.7 Data for Verification of Model 2

Q	h_s	C_0	S	S_f
0.12	0.2	0.054	0.00015	0.0233
0.12	0.25	0.0375	0.00014	0.0311
0.12	0.3	0.040	0.00016	0.0357
0.12	0.35	0.0425	0.00022	0.0496

Similarly, values of S_f (y) computed from equation 4.26 are presented alongside the estimated values (y_{est}) for corresponding values of Q, C_0, and h_s in Table 4.8. In order to compute the standard error of estimates, it was also necessary to tabulate values of $(y_1 - y_{est})^2$ (see Table 4.8).

Table 4.8 Observed and Estimated Values of Storage factor

h_s	Y	y_{est}	$(y-y_{est})^2$
0.2	0.0233	0.0252	3.61×10^{-6}
0.25	0.0311	0.0297	1.96×10^{-6}
0.3	0.0357	0.0335	4.84×10^{-6}
0.35	0.0372	0.0367	5.0×10^{-6}

A plot of measured values against predicted values is presented in Fig. 4.11. The coefficient of correlation R^2 is 0.966.

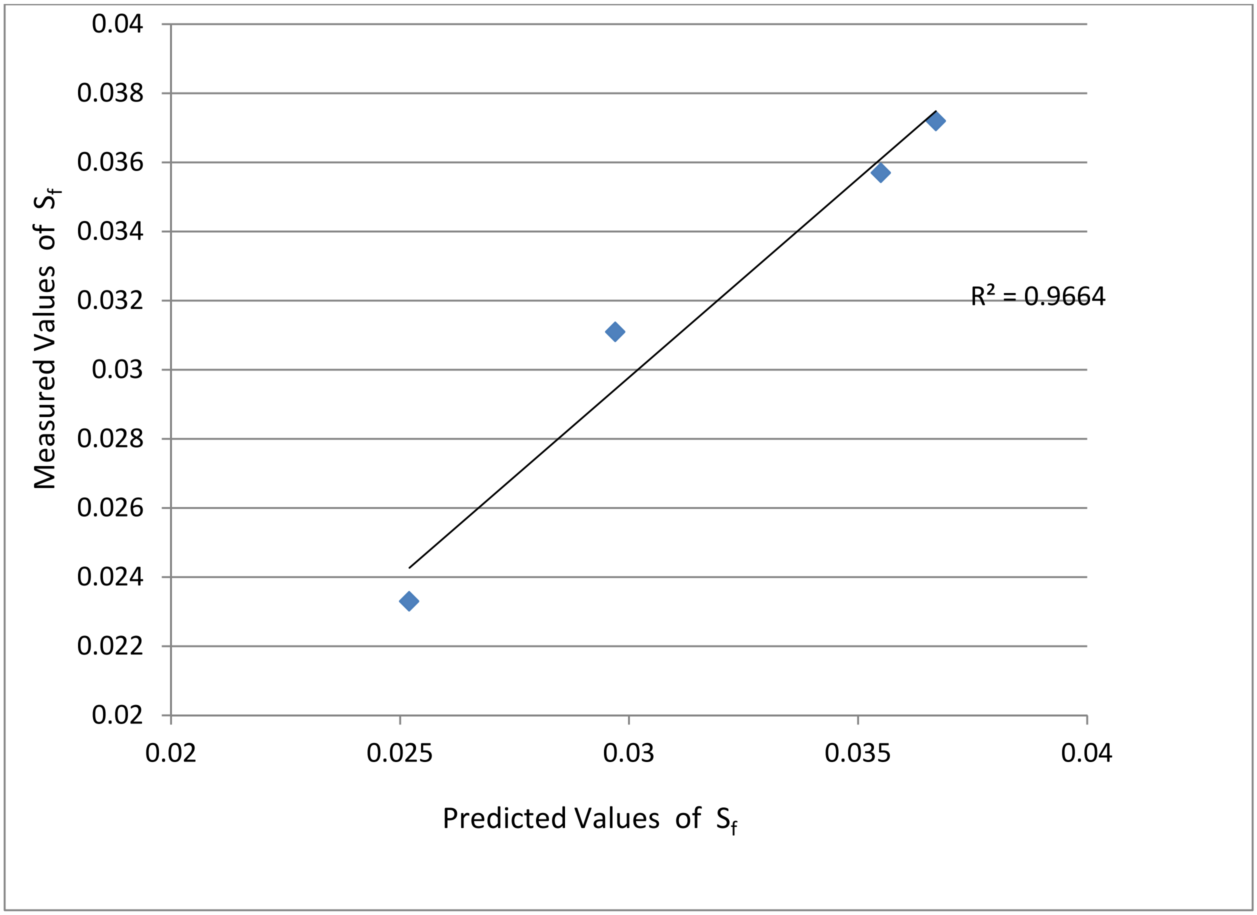
$$\text{Standard error} = \sqrt{\frac{\sum(y_1 - y_{est})^2}{n}} \qquad 4.34$$

where n= No of sample

$$\text{Se} = \sqrt{\frac{1.541\text{x}10^{-5}}{4}}$$

$$= 0.00196$$

The standard error is not significant, indicating closeness of the predicted values with the observed values.



0.04
0.038
0.036
0.034
0.032
0.03
0.028
0.026
0.024
0.022
0.02
Measured Values of S_f
$R^2 = 0.9664$
0.02
0.025
0.03
0.035
0.04
Predicted Values of S_f

4.5 COMPUTER PROGRAMME

In order to aid the application of equation 4.17 in predicting the sediment concentration upstream of Sandbags, a computer programme was developed. The programme is in Microsoft visual basic format. The input variables include C_0, S, t and Q. The programme code is presented in Appendix 8.

4.6 PLACEMENT OF SANDBAGS:

The sand bags should be placed in such a way as to guarantee stability of the bags. They could be stacked like bricks or interlocked on the earth so that the velocity of run-off could be checked. For long term applications, polypropylene or plastic materials could be used.

Typical placement arrangements are illustrated in Fig 4.12. The efficiency of the various arrangements with respect to sediment trapping ability was investigated. It was observed that option ‘A’ with vertical downstream face led to scouring of the area of gully bed on which the runoff drops due to high fall velocity. However, the fall energy could be dissipated by cascading the downstream face as in option ‘B’. On the other hand, option ‘C’ with cascades on both upstream and downstream faces did not ensure uniform deposition of the sediments.

Option ‘B’ therefore guarantees best hydraulic results. The arrangement is consistent with recommendations for placement of Gabions according to Agostine and others (1985) and Blake (1975). It also agrees with the BMP recommendations on the placement of sandbag barriers. Field observations at locations where sandbags were placed indicated irregular dumping of the bags following no particular pattern (see Appendices 10 and 11).The size of the sandbags were also not uniform. In some cases, the sandbags were stacked vertically in an unstable pattern which could easily lead to failure by overturning.

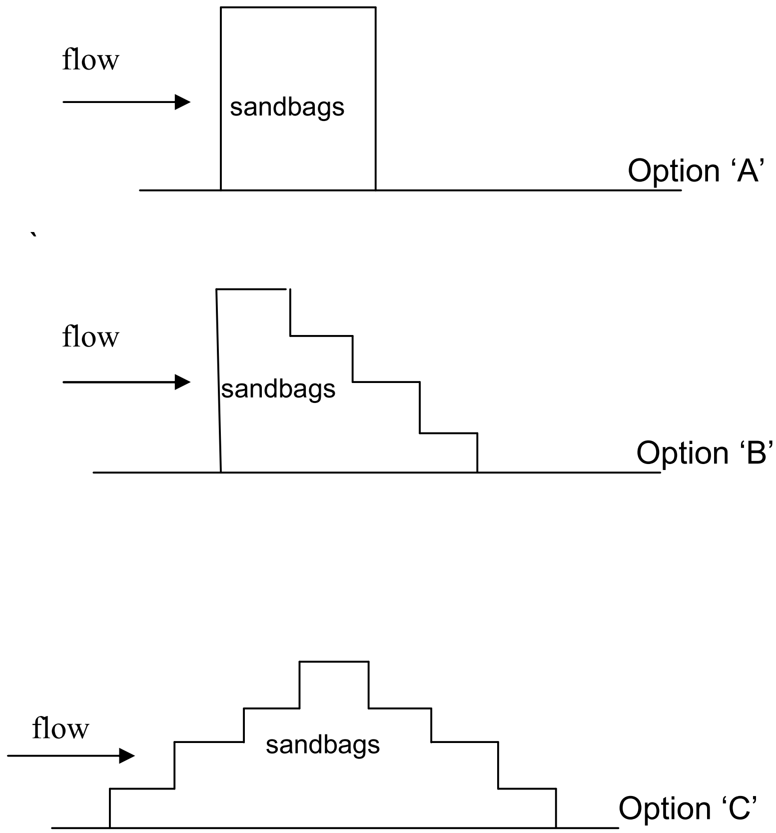


Fig. 4.12 Typical Arrangements of sandbags.

4.7 PROFILE OF SEDIMENT DEPOSITS:

Studies of the profile of the sediments deposited behind the sandbags indicate that the sediments tend to form a horizontal surface at any height as shown in Fig. 4.13. The sediment storage continues until the deposited materials flush with the top level of the bags and no other deposits are made. Thus the length of the sediments deposited would depend on the bed slope, $\theta$of the gully:

$$\frac{h}{l_s} = tan\theta \qquad 4.27$$

Where, h = maximum depth of sediments

θ= bed slope

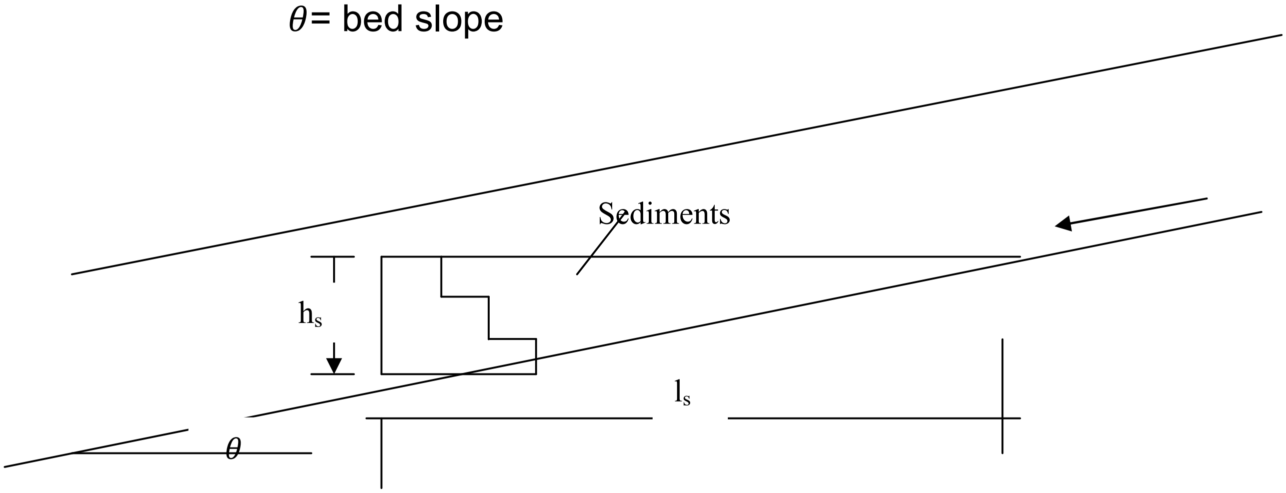


Fig. 4.13. Profile of Sediment deposits Upstream of Sandbags

4.8 STABILITY ANALYSIS

The sandbag units are considered as a massive gravity structure on the foundation. Soil subjected to hydrostatic pressure on the upstream face and vertical forces (weight o the structure, weight of water on the steps and uplift). Thus the design considerations for stability of sandbags are generally the same as for mass gravity structure.

The following factors affecting stability are considered:

a. Unit weights:

i. Water: The density of water varies between 1000 – 1100kg/m^3, but it can reach excess of 2000kg/m^3, depending on its turbidity.

ii. Sand: The unit weight of naturally occurring sand, which is used in filling the sand bag.

iii. Soil: The density of soil depends on the specific gravity of the individual grains, porosity n and the degree of saturation Thus for submerged soil, the unit weight is,

$$\gamma_{sw} = (\gamma_s - \gamma_w)(1 - n) \qquad 4.27$$

b. **Horizontal thrust:**

An illustration of forces acting on submerged Sandbag Checkdam is presented in Fig 4.14.

Hydrostatic pressure, H_a acts on the upstream face.

$$H_a = \frac{1}{2}\gamma_w {h_1}^2 \qquad 4.28$$

Similarly, H_p acts on the downstream face.

$$H_p = \frac{1}{2}\gamma_w {h_2}^2 \qquad 4.29$$

c. Hydraulic Uplift

If a hydrostatic distribution of pressure is considered as acting on the upstream and downstream faces of the structure, the uplift pressure S_w is the resultant of the trapezoidal pressure diagram as illustrated in Fig 4.14,

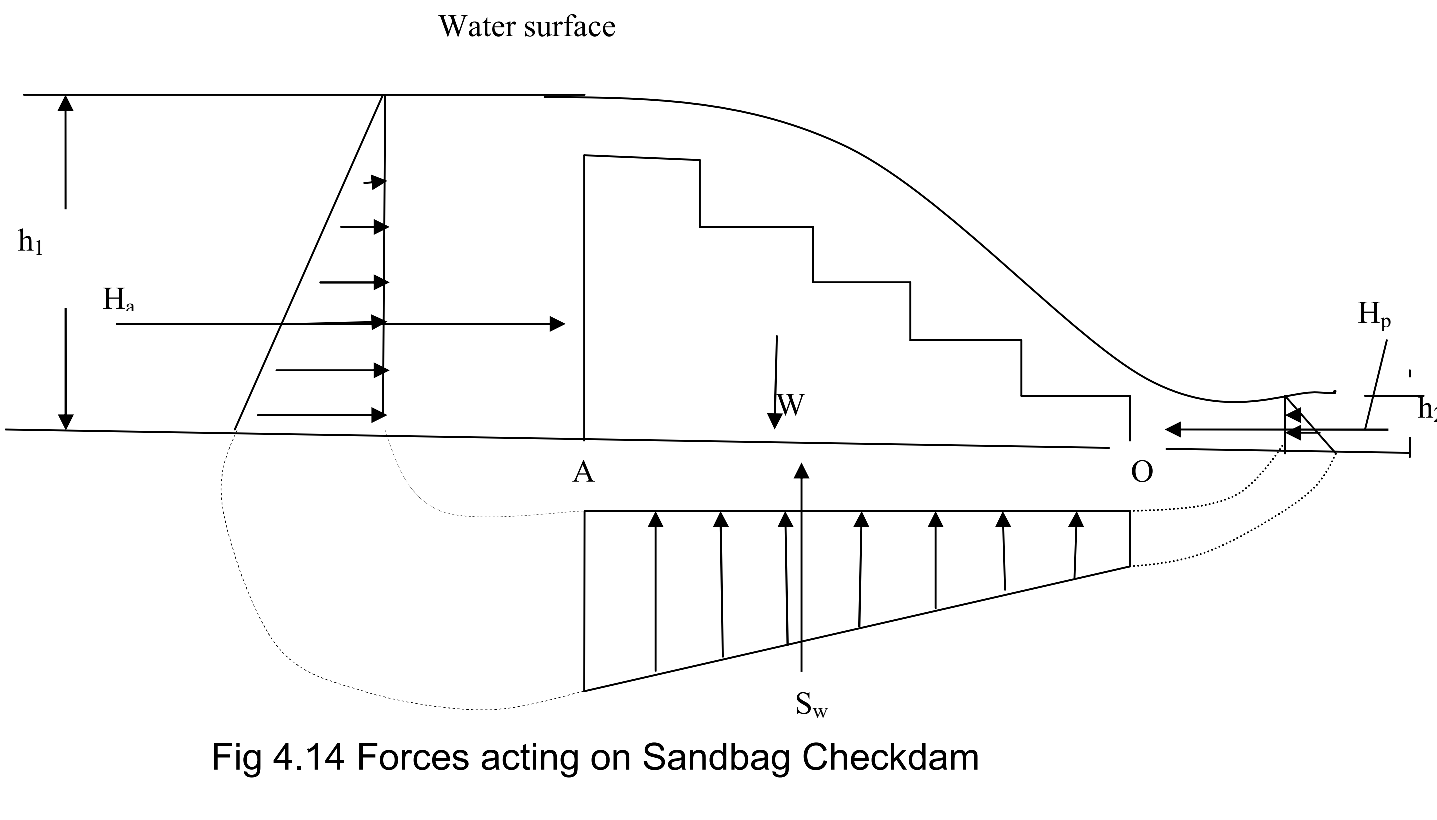


Fig 4.14 Forces acting on Sandbag Checkdam

4.9 DESIGN OF SANDBAG CHECKDAM

4.9.1 Hydraulic Design

Relevant Data; Gully profile (vertical and horizontal) sediment concentration and flow rate.

(a) Height of Sandbag.

A target height is fixed for a particular phase of reclamation. This should be based on sound Engineering judgment. The volume of silt deposited over a rainfall period could be assessed using equation… Based on the result and also considering the gully profile the selected height could be adjusted.

(b) Spacing of Sandbag Locations:

The horizontal distance between the locations of the bags depends on the height of sandbags and the bed slope. The optimum distance between sandbags corresponds with the intersection of a horizontal line from the top of the sandbags with slope. Fig 4.15 shows the relationship between height of sandbags and the horizontal spacing of the bags for different bed slopes. The chart indicate that the intervals for placement increase as the bed slope decreases.

The chart could be applied in design of sandbags for erosion control works. The optimum horizontal spacing could easily be determined for any chosen height of sandbags once the bed slope is known.

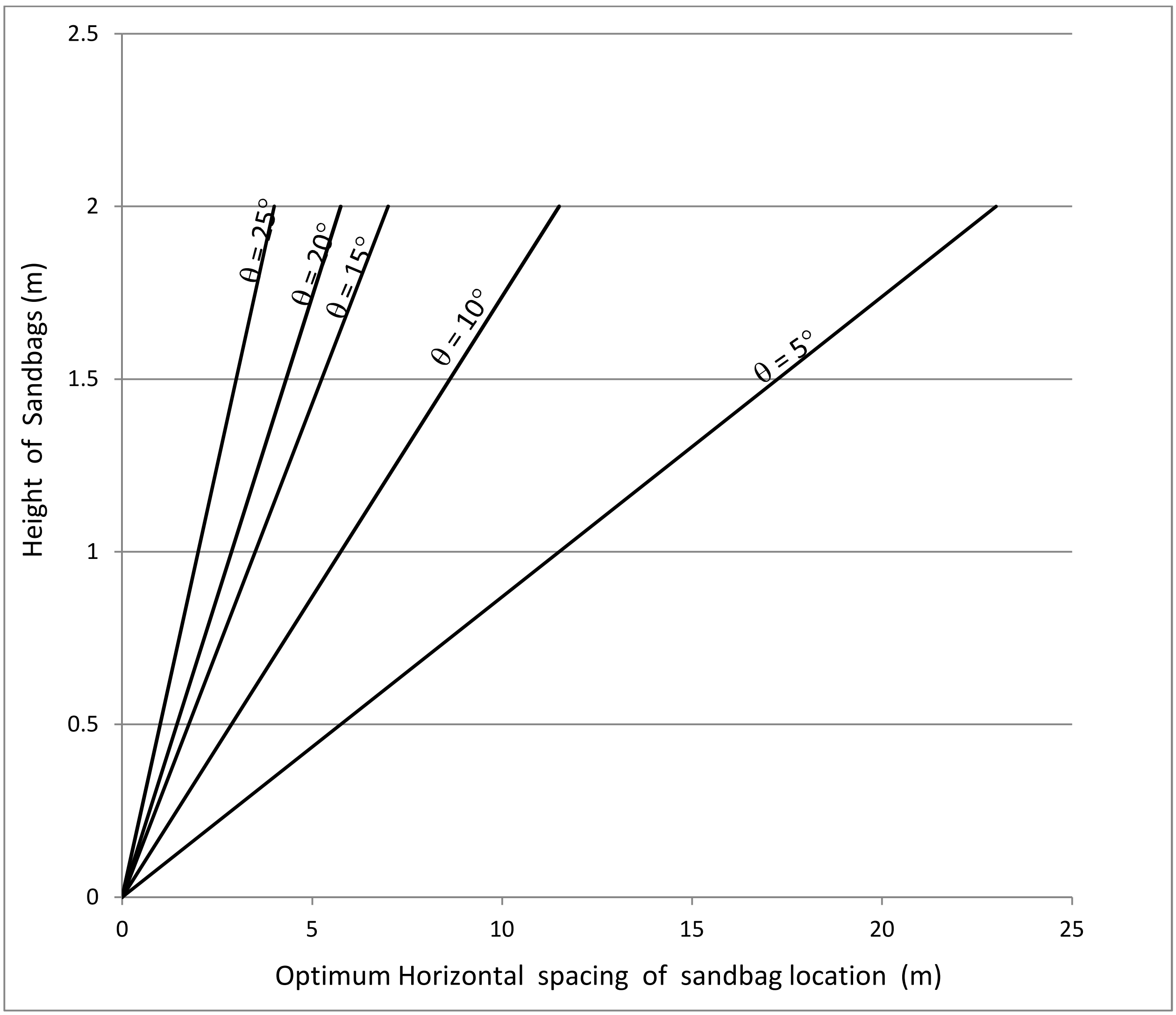


Fig. 4.15 Design Chart for optimum location of Sandbag Checkdams

(b)Seepage Control

Since Sandbag Checkdam causes a rise in upstream head, water tends to seep under and around the structure. The problem is to minimize and control this seepage since if this flow has a velocity capable of removing individual particles of the foundation soil, harmful effects could result.

'Piping' is a phenomenon which occurs when the exit gradient (=head/ length of seepage path) reaches 1 at the point of outflow. This can be reduced by lengthening the seepage path or reducing th head of water causing the seepage (Agunwamba, 2007).

For seepage control, Bligh's formula could be applied. According to this, the total length L of seepage flow under and around structures must be

$$L > c\Delta h \qquad 4.24$$

(Agostini and others, 1985)

where; Δh is the difference between upstream and downstream water surfaces

c is a coefficient depending on the type of soil. Recommended values of c are given in Table 4.9

Table 4.9 Values of coefficient c for control of seepage (Bligh, 1912)

c	Size of particles (mm)	Type of particle
20	0.01- 0.05	Fine silt and mud
18	0.06 -0.10	Coarse silt and very fine sand
15	0.12- 0.25	Fine sand
12	0.30-0.50	Medium sand
10	0.60 -1.00	Coarse sand
9-4	2.0	Gravel
6-3	0.005	Hard clay

4.9.2 Structural Design:

(a) **Layout of bags**

Based on the selected height, a bag layout pattern is chosen. Results of study indicate that transverse interlocking arrangement is ideal. The downstream face should be cascaded to prevent the problem of scouring. The relationship between height of each layer (h_s) and the layer length (l_l) is

$$l_l = 3h_s \qquad 4.25$$

For the polypropelene bags used in the pilot studies h_s varies from 0.25 – 0.35 while h_b is determined from the gully profile.

The relationship between the base width (b) and height of bag (h_b) is

$$b = 3h_b \qquad 4.26$$

Fig. 4.16 illustrates the relationship between height of sandbags, h_b and the weight, W of the sandbags for different unit weights of fill material based on the above arrangement.

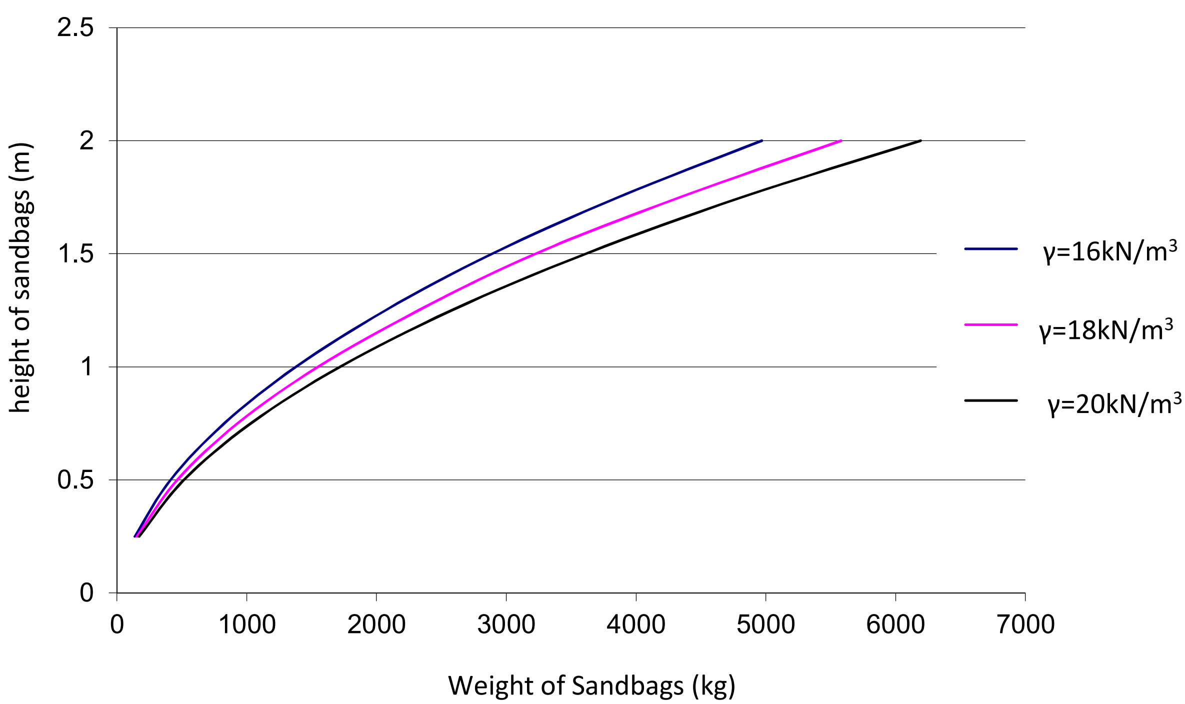


Fig 4.16 Relationship between height (h_b) and Weight (W) of Sandbags

(b) Stability against overturning:

Referring to Fig 4.14, the structure tends to overturn round point O and therefore stability against overturning is ensured when the anti-clockwise moment of stabilizing forces is greater than the clockwise moment of the overturning forces.

The coefficient of stability against overturning is,

$$S_r = \frac{M_S}{M_r} \qquad 4.27$$

Where, M_S = Moment of overturning forces, M_r = Moment of overturning forces. For stability, it is necessary that $S_r \geq 1$, that is , $M_s \geq M_r$;

4.9.3 Foundation Design

The maximum foundation pressure should not be less than the bearing capacity K_t of the soil (See Appendix 13).

It is necessary to check the shearing resistance of the soil to shearing stress. The maximum load which can be applied to the foundation base without causing failure by shear depends on the resistance to shearing stress of the soil τ_f (Agostini , 1985)

$$\tau_f = C + \sigma \tan\emptyset \qquad 4.28$$

Where C = Cohesion in KN/m^2

σ =Maximum pressure in KN/m^2

$\emptyset$= Angle of internal friction

4.10 DESIGN SPECIFICATIONS

(a) Materials:

Sandbags should be woven polypropylene or polyamide fabric, minimum weight 40cm /yd^2m, (ASTM, D3786)

-Biodegradable textiles:

(b) Sizes:

Each sandbag should have a length of 600 – 750mm and width 350 – 450mm. Bag dimensions are nominal and may vary depending on locally available materials.

(c) Fill materials:

All sandbags fill materials should be non–cohesive permeable material free from clay and deleterious material.

(d**) Inspection and maintenance:**

- Regular inspections (weekly during rainy season)
- Ruptured bags should be replaced.

4.11 LIMITATIONS OF SANDBAG CHECKDAM

Based on our field observations, the following limitations were noted on the use of sandbags:

(i) Water going around the bags and eroding the sides.

(ii) Limited durability: Degraded sandbags may rupture, spilling the sand .

To reduce the problem of water going around the sides, it is necessary to turn the ends of bags at the gully walls. That is, placing the sandbags perpendicular as illustrated in Fig 4.17.

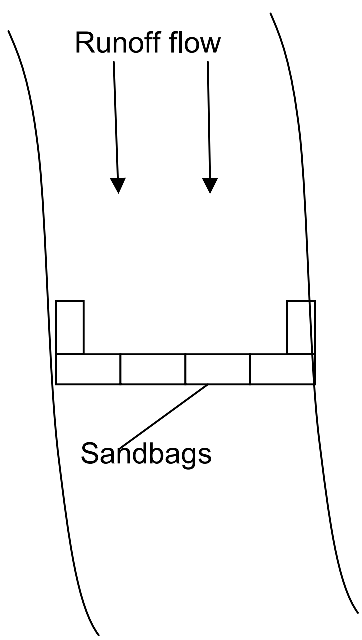


Fig. 4.17 Ideal Layout Of Sandbags

4.12 COST ANALYSIS

A comparative analysis of the cost/unit height of sand bags in a gully and that of conventional concrete check dam was carried out and the results plotted graphically. The basic rates were computed from prevailing market prices of basic materials namely cement, sand, granite, steel rods and wooden formwork. A typical section of reinforced concrete Checkdam is presented in Appendix 14.

Since naturally occurring Soil materials could easily be used for Sandbags, the major cost components are therefore cost of the bags and labour for filling the bags.

$$cost\ of\ unit\ bag = N50.$$

$$Labour\ cost\ for\ filling\ and\ placement = N25$$

$$Total\ cost\ of\ unit\ bag = N75.$$

Based on this rate, the costs/m length for different heights of sandbags are presented alongside the estimates for conventional concrete works in Table 4.10

Table 4.10 Capital Costs of Sandbags and Concrete Checkdams

Height of Sandbag(m)	Costs N/m	
	Sandbags	Concrete
0.5	100	6,500
1.0	750	10,400
1.5	1,750	25,000
2.0	2,750	37,000
2.5	6,250	55,000

The variations of cost with height for conventional concrete check dam and sandbags respectively are presented in Fig 4.17. From the graph, the average cost ratio of sandbags to concrete check dams was found to be 1:12. This is however based only on capital costs, as maintenance costs were not considered over short term applications in small gullies.

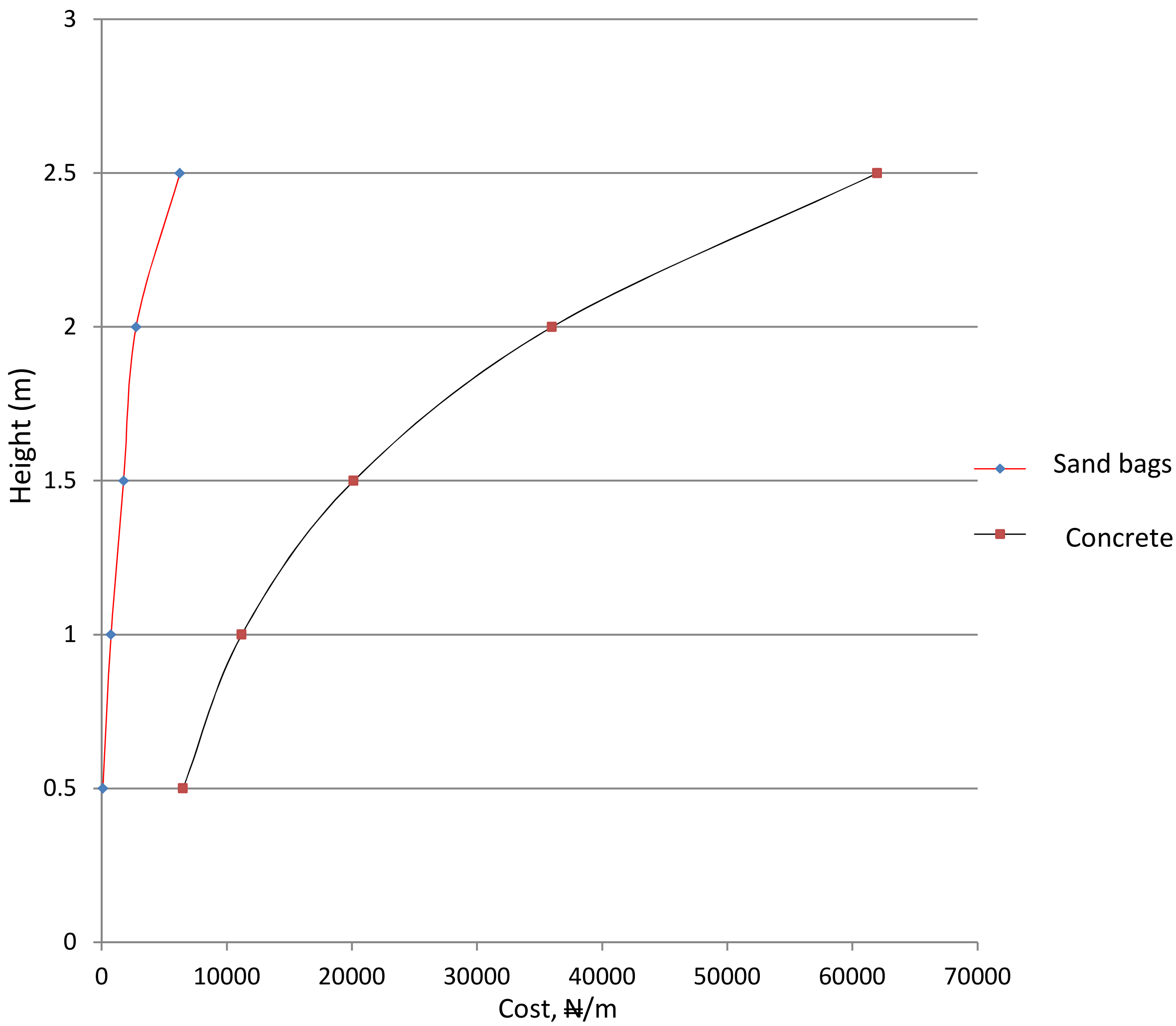


Fig. 4.17 Variation of Cost with Height for Sandbags and Concrete Checkdam

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

The study was focused on the use of sandbags for erosion control. Field studies on the hydraulic performance of sandbags placed in gullies showed that the bags performed efficiently in reducing the speed of runoff resulting in siltation upstream. From the results of the studies, we make the following conclusions:

(i) The concentration of sediments in the runoff upstream of sandbags in a gully at any time interval could be predicted using the mathematical model formulated from the principle of material balance. The model relates the flow rate Q with the relative sediment concentration and sediment accumulation rate.

(ii) The empirical model relating the height of sandbags h_s with the storage factor (S_f) could also be useful in predicting the rate of sediment accumulation for different height of sandbags. The storage factor is a dimensionless parameter relating the flow rate (Q), initial relative concentration C_0, sediment accumulation rate (S) and height of sandbags (h_s)

(iii) Transverse interlocking arrangements of the sandbags yield better results than longitudinal arrangement since results showed that upstream sediment concentration was relatively higher for transverse placement under the same flow conditions.

(iv) The horizontal distance between the locations of the sandbags depends on the chosen height and the bed slope. The optimal distance between sandbags in the gully corresponds with the intersection of a horizontal line from the top of the bags with slope. The design chart relating height of sandbags with optimum horizontal distance for different gully bed slopes is a major contribution of this study.

(v) For short term applications sandbags are relatively economical. A comparative analysis of the capital costs of sandbags and conventional concrete check dam showed a cost ratio of 1:12. This ratio is however bound to reduce in the long term due to anticipated higher operation and maintenance costs of sandbags.

5.2 RECOMMENDATIONS:

Based on the findings of the study and the conclusion above, we make the following recommendations.

i. Data on flow conditions and gully characteristics should be obtained for effective application of the sandbags. A depth – velocity curve developed for a particular gully would be of relevance.

ii. Sandbags should be placed in transverse interlocking arrangement for better efficiency. The downstream face should also be cascaded to prevent the problem of scouring. A bed of sandbags should also be buried at the downstream toe to reduce the impact of falling water on the soil.

iii. Based on stability considerations, a height to base length ratio of 3:1 is recommended for placement of the bags.

iv. Though propylene bags were used for studies, environmentally friendly bio-degradable bags are recommended for large scale application.

v. Further studies should investigate the application of sandbags in deeper gullies. Also a wider range of field data should be used for validation of the models

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Rainfall Intensity duration Curves for Kano

Appendix

Precipitation records .

Date	Rainfall amount (mm)	Duration (hr)	Intensity mm/hr
5/6/08	28	2.85	9.82
15/08/08	17	2.25	7.56
2/9/09	20.3	1hr	20.3
14/9/09	38	2.08	18.3

Appendix

Appendix

Values of C_1 (Case 1:Q = 0.12m^3/s, C_o = 0.02, S = 0.00022 m^3/s)

T =	10 mins	20mins	30 mins	40 mins
C_1	0.05	0.1	0.15	0.2
C_1	0.04	0.05	0.06	0.07
C_1	0.03	0.035	0.045	0.06

Values of C_1 (Case 2: Q = 0.12m^3/s, C_o = 0.036, S = 0.0002 m^3/s)

T	10mins	20mins	30mins	40mins
C_1	0.09	0.23	0.38	0.52
C_1	0.1	0.28	0.41	0.56
C_1	0.06	0.18	0.37	0.42

Appendix

Values of C_1 . (Case 3: Q = 012m^3/s, C_o = 0.036, S = 0.00024 m^3/s)

T	10mins	20mins	30mins	40mins
C_1	0.04	0.19	0.27	0.34
C_1	0.06	0.21	0.32	0.34
C_1	0.1	0.21	0.29	0.33

Values of C_1 (Case 4: Q = 012m^3/s, Co = 0.041, S = 0.00019 m^3/s)

T	10mins	20mins	30mins	40mins
C_1	0.12	0.25	0.39	0.44
C_1	0.16	0.28	0.38	0.51
C_1	0.11	0.156	0.299	0.431

Appendix

Values of C_O and k

.

Appendix