DESIGN OF A WATER RETAINING STRUCTURE (EARTH DAM) FOR IRRIGATION IN GIDAN KWANO

BY

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MATRIC NO.2006/24080EA

BEING A FINAL YEAR PROJECT REPORT SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG.) DEGREE IN AGRICULTURAL & BIORESOURCES ENGINEERING, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGER STATE

NOVEMBER, 2011.

DECLARATION

I hereby declare that this project work is a record of a research work that was undertaken and written by me. It has not been presented before for any degree or diploma or certificate at any university or institution. Information derived from personal communications, published and unpublished work were duly referenced in the text.

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27/02/2012
Date

CERTIFICATION

This is to certify that the project entitled "Design of a Water Retaining structure (Earth Dam) for Irrigation in Gidan Kwano" by Yusuf, Amudat Alabi meets the regulations governing the award of the degree of Bachelor of Engineering (B. ENG.) of the Federal University of Technology, Minna, and it is approved for its contribution to scientific knowledge and literary presentation.

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DEDICATION

I dedicate this project to the Almighty God, The Alpha and Omega, The Author and Finisher of my Faith who in His infinite mercy saw me through my under graduate years inspite of all odds. And to all farmers in Gidan Kwano who with their little support still provide for themselves and their environs.

ACKNOWLEDGEMENT

I want to acknowledge God Almighty for His faithfulness and loving kindness in my life. He is the lifter of my head, the Author and Finisher of our faith for the wisdom, strength and grace He granted me all through my stay at the university and for making this project a reality. My gratitude and thanks also goes to my supervisor Mr John Jiya Musa for his understanding, counsel and unfailing support. I also want to really appreciate my Lecturers in persons of Dr. P. A. Idah (H.O.D), Engr. Dr. O. Chukwu for his fatherly care, Dr. I. E. Ahaneku, you are a father indeed, thanks for your encouragement, guidance and support in the cause of this project work. To my lovely level adviser, Mrs. Bosede Orhevba for your close observations, Engr. M.A. Sedeeq for your labour and support, you are indeed a teacher. Dr. B. A. Alabadan you are a loving father, Engr. S. E. Adebayo, Dr. Mrs. Z.D. Osunde and to mention but a few. I deeply appreciate you all the Lecturers of Agricultural and Bio resources Engineering Department and other Departments of FUT minna for imparting and making my life worthwhile.

My sincere gratitude also goes to the Staffs of Upper Niger River Basin Development Authority Minna, Niger state. Engr. Mamann Sanni, Engr. John Ojo, Engr. Hassan for all of your constructive criticisms which enhanced the quality of this work, you all were more zealous than even me, God bless you richly. Also to Engr. Kabir and Engr. Aminu (Mgr Maizube farms)

I also want to extend my heartfelt gratitude to my beloved parent Engr. A.M. Yusuf and Mrs. Kuburat I. Yusuf for my upbringing, moral and financial support, Mummy, you are the best Mother in the world. My lovely siblings, Engr. Abdulrahaman Yusuf, Usman Yusuf (the builder), Hauwa Yusuf, Sadiyat Yusuf, Fatima Yusuf, Yusuf Yusuf (papa), Aminat Yusuf (Queen my twin sister). And to my friends and loved ones, Lanre Atolagbe, Adesipo

Adebgite (pastor), Kayode Josphe (Pep) you are a pillar, Taiwo Akande, Queen Adamu, Nkem Nwadike, Mary Idakwo, Bimpe Fase, Andrew Efomah, Opeyemi Micheal Kolade (my able class rep.), Emanuel Maduegbuna, Grace Thomas, Oyedika Obudozia, Adetola Adewoye to mention a few, I really appreciate your kind and warm gestures, they were the grand impetus for facing the challenging times of work. Special thanks to all members of the Fellowship of Christian Students (FCS HDP), Uncle Bode Badaki and my Co-ex-executives. I love you all.

I will not fail to deeply appreciate my spiritual parents Mr. Johnny and Mrs. Ajuma Nnawuihe, and Aunty Lucy Shehu and her Husband to be, thanks for your encouragement, love and financial support. God bless and reward you all.

ABSTRACT

The design of water retaining structure (Earth Dam) was carried out in Gidan Kwano Minna Niger State, the design involved collection of Rainfall data, Relative humidity and Evapotranspiration data, Topographical map survey of the study area, estimation of Reservoir storage capacity and the area of the dam, the length of the dam was estimated by tracing the contour lines to graphs. The hydraulic design of the embankment, Trapezoidal spillway design and the crop water requirements were put into consideration. The dam has a height of 20m, crest width of 7m, length of 700m, Total volume of $525,000m^3$, Catchment area of $90km^2$, Peak runoff rate of $0.636m^3/s$, Spillway discharges of $23.99m^3/s$, $59.99m^3/s$ and $4.0m^3/s$ with corresponding Manning coefficient value using 0.025, minimum value of Manning Nigerian developed coefficient using 0.01 and maximum value of Manning Nigerian developed coefficient using 0.15. The dam has a catchment yield of $67080m^3$ with an Earth work volume of $374976m^3$, the seepage through the dam has a value of $5.81\times10^{-7}m^3/s$, and the volume of Sediment likely to occur after a period of 26years is $82,312.35m^3$ which has an Active storage of $442,687.65m^3$. The spillway discharge of $59.99m^3/s$ approximately $60m^3/s$ for this design is best convenient because of the larger volume of spill discharge which will help prevent the dam from failing, hence elongating the dam cycle.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the study

Retaining structures are structures that hold water for future use, they prevent water flow into specific land regions, Retaining structures include Dams, floodgates, Weir, levees (also known as dikes), swimming poll etc. Water stored in a reservoir is used for various purposes such as irrigation, municipal and industrial supply, hydropower and for flood control, retention of debris, navigation etc (Arora, 2007).

Dams are massive barriers built across rivers and streams to confine and utilize the flow of water for human purposes. A dam and a weir are similar, they both retain water for usage. The only dissimilarity between them is that while a weir can serve as a spill structure (overflow weir) in addition to its function as a retaining structure, Dams, especially Earth dam normally have its spill structure separate.

Generally, dams are constructed in the mountain reach of the river where the valley is narrow and the foundation is good, bifurcation is constructed on the downstream of a dam quite away from its boulder reach or alluvial reach of the river to divert the water released from the dam into canals for irrigation and other purposes. Dams are the most important hydraulic structures built on the rivers, which are very huge structures (Arora, 2007).

1.2 Statement of Problem

Lack of sufficient rainfall throughout the year has caused ad vast effect in farming and related water usage, hence reduction in food production for the sustenance of life and low power output in PHCN dams. The needs for construction of dams or weirs to store sufficient water during the period of excess rainfall to meet these needs (enough for food production and power generation) cannot be overemphasized. However, Dams can have devastating effects on rivers, fresh water, ecosystems which directly affects the livelihoods of millions of people, and the aquatic life that live in water. This can be addressed by the environmental input assessment of reservoir which is outside the scope of this study.

1.3 Objectives of the Study

1. To design a water retaining structure (an Earth Dam) for Gidan Kwano
2. To help control water erosion or flooding situations.
3. To design a spillway for the dam using both Manning existing coefficient and the developed Manning Nigeria coefficient and compare the difference.

1.4 Justification of the Study

The inadequate rainfall throughout the year has limited crop production thereby causing food scarcity for man and animals, also the increased and expansion of the Nation's population have created a definite need to develop additional water supply, hence, increase in crop production. The reliable and effective method to obtain these volumes of water needed is to create storage reservoirs (dams). The result of this storage reservoir will go a long way at checking issues of flooding, excessive water loss during the times of abundant rainfall, especially in the month of April to October and also enhance food production from the usage of stored water.

1.5 Scope of the Study

The project entails the Design of a water retaining structure (Earth Dam) using the topographical map of Gidan Kwano campus carried out some years ago, Area Capacity curve was developed and rainfall intensity was also developed using catchment characteristics.

CHAPTER TWO

2.0 LITRATURE REVIEW

2.1 Historical Background

Dams are structures built across a river to create a reservoir on its upstream side for impounding water used for irrigation, municipal and industrial supply, hydropower and recreation. Etc. The word dam can be traced back to middle English and before that from middle Dutch, as seen in the names of many old cities. Adam, L. (2006). Early dam building took place in Mesopotamia and the middle East, dams were used to control the water level for Mesopotamia's weather affected the Tigris and Euphrates rivers and could be quite unpredictable (Donald 1996).

The earliest known dam is Jawa Dam in Jordan, 100kilometers (62 miles) North East of the Capital Amman. This gravity dam featured 4.5m (15ft) high and 1m (3ft 3in) wide stone watt, supported by a 50m (160ft) wide earth rampart. The structure is dated to 3000BC. The Ancient Egyptian Saddel-Kafara Dam at Wadi Al-Gavawi, located about 25km (16miles) South of Cairo was 102m (335ft) long at its base and 87m (285ft) wide. The structure was built around 2800 or 2600 BC as a diversion dam for flood control, but was destroyed by heavy rain during construction or shortly afterwards (Needham and Joseph 2006). By the mid-late third century BC an intricate water management system within Dholavia in modern day Indian was built. The system included 16 reservoirs, dams and various channels for collecting water and storing it.

The Romans were characterized by their ability to construct dam, they came up with the novel concept of large reservoir dams which could secure a permanent water supply for urban settlement also over the dry season. Their pioneering used of water-proof hydraulic mortal and particularly Roman concrete allowed for much larger dam structures. The highest Roman dam was the Subiaco Dam near Rome. Its record height of 50m, it was accidentally destroyed in 1305 (Hodge, 1992).

Roman Engineers made use of ancient standard designs like embankment dams. They displayed a high degree of inventions of basic dam designs which include arch-gravity dams, arch dams, buttress dams and multiple arches. Arch buttress dams all employed in the 2nd century AD. The Kallanai is constructed of stone, over 300m long, 4.5m high and 20m wide, across the main stream of the oldest water diversion or water-regulator structures in the world which is still in use. The purpose of the dam was to divert the water of the Kaveri across the fertile Delta region for irrigation via canals. It is considered to be the oldest dam still in used (Hodge, 2000).

Many large dams also survive at Merida in Spain. The oldest surviving and standing dam in the world us believed to be the Quatinanh barrage in modern-day Syria. The dam is assumed to date back to the Egyptian Pharaoh Sethi (1319-1304BC) and was enlarged in the Roman period between 1934-1938. It still supplies the city of Homs with water. The Kallanai dam is a massive dam of stones over 300m long, 4.5m high and 20m wide across the main stream of the Kaveni river in Indian. The basic structure dates to the 2nd century AD, it purpose was to divert waters of the Kavei across the fertile Delta region for irrigation via canals. Du Jiang Yan is the oldest surviving irrigation system in China. It was finished in 251BC. A large earthen dam made by the prime minister of Chu (state), Sunshu A O, flooded a valley in modern-day Northern Auhui Province that created an enormous irrigation reservoir (62 miles in circumference) which is still present today (Rasheed *et al* 1996).

In the Netherlands, a low–lying country, dams were often applied to block rivers in order to regulate the water level and to prevent the sea from entering the marsh lands. This types of dam often marked the beginning of a town or city because it was easy to cross the river at such a place, it also often gave rise to the respective place's names in Dutch. For example, the Dutch Capital Amsterdam started with a dam through the river Amstel in the late 12^{th} century, and the Rotterdam started with a dam through the river Rotte, a minor tributary of the Nieuwe Maas. A French engineer Benoit Fourneyron developed the first successful water turbine in 1832. The era of large dams initiated after Hoover Dam was completed om the Colorado river near Las Vegas in 1936, by 1997 there were an estimated 800,000dams worldwide about 40,000 of them are over 15m. (James *et al,* 2002)

2.2 Classifications of Dams

1. Classification based on hydraulic design.
2. Classification based on function served.
3. Classification based on rigidity.
4. Classification based on material used.
5. Classification based on structural behavior. Etc

2.2.1 Classification Based on Hydraulic Design

On the basis of hydraulic design, dams may be classified as over-flow dams and non-overflow dams (Sturm, 2001).

Over flow dams: An overflow dam is designed to act as an overflow structure. The surplus water which cannot be retained in the reservoir is allowed to pass over the crest of an over flow dam which act as a spillway. The over flow dam is made of a material which does not erode by the action of the overflowing water. Generally, cement concrete is used in over flow dams and spillways. Gravity dams have over flow sections for some length and the rest of the length as non-over flow dam. Though, sometimes the entire length of the dam of low height is designed as an overflow dam. The over flow dam is also called the spillway section.

I. **Non overflow Dam:** A non-over flow dam is designed such that there is no flow over it because there is no overflow. A non- overflow dam can be built of any material, such as concrete, masonry, earth, rock-fill and timber. The non-over flow dam is usually provided in the part of the total length of the dam however, it may be provided for the entire length and a separate spillway is provided in the flanks or in a saddle away from the dam (Arora 2005).

2.2.2 Classification Based on Function Served

Depending upon the function served by the dams, the dams can be classified as follows

I. Storage Dams: Storage dams are constructed to store water during the raining season when there is a large flow in the river. The stored water is utilized later during the period when the flow in the river is reduced and is less than the demand. Storage dams are the most common type of dams and in general dams means a storage dam unless otherwise qualified.

II. Detention Dams: They are constructed for flood control, it retards the flow in the river on it's downstream during floods by storing some food waters. Thus the effect of sudden floods is reduced to some extent. The water retained in the reservoir is later released gradually at a control rate according to the carrying-capacity of the channel downstream of the detention dam and the area downstream of the detention dam is protected against flood.

III. Diversion Dams: The diversion dam is constructed for the purpose of diverting water of the river into an off-taking canals or a conduit. The diversion dam is usually of low height and has a small storage reservoir on it's upstream. It is a sort of storage weir which also divert water and has a small storage.

IV. Debris Dams: These are dams constructed to retain debris such as sand, gravel, and drift wood flowing in the river with water.

V. Coffer Dams: A coffer dam is not actually a dam but an enclosure constructed around the construction site to exclude water so that the construction can be done dry. A coffer dam is thus a temporary dam constructed for facilitating construction. It is usually constructed on the upstream of the main dam to divert water into a diversion tunnel during the construction of the dam. (Arora, 2007)

2.2.3 Classification Based on Rigidity

Dams are classified on the basis of rigidity as :

I. Rigid Dams: the rigid dam is quite stiff, it is constructed of stiff materials such as concrete, masonry, steel and timber. These dams deflect and deform very little when subjected to water pressure and other forces.

II. Non-rigid dams: A non-rigid dam is relatively less stiff compared to a rigid dam. The dams constructed of earth and rock fill are non-rigid dams. There are relatively large settlements and deformations in a non-rigid dam (Herzog, 2009)

2.2.4 Classification Based on Materials of Construction

Based on the materials used in construction, dams are classified as follows:

I. Masonry Dam
II. Concrete Dam
III. Earth Dam
IV. Rock-fill Dam
V. Timber Dam
VI. Steel Dam
VII. Combine Concrete-cum-Earth Dam: In this type of dam, a part of the length is constructed as an earth dam and the rest a concrete dam. However, if the concrete section is only for the overflow portion (or the spillway section) and the remaining length as an earth dam, is classified as a concrete-cum-earth dam.
VI. Composite Dam: a composite dam has a section which consists of two materials. Generally, a composite dam has some portion as rock-fill and some portion as earth-fill in the same direction (Arora, 2007)

2.3 Types of Dams

Based on the structural action, dams are classified as follows:

1. Gravity Dams
2. Earth Dams
3. Rock-fill Dams
4. Arch Dams
5. Buttress Dams
6. Steel Dams
7. Timber Dams

2.3.1 Gravity Dams

A gravity dam is a solid structure made of concrete or masonry, constructed cross a river to create a reservoir on its upstream. The section of the gravity dam is approximately triangular in shape with its apex at its top and maximum width at bottom. The section is proportioned such that it resist the various forces acting on it by its own weight. A gravity dam is also called a solid dam to distinguish it from hollow gravity dam in which hollow spaces are kept to reduce the weight. Early gravity dams are built of masonry, but most of the modern gravity dams are made of concrete.

A gravity dam resists the water pressure and other forces due to its weight (or gravitational forces). Thus, the stability of gravity dam depends upon its weight. The gravity dam are usually made of cement concrete. They are generally straight in plan (i.e. axis is straight from one abutment to the other) and are called straight gravity dams. However, sometimes they are slightly curved in plan, with convexity towards the upstream and are called curved gravity dams. The gravity dams are approximately triangular in cross section with apex at the top.

The gravity dam are generally more expensive than the earth dams but, they are more durable. They are quite suitable for the gorges with very steep slopes. They require strong rock foundation. However, if the foundation consists of soil, the height of the gravity dams is usually limited to 20m or so (Arora 2007).

2.3.1.1 Forces Acting on Gravity Dam

A gravity dam is subjected to the following main forces

- Weight of the dam
- Water pressure
- uplift pressure
- wave pressure
- silt pressure
- ice pressure
- wind pressure
- earthquake forces

2.3.2 Earth Dams

An earth dam is made of earth (or soil). It resists the forces exerted upon it by mainly due to shear strength of the soil. Although the weight of the earth dam also helps in resisting the forces, the structural behavior of an earth dam is entirely different from that of a gravity dam.. the earth dams are usually built in wide valleys having that slopes at flanks (Abutments). The foundation requirements are less stringent than those of gravity dams, and hence they can be built on all types of foundations. However, the height of the dam will depend upon the strength of the foundation material (Arenillas, 2003)

The section of an earth dam can be homogeneous when the height of the dam is not great. Generally, the earth dams are of zoned sections, with an impervious (called core) in the middle and relatively pervious zones (called shells or shoulders) enclosing the impervious zone on both sides. If the earth dam is built on a pervious foundation, a concrete cutoff wall or a steel sheet pile line on the downstream to carry away the water that seeps through the dam and itsd foundation earth dams are usually cheaper than the gravity dams if suitable earth in abundant quantity is easily available near the site.

Earth dams can be divided into the following three types depending upon the section of the dam

1) Homogeneous earth dams
2) Zoned earth dams
3) Diaphragm earth dams

2.3.3 Rock fill Dams

A rock fill dam is built on fragment (called rock) and boulders of large size. An impervious membrane is placed on he rock fill on the upstream side to reduce the seepage through the dam. The membrane is usually made of cement concrete or asphaltic concrete. In early rock fill dams, steel and timber membrane ere also used, but now they are obsolete a dry rubble cushion is placed between the rock-fill and the membrane for the distribution o water load and for providing a support to the membrane.

Sometimes, the rock fill dams have an impervious earth core in the middle to check the seepage instead of an impervious upstream membrane. The earth core is placed against a dumped rock fill. It is necessary to provide adequate filters between the earth core and he rock fill on the upstream and downstream sides of the core so that the soil particles are not carried by water and piping does not occur. The sides slopes of rock fill are usually kept equal to the angle of repose of rock.

Rock fill dams require foundation stronger than those for earth dams. However, the foundation requirements are usually less stringent than those for gravity dams. Rock fill dams are quite economical when a large quantity of rock is easily available near the site (Chanson *et al* 2000)

2.3.4 Arch Dams

An arch dam is curved in plain, with its convexity towards the upstream side. An arch dam transfers the water pressure and other forces mainly to the abutments by arch action. An ach dam is quite suitable for narrow canyons with strong flanks which are capable of resisting the thrust produced by the arch action. The section of an arch dam is approximately triangular like a gravity dam but the section is comparatively thinner. The arch dam may have a single curvature or double curvature in the vertical plane. Generally the arch dam of double curvature are more economical and are used in practice. The arch dam require good quality concrete for resisting stresses, the quality of concrete required in an arch dam is less than that of a gravity dam. The arch dams are subjected to large stresses because of changes in temperature, shrinkage of concrete and yielding of abutments.(Herzog 2009)

2.3.4.1 Types of Arch Dams

The following are the different types of arch dams identified by (Chanson et al 2000)

1. Single curvature-arch dams

   Single curvature arch dam is curved only in a plane because it has a curvature only in a horizontal plane. It is sub-divided into three types, namely:

   - Constant radius arch dams
   - Variable-radius arch dams
   - Constant-angle arch dams

2. Double curvature arch dams: a double curvature arch dam is curved both in plane and in elevation, as it has curvature both in the horizontal and vertical planes. It is also called cupola arch dam.
3. Arch-gravity dam: an arch gravity dam is a thick arch dam which behaves structurally partly as a gravity dam and partly as an arch dam.

2.3.5 Buttress Dams

Buttress dams are of three types:

1. Deck type: A deck type buttress can consists of a sloping supported by buttress are triangular concrete walls which transmit the water pressure from the deck lab to the foundation. Buttresses are compression members. The deck is usually a reinforced concrete slab supported between the buttresses, which are equally spaced.
2. Multiple arch dam: in a multiple arch type buttress dam, the deck slab is replaced by horizontal arches supported by buttresses. The arches are usually of small span and made of concrete.

3. Massive-head type dam: In a massive-head type buttress dam there is no deck slab instead of the deck, the upstream edges of the buttresses are flared to form massive heads which span the distance between the buttresses.

I. The buttress dam requires less concrete than gravity dam but, they are not necessarily cheaper than the gravity dam because of extra cost of form work, reinforcement and more skilled labour. The foundation requirements of buttress dams are usually less stringent than those of the gravity dam (Arora, 2007)

2.3.6 Steel Dams

A steel dam consist of a steel frame work with a steel skin plate on it upstream face. Steel dams are generally of two types

- Direct-strutted steel dams: Here the water pressure is transmitted directly to the foundation through inclined struts.
- Cantilever type steel dam: in cantilever steel dam, there is a bent supporting the upper part of the deck which is formed into a cantilever truss. This arrangement introduces a tensile force in the deck girder which can be care f by anchoring it into the foundation at the upstream toe.

Steel dams are costly and are subjected to corrosion; they are sometimes used as temporal coffer dams during the construction of the permanent dams. They are supplemented with timber or earth-fill on the inner side to make them water tight. The area between the coffer dams are dewatered so that the construction may be done in dry for the permanent dam.

2.3.7 Timber Dams

A timber dam consist of frame work made of timber with a facing of timber planks, the framework is comprised of struts and beams. It transfers the water pressure on the upstream planks to the foundation. Timber dams are mainly of three types

I. A-frame type: The A-frame type timber dam is built of timbers and planks making the shape of English letter A. its stability depends upon the weight of the water on the deck and upon anchorage of sills.

II. Rock-filled crib type: In this type of timber dam, cribs of timber members are drift-bolted together the timber members re generally o round or square section and are placed at 2.5m centers in both directions. The bottom members of the cribs are generally pinned to the rock foundation, the space between the various members is filled with rock fragment or boulders to give stability. A top plank is then placed on the crib.

III. Beaver type: A beaver type consist of timber members of round section forming a bent, the butts of all the timber member point downstream. Spacer logs are placed between the butts and drift-pined to the other logs, the tips of the timber members pointing to the upstream are also drift-pinned together. The bottom members are fixed to the foundation with anchor bolts (Hodge *et al* 2000)

2.4 Head Dyke

I. Head dyke is a water control structure usually of a rectangular weir type equipped with wooden shutters (planks) for regulating the water level behind the dam. It consists of crest, apron, cut-off and wings. (Arora, 2007)

2.5 Weirs

A weir is a small overflow dam used to alter the flow characteristics of a river or stream. In most cases weirs take the form of a barrier across the river that causes water to pool behind the structure (not unlike a dam), but allows water to flow over the top. Weirs are commonly used to alter the flow regime of the river, prevent flooding, measure discharge and to help render a river navigable (Herzog *et al* 2009)

2.5.1 Functions of Weirs

Weirs allow hydrologists and engineers a simple method of measuring the volumetric flow rate in small to medium-sized streams, or in industrial discharge locations. Since the geometry of the top of the weir is known, and all water flows over the weir, the depth of water behind the weir can be converted to a rate of flow. The calculation relies on the fact that fluid will pass through the critical depth of the flow regime in the vicinity of the crest of the weir. If water is not carried away from the weir, it can make flow measurement complicated or even impossible.

The discharge can be summarized as

$$Q = CLH^n$$

Where

- Q is flow rate
- C is a constant for structure
- L is the width of the crest
- H is the height of head of water over the crest
- n varies with structure (e.g. 3/2 for horizontal weir, 5/2 for v-notch weir)

A weir may be used to maintain the vertical profile of a stream or channel, and is then commonly referred to as a *grade stabilizer* such as the weir in Duffield, Derbyshire.

The crest of an overflow spillway on a large dam is often called a *weir*.

Weirs, referred to as low head barrier dams in this context, are used in the control of invasive sea lamprey in the Great Lakes. They serve as a barrier to prevent decolonization by lamprey above the weir, reducing the area required to be treated with lampricide, and providing a convenient point to measure water flow (to calculate amount of chemical to be applied).

Mill ponds provide a watermill with the power it requires, using the difference in water level above and below the weir to provide the necessary energy (Chanson, 2009)

2.5.2 Drawbacks Weirs

- Because a weir will typically increase the oxygen content of the water as it passes over the crest, a weir can have a detrimental effect on the local ecology of a river system. A weir will artificially reduce the upstream water velocity, which can lead to an increase in siltation.

- Weirs can also have an effect on local fauna. While a weir is easy for some fish to jump over, other species or certain life stages of the same species may be blocked by weirs due to relatively slow swim speeds or behavioral characteristics. Fish ladders provide a way for fish to get between the water levels.
- Even though the water around weirs can often appear relatively calm, they can be extremely dangerous places to boat, swim, or wade as the circulation patterns on the downstream side-- typically called "hydraulics"-- can submerge a person indefinitely.
- This phenomenon is so well-known to canoeists, kayakers, and others who spend time on rivers that they even have a rueful name for weirs: "drowning machines".[1]

II. The weir can become a point where garbage and other debris accumulate. However, a walkway over the weir is likely to be useful for the removal of floating debris trapped by the weir, or for working staunches and sluices on it as the rate of flow changes. This is also sometimes used as a convenient pedestrian crossing point for the river (Arora, 2007).

2.5.3 Types of Weirs

There are different types of weir. It may be a simple metal plate with a V-notch cut into it, or it may be a concrete and steel structure across the bed of a river (Gonzalez, 2007).

2.5.3.1 Broad-crested Weir

A broad-crested weir is a flat-crested structure, with a long crest compared to the flow thickness (Chanson 1999, 2004, Henderson 1966, Sturm 2001). When the crest is "broad", the streamlines become parallel to the crest invert and the pressure distribution above the crest is hydrostatic. The hydraulic characteristics of broad-crested weirs were studied during the 19th and 20th centuries. Practical experience showed that the weir overflow is affected by the upstream flow conditions and the weir.

2.5.3.2 Sharp crested Weir (fayoum weir)

I. A sharp-crested weir allows the water to fall cleanly away from the weir. Sharp crested weirs are typically 1/4" or thinner metal plates. Sharp crested weirs come in many different shapes such as rectangular, V-notch and Cipolletti weirs. (Arora, 2007)

2.5.3.3 Combination Weir

The sharp crested weirs can be considered into three groups according to the geometry of weir:

a) the rectangular weir,

b) the V or triangular notch and

c) special notches, such as trapezoidal, circular or parabolic weirs.

For accurate flow measurement over a wider range of flow rates, a combination weir combines a V-notch weir with a rectangular weir. An example is manufactured by Thel-Mar Company and has flow rates engraved along the side of the weir. This is typically used in pipes ranging from 4" to 15" in diameter.

2.5.3.4 V-notch Weir

The V-notch weir is a triangular channel section, used to measure small discharge values. The upper edge of the section is always above the water level, and so the channel is always triangular simplifying calculation of the cross-sectional area. V-notch weirs are preferred for low discharges as the head above the weir crest is more sensitive to changes in flow compared to rectangular weirs. (Arora, 2007)

2.5.3.5 Minimum Energy Loss Weir

The concept of the Minimum Energy Loss (MEL) structure was developed by Gordon McKay in 1971. The first MEL structure was the Radcliffe storm waterway system, also called Humpybong Creek drainage outfall, completed in 1960 in the Radcliffe peninsula (Australia). It consisted of MEL weir acting as a streamlined drop inlet followed by a 137 m long culvert discharging into the Pacific Ocean. The weir was designed to prevent beach sand being washed in and choking the culvert, as well as to prevent salt intrusion in Humpybong Creek without afflux. The structure is still in use and passed floods greater than the design flow in several instances without flooding (Chanson 2007).

The concept of the Minimum Energy Loss (MEL) weir was developed to pass large floods with minimum energy loss and afflux, and nearly-constant total head along the waterway. The flow in the approach channel is contracted through a streamlined chute and the channel width is minimum at the chute toe, just before impinging into the downstream natural channel. The inlet and chute are streamlined to avoid significant form losses and the flow may be critical from the inlet lip to the chute toe at design flow. MEL weirs were designed specifically for situations where the river catchment is characterized by torrential rainfalls and by very small bed slope. The first major MEL weir was the Clermont weir (Qld, Australia 1963), if the small control weir at the entrance of Redcliffe culvert is not counted. The largest, Chinchilla weir (Qld, Australia 1973), is listed as a "large dam" by the International Commission on Large Dams (Arora, 2007)

2.6 Spillways

A spillway is a section of a dam designed to pass water from the upstream side of a dam to the downstream side. Many spillways have floodgates designed to control the flow through the spillway. Types of spillway include: A service spillway or primary spillway passes normal flow. An auxiliary spillway releases flow in excess of the capacity of the service spillway. An emergency spillway is designed for extreme conditions, such as a serious malfunction of the service spillway. A fuse plug spillway is a low embankment designed to be over topped and washed away in the event of a large flood. Fuse gate elements are independent free-standing block set side by side on the spillway which works without any remote control. They allow increasing normal pool of the dam without compromising the security of the dam because they are designed to be gradually evacuated for exceptional events. They work as fixed weir most of the time allowing over spilling for the common floods (Britannica, 2001)

2.7 Dam failure

Dam failures are generally catastrophic if the structure is breached or significantly damaged. Routine deformation and monitoring of seepage from drains in and around larger dams is useful to anticipate any problems and permit remedial action to be taken before structural failure occurs. Most dams incorporate mechanisms to permit the reservoir to be lowered or even drained in the event of such problems. Another solution can be rock grouting - pressure pumping Portland cement slurry into weak fractured rock.

During an armed conflict, a dam is to be considered as an "installation containing dangerous forces" due to the massive impact of a possible destruction on the civilian population and the environment. As such, it is protected by the rules of International Humanitarian Law (IHL) and shall not be made the object of attack if that may cause severe losses among the civilian population. To facilitate the identification, a protective sign consisting of three bright orange circles placed on the same axis is defined by the rules of IHL.

The main causes of dam failure include inadequate spillway capacity, piping through the embankment, foundation or abutments, spillway design error (South Fork Dam), geological instability caused by changes to water levels during filling or poor surveying (Vajont Dam, Malpasset, Testalinden Creek Dam), poor maintenance, especially of outlet pipes (Lawn Lake Dam, Val di Stava Dam collapse), extreme rainfall (Shakidor Dam), and human, computer or design error (Buffalo Creek Flood, Dale Dike Reservoir, Taum Sauk pumped storage plant) (Goldstein 2011)

2.8 Purpose of Dam Creation

Functions	Example
Power generation	Hydroelectric power is a major source of electricity in the world. Many countries have rivers with adequate water flow, that can be dammed for power generation purposes. For example, the Itaipu Dam on the Paraná River in South America generates 14 GW and supplied 93% of the energy consumed by Paraguay and 20% of that consumed by Brazil as of 2005.
Water supply	Many urban areas of the world are supplied with water abstracted from rivers pent up behind low dams or weirs. Examples include London - with water from the River Thames and Chester with water taken from the River Dee. Other major sources include deep upland reservoirs contained by high dams across deep

	valleys such as the Claerwen series of dams and reservoirs.
Stabilize water flow/irrigation	Dams are often used to control and stabilize water *flow*, often for agricultural purposes and irrigation. Others such as the Berg Strait dam can help to stabilize or restore the water *levels* of inland lakes and seas, in this case the Aral Sea
Flood prevention	Dams such as the Blackwater dam of Webster, New Hampshire and the Delta Works are created with flood control in mind
Land reclamation	Dams (often called dykes or levees in this context) are used to prevent ingress of water to an area that would otherwise be submerged, allowing its reclamation for human use.
Water diversion	A typically small dam used to divert water for irrigation, power generation, or other uses, with usually no other function. Occasionally, they are used to divert water to another drainage or reservoir to increase flow there and improve water use in that particular area.
Navigation	Dams create deep reservoirs and can also vary the flow of water downstream. This can in return affect upstream and downstream navigation by altering the river's depth. Deeper water increases or creates freedom of movement for water vessels. Large dams can serve this purpose but most often weirs and locks are used.
Recreation ınd aquatic)eauty	Dams built for any of the above purposes may find themselves displaced by time of their original uses. Nevertheless the local community may have come to enjoy the reservoir for recreational and aesthetic reasons. Often the reservoir will be placid and surrounded by greenery, and convey to visitors a natural sense of rest and relaxation.

(US Army Corps of Engineers 2008)

2.9 Design Criteria for Earth Dams

1. A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost. Borrow pits should be as to the dam site as possible, so as to reduce the leads.
2. Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during flood.
3. Sufficient freeboard must be provided for wind set-up, wave action and frost action.
4. The seepage line (i.e phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.
5. There should be no possibility of free flow of water from the upstream to the downstream face.
6. The upstream face should be properly protected against wave action, and the downstream face against rains and against wave up to tail water. Provisions of horizontal beams at suitable intervals in the downstream face may be thought of, so as to reduce the erosion due to rain water. Riprap should be provided either on the upstream slope and also on the downstream slope near the toe and up to slightly above the tail water so as to avoid erosion.
7. The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain etc.
8. The upstream and downstream slopes should be so designed as to be stable under worst conditions of loading. These critical conditions occur for the u/s slope during sudden

drawdown of the reservoir, and for the downstream slope during steady seepage under full reservoir.

9. The upstream and downstream slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength of the soil, so as to provide a suitable factor of safety.
10. Since the stability of the embankment and foundation is very critical during construction or even after construction (i.e during the period of consolidation), due to development of excessive pore pressures and consequent reduction in shear strength of soil, the embankment slopes must remain safe under this critical condition also.
11. All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of an earth dam (Britannica, 2001)

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Study Area

The Federal University of Technology permanent site is known to have a total land mass of eighteen thousand nine hundred hectares (18,900 ha) which is located along kilometer 10 Minna – Bida Road, South – East of Minna under the Bosso Local Government Area of Niger State. It has a horse – shoe shaped stretch of land, lying approximately on latitude of 09^0 35' N and longitude of 06^0 28' E. The site is bounded at Northwards by the Western rail line from Lagos to the northern part of the country and the eastern side by the Minna – Bida Road and to the North – West by the Dagga hill and river Dagga. The entire site is drained by rivers Gwakodna, Weminate, Grambuku, Legbedna, Tofa and their tributaries. They are all seasonal rivers and they eventually drain into River Dagga which is the major River in the catchment.

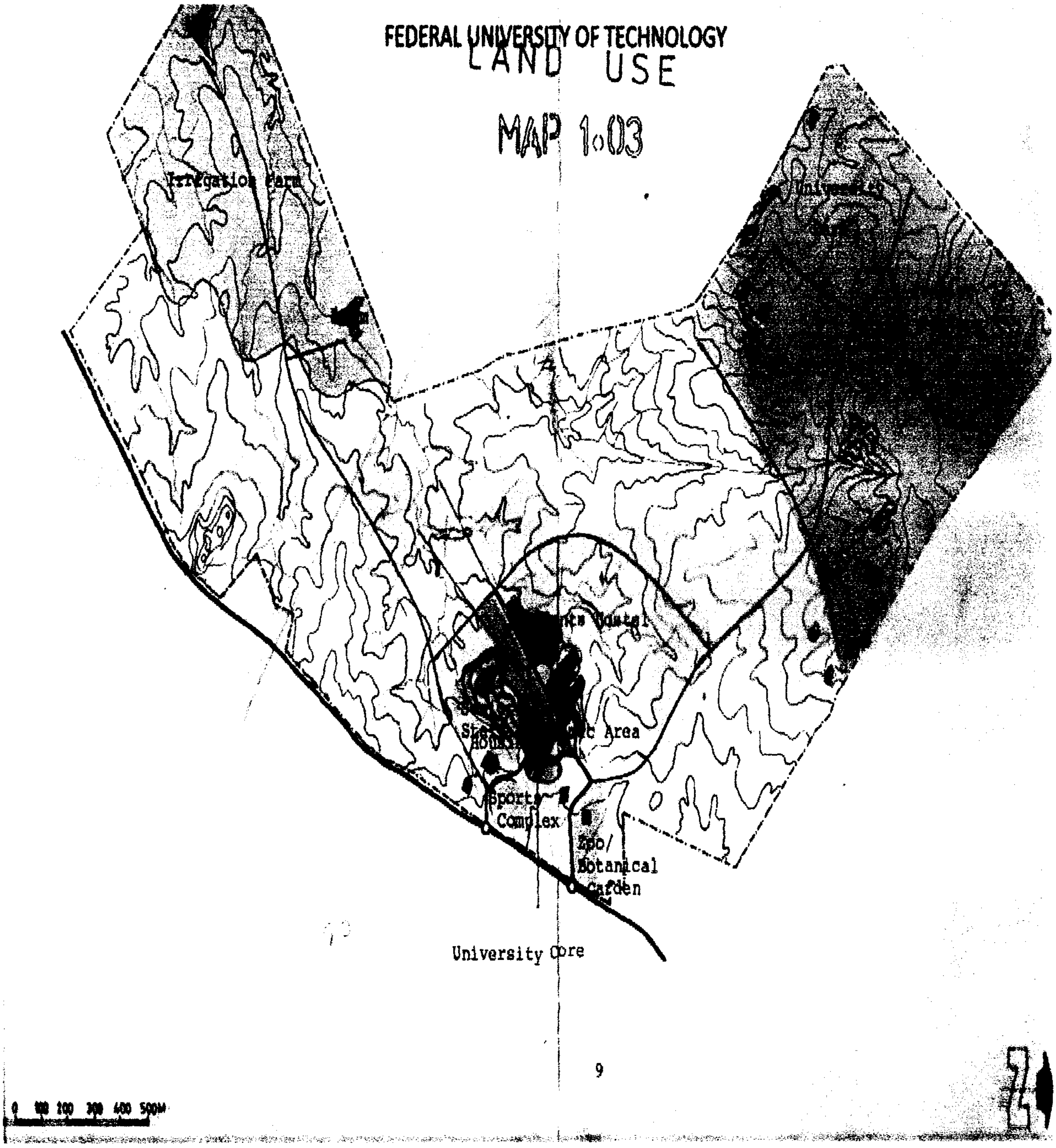


Fig. 3.1. Federal University of Technology Minna Map

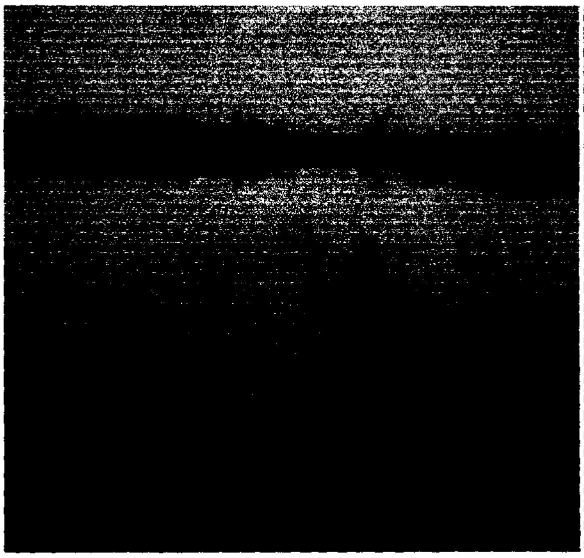


Plate I: Dan Zaria Dam

Plate II: FUT Main Campus Dam

Plate III: Sediment of FUT Main Campus Dam

3.1.1 Vegetation and Land Use

Minna falls within the semi-wood land or tree forest vegetation belt with derived dry grass or shrub land known as the southern guinea savannah. This is also known as the transition belt, which lies between the savannah grass/shrub land of the north and the rain forest of the south. Due to intensive fallow type of agricultural practice and grazing of the land, the area is dominated by stunted shrubs; interspersed with moderate height tree and perennial foliage. Similarly, due to human activities and land use abuse which is characteristic of most expanding urban centre in Nigeria, the site is fast losing its remaining tree species to development. Along some river course and lowland areas, the vegetation is more wooded and resembles some forest affinities. The area is still being used as farm and grazing land by the residents of Minna and her environs (Musa 2003).

3.1.2 Climate

3.1.2.1 Rainfall

Minna generally is known to experience rainfall from the month of May to the month of October and on rear occasions, to November. It is known to reach its peak between the months of July and August. Towards the end of the rainfall season, around October, it is known to be accompanied by great thunder storms (Musa, 2003).

3.1.2.2 Temperature

The maximum temperature period in this area is usually between the months of February, March and April which gives an average minimum temperature record of 33^0C and maximum temperature of 35^0C (Minna Airport Metrological Centre, 2000). During the rainfall periods, the temperature within the area drops to about 29^0C.

3.1.3 Soils of the Area

The major soil found in this area is the sandy loam type with a sparse distinction of the sandy – clay soil and sandy soils. This has so far encouraged the residents of Minna metropolis and neighboring villagers to use the land for agricultural activities such as farming and grazing by the nomadic farmers (Musa, 2003).

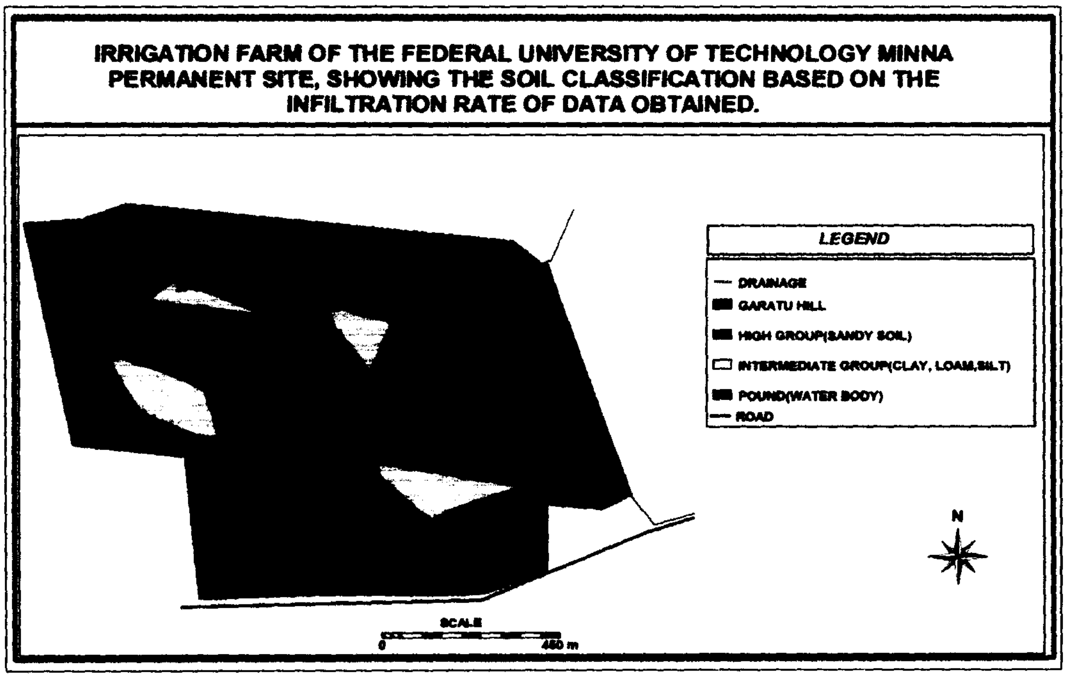
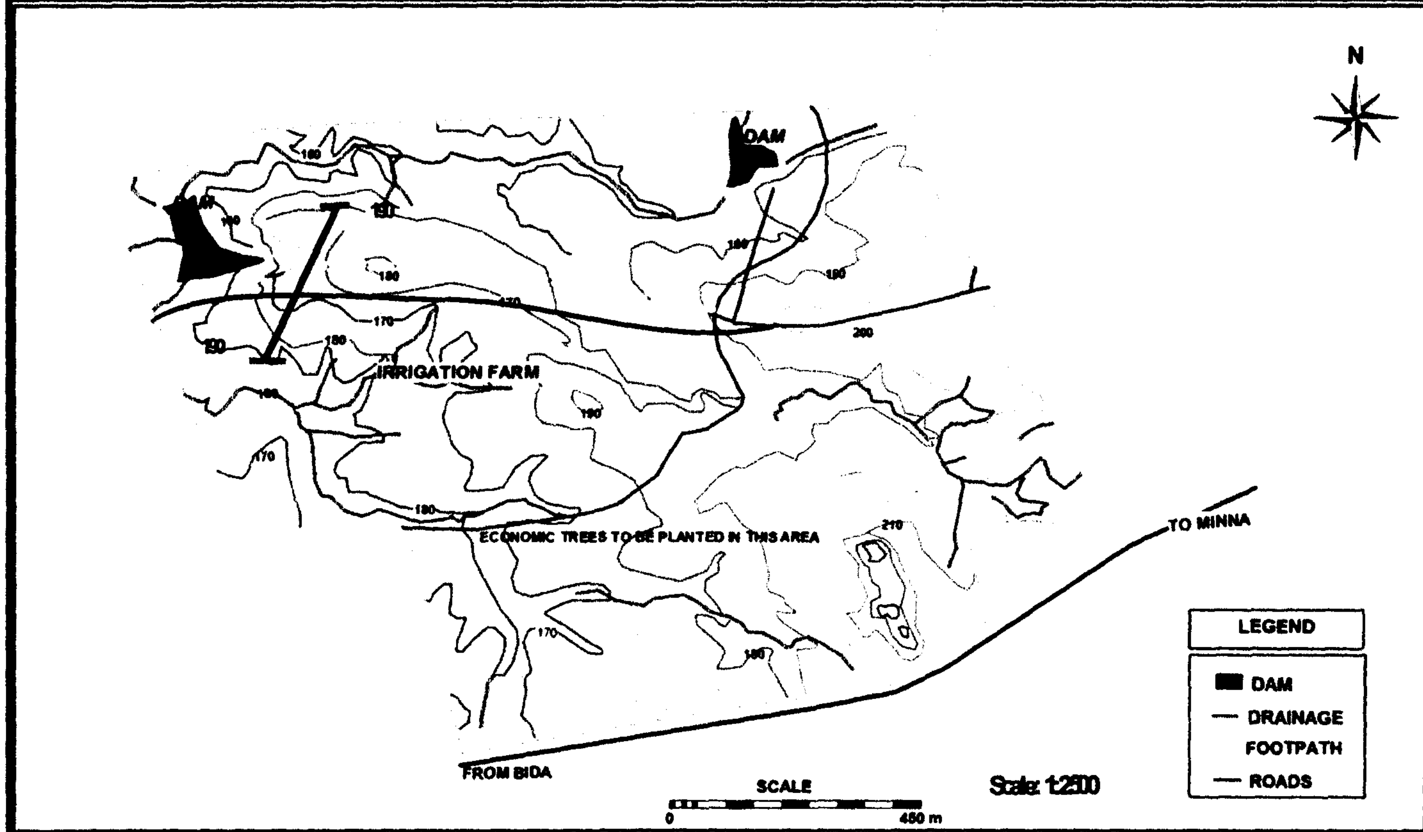


Fig: 3.2 Soil Classification Map

3.1.4 Topographical Mapping

The topographical mapping of the Gidan-Kwano campus of the Federal University of Technology, Minna with the University farm site was gotten from the planning unit of the school which as well shows the proposed dam site for the school. This shows the contour and other necessary details of the area and has a scale of 1:25000 as shown bellow. The initial proposed dam as seen in the contour map is not detailed and therefore can not be improved upon. An alternative dam axis was selected which is found to be better than those proposed, this is between contours 170 to 190, having a head of 20m.

FEDERAL UNIVERSITY OF TECHNOLOGY MINNA PERMANENT SITE FARM



3.2.Area and Volume Estimates

The area covered by the contour lines along the dam axis from the map was mapped out on a graph paper which was used in calculating the water shore line area and the catchment area of the dam. These sizes mapped out and drawn on graph have certain numbers of boxes on the graph which was counted and estimated. This is presented in chapter four under the result and discussion.

Using the elevations (difference between each contour), the volume of each contour can be obtained by simply multiplying the difference in the elevations by the Area of the contour, while the Accumulated volume is the addition of each volumes.

3.3.Laboratory Test

For an adequate and a safer long lasting dam design, the following soil test needs to be carried out on the samples obtained at the various test pits to determine the engineering characteristics and the classification of the soils.

I. In-situ moisture content test
II. Specific gravity test
III. Sieve analysis test for grain size classification Atterberge limit tests to determine liquid limit, plasticity limit and other parameters that could give fair prediction of maximum densities in compaction.
IV. Compaction test to assess the moisture dry density relationship, the optimum moisture content (O.M.C), and the maximum dry densities.
V. Undrained Triaxial test to determine cohesion, angle of internal friction and shear strength of the soils.

VI. Permeability test which is to be analyzed by the above test result.

3.4 Catchment Yield

The catchment yield, 'Y', is based on the expected annual runoff from a catchment and is an important factor in assessing the feasibility of a dam and in determining the required height of the embankment. The latter is important to allow the dam designer to size the dam to suit expected inflow and estimate the area that can be irrigated. It is estimated as follows:

1. Where the average percentage of runoff is not known, use, as a guide, a figure of 20 percent of the mean annual rainfall for the catchment area (Ahaneku, 1997). If more information is known, take the rainfall on a return period of 1 in 10 years as a guideline.
2. Calculate the annual runoff for the catchment, in mm, based on the percentage determined above. This is 'Rr'.
3. Measure the catchment area 'A' in km^2, upstream of the proposed embankment. Ignore any upstream dams (as these may already be full at the time of a flood event – often at the end of a rainy season – and thus offer no retardation of any flood moving downstream) and calculate the area of the whole catchment.
4. The annual runoff for the catchment (the catchment yield in an average year), Y, in m3, is given by:

$$Y = Rr \times A \times 1000 \qquad 3.1$$

3.5 Preliminary Volume of Earthworks

The volume of earthwork can be estimated as follows:

$$V = 0.216HL(2C + HS) \qquad 3.2$$

Where: V is the volume of earthworks in m^3.

H is the crest height (FSL+ freeboard) of the dam in m.

L is the length of the dam, at crest height H, in m (including spillway).

C is the crest width in m.

S is the combined slope value.

3.6 Freeboard

The free board is the vertical distance between the crest of the embankment and the still water surface in the reservoir. This must be sufficient to guard against overtopping by wave action, wave run-up and the wind setup or facilitation of overtopping due to effects of embankment and foundation settlement or earthquake. Normal freeboard is measured with respect to full reservoir level while minimum freeboard is measured with respect to maximum water level in the reservoir.

For small earth dams with rip-rap slopes, freeboard should be sufficient to prevent overtopping due to wave ride up equal to 1.5 times the height of the wave. Normal freeboard is based on a wind velocity of 161km/h and minimum freeboard on a velocity of 80km/h.

3.7 Side Slopes

The USBR (2008) side slopes of **3:1** for upstream face and ratio **2.5:1** for downstream face for reservoir fetch of not more than 1 mile. The fetch for this proposed dam has been seen not up to 1 mile which gave an assurance of safety in assuming these values.

For the slope protection, the recommendation under this is that a total of 1m thick rip-rap materials in differing size combinations should be used for reservoir fetch of less than 2.5miles. The practice in this part of the country, where wind effects are minimal and rain fall is light, the adoption of 300mm thick hand placed fresh rock on the upstream face and 200mm on the downstream face is justified.

3.8 Selected Dam Axis

From the contour map for the site, the dam axis selected was considered based on the contours , a high contour area was chosen so as to have a larger storage capacity. These contours selected are between 170 and 190 as shown on the contour map.

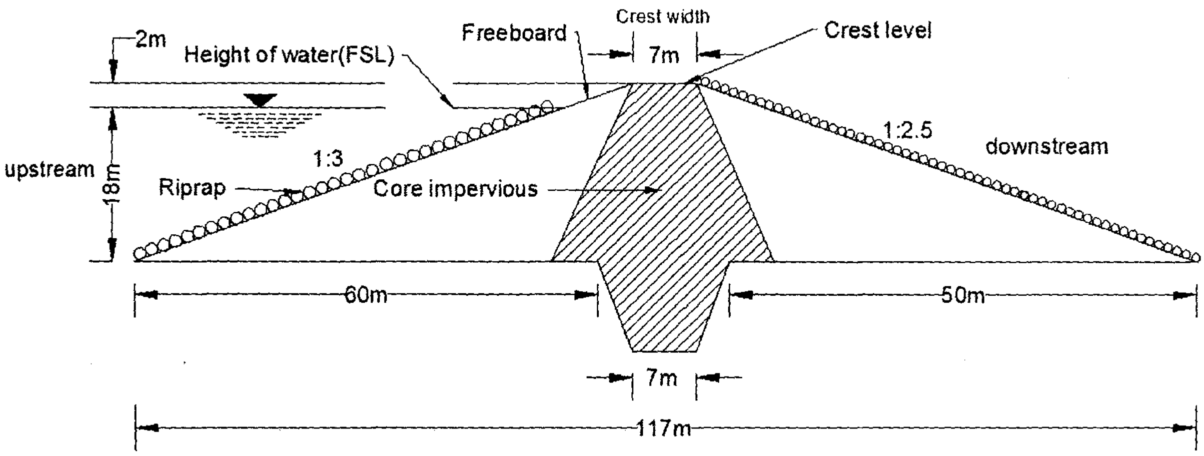


Fig: 3.10

3.9 Flood Maximum Water Level and Normal Water Level

Using the topographical map gotten from the area of the proposed dam site which is shown above, the contours of the dam axis drawn on the map is used in obtaining the maximum lood water level and the normal water level. The level which will not endanger the road and esidential houses is 190 while the bed of the river is at contour 170. The differences in this two alues is 20m which represent the height of the dam and the point of maximum flood level, while e normal water level is 2m below this point. Water is not expected to flow over the Dam walls therefore the need to have a freeboard. A freeboard is that height below the dam crest where structure is located to take excess water out of the dam. It is a function of wave height, expected out flow etc, for the purpose of this design, a value of 2m is adopted. This value is common in most dams in the central part of the country, e.g Tunga Kaw dam has a freeboard of 2m (Sani, 2006).

3.10 Peak Run-off Rate

The design capacity of the spillway is based on the peak run-off rate for a 26-year period. The 26-years period is adopted based on the relatively small size of the project and considering the apparent low hazards potential.

The Rational formula is used to compute the peak run-off for the spillway using both the manning's roughness ratio n, and the developed Manning Nigeria coefficient.

$$Q = CiA \qquad 3.3$$

Where :

Q= the design peak run-off rate in m^3/sec

C= the run-off coefficient.

i= the rainfall intensity in mm/hr for the design return period and for a duration equal to the time of concentration.

A= water shed area in ha

T= time of concentration of water shed(this is the time required for water to flow from the remotest point of the area to the outlet).

3.11 Estimates of Seepage and Water Losses

Water loss is one of the factors necessary to be considered and designed for in the design of earth dams. Losses could be due to evaporation which depend on the prevailing temperature and wind for the project area, conveyance, percolation and seepage.

3.11.1 Seepage losses

Seepage takes place through any dam material, however, by locating this seepage point and designing for it, damage by piping can be avoided. The seepage or saturated line is the gradient where there is no hydrostatic pressure. It is important to know where saturation line will egress of the downstream slope and what quality of seepage can be expected.

For the seepage estimates, the approximate method is used:

$$\boldsymbol{Q} = \frac{4kh^2}{9l} \qquad 3.4$$

Where Q = discharge in m^3/s

L = mean length of seepage path in m

K = hydraulic conductivity of soil

H = height of water

This formula was used for estimating seepage through the embankment with the results presented in chapter four of this project.

3.11.2 Evaporation Losses

The reservoir losses are determined by considering the rate of evaporation in each month and the corresponding mean water surface area in the particular month.

3.12 Reservoir Storage Capacity

The reservoir area and storage capacity was generated from the topographical map. The areas were measured using a planimeter on the scale **1:25000** shown on the contour map. The catchment area for the proposed dam in Gidan-Kwano campus of the University is taken to be $90km^2$, and mean annual rainfall is 104mm/hr.

3.13 Crest Width of Dam

The crest width of the dam is determined based on the minimum requirements for safety and functionality of structure. i.e based on the following factors:

- Nature of embankment materials and minimum allowable percolation distance through the embankment at normal water level.
- Height and importance structure
- Required width to provide embankment mass for resistance to earthquake shock and
- Road way requirement.

According to USBR recommendation, the following empirical formula could be adopted

$Bt = \left(\frac{H}{5}\right) + 3$ for low dams where H<30.00m

H is the height of the dam

Hence, crest width of the dam Bt $= \left(\frac{20}{5}\right) + 3 = 7m$

3.14 Estimates of Storage Required

It essential to estimate the economic amount of water required from the dam. This will, for irrigation dams, comprise irrigation requirement, other uses (livestock/domestic water), and losses to seepage and evaporation and the dead storage.

- Irrigation requirement can be calculated by multiplying the gross annual irrigation requirement per hectare by the area proposed. This may have to be adjusted once the estimated storage for the dam chosen is calculated.
- Environmental flows to release normal flows into the river or to comply with any legal requirements downstream.

3.15 Demand Studies

3.15.1 Agricultural demand

The predominant crops currently grown in the area includes maize, tomatoes, pepper, groundnut and Yam usually during the raining season. Since the basic reasons for the design of this dam is for irrigation purpose so that farmers in the area can grow crop even during dry season to be able to have enough food supply in the state, the water requirement of the crops were taken into consideration in sizing the reservoir capacity. Because the need for irrigation is not limited to the area of the irrigation farm shown on the topographical map alone, it is noted that it is as well needed in other areas, for this purpose; an extra 25% is therefore anticipated as the demand likely to be served by the reservoir. This gives an average total area of $300m \times 300m = 90000m^2$

3.16 Water Requirements

The water requirement of a crop means the total quantity and the way in which a crop requires water, from the time it is sown to the time it is harvested. This vary with crops as well as with the place, hence, crops have different water requirement and the same crop may have different water requirement at different places of the same country; depending upon the variation in climates, type of soils, methods of cultivation and useful rainfall etc. (Garg 2005)

The estimate composition of crops to be grown are:

80% maize

20% tomatoes and pepper

The water requirement for these crops is estimated using 1000000litres for 1ha in 2days (Maizube farm 2011). This gives: $500m^3/ha/day$

3.17 Catchment Area Estimation

Using the maps gotten from the Physical Planning Development Unit (PPDU) unit of the school, Federal university of Technology, Minna, and the drainage map of the area, the catchment size was estimated to be $90Km^2$.

The wind speed, rainfall data, evapotranspiration, temperature, and the relative humidity for the project site were obtained to represent the area in the appendix. These datas were collected for the past 26 years, analyzed and the mean intensity of rain of 104mm/hr and maximum of 143.43mm/hr were obtained.

The runoff is related to the rainfall intensity by using the Rational formula

$$Q = CIA$$

Where Q = the peak runoff rate in m^3/s

C = runoff coefficient

I = rainfall intensity in mm/hr

For the design return period and for duration to the "time of concentration" of the watershed (catchment area)

A = Area of catchment in square kilometers

The above formula is true for catchment area of 12 square kilometers maximum. A corrective factor is then applied for areas above the 12 square kilometer; this factor is derived from the work done on the drainage scheme in which could be applied for catchment areas in the sub-sahara region of Africa. (Balasha-Jalon consultant 1977).

$$Factor = 1/e^{\left(1-\frac{12}{A}\right)}$$

Where A = the area of catchment in square kilometers.

Therefore since the catchment area is over $12km^2$, this factor of safety is considered in the estimation of the Peak runoff.

Factor = 1/e(1-12/90)

= 1/e(0.867)

= 0.42

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Design of Spillway

The spillway structure is to be of the uncontrolled, trapezoidal side channel type with the crest pavement laid on the natural ground surface. The selection is based on its relatively lower coefficient of discharge which gives longer length of spillway.

4.1.1 Location of Spillway

The spillway is located on the right flank of the dam embankment, emptying through a channel into the downstream side of the main stream.

4.2 Computation of Basic Parameters

4.2.1 Time of Concentration:

The time of concentration is a function of the watershed characteristics as the maximum length of flow, size and gradient of the watershed, among other factors.

The time of concentration is estimated from the empirical formula :

$$T_c = \left(\frac{0.87l^3}{h}\right)^{0.385} \qquad 4.1$$

Where; L= maximum length of flow in m

S= watershed gradients

Tc= time of concentration in minutes

$$Tc = \left[\frac{0.87l^3}{h}\right]^{0.385}$$

$$= \left[0.87 \times \frac{700^3}{20}\right]^{0.385}$$

$$= 577.97\text{hr}$$

$$=0.16\text{sec}$$

4.3.2 Rainfall intensity, I

$$I = \frac{5.82(T^{0.16})}{(T_C+0.4)^{0.77}} \qquad 4.2$$

$$I = 5.82 \times \frac{26^{0.16}}{(0.16+0.4)^{0.77}}$$

$$= 15.14cm/hr$$

$$= 151.4mm/hr$$

4.3.3 Peak runoff rate Q

$$Q = CIA/360 \qquad 4.3$$

$$= 0.4 \times 151.4 \times 9 \times 0.420/360$$

$$= 0.636m^3/sec$$

4.4 Discharge Computation for a trapezoidal Spillway

For the trapezoidal spillway of most optimum cross section the geometric parameters have the following properties:

(i) Area of Trapezoidal Spillway $=\frac{b+2ny}{2} = y\sqrt{n^2+1}$ 4.4

(ii) Hydraulic radius $R = A/P = \frac{y}{2}$ 4.5

(iii) $\tan\theta = \tan 60 = \sqrt{3}$ 4.6

Where b = bottom width of the trapezoidal spillway (m)

y = depth of flow

n= side of slope

R = hydraulic radius

$\theta = angle\ made\ by\ the\ slope\ side\ with\ the\ horizontal$

Also Manning's coefficient N is taken to be = 0.025 for this design

Q= AV 4.7

Q= Discharge in m^3/s

A= area in m^2

V= Velocity in m/sec

Using the following data, the Velocity and Discharge of the spillway can be calculated:

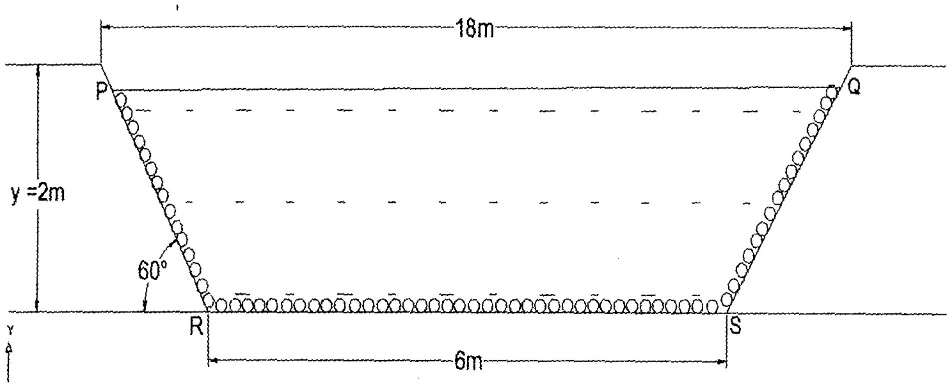


Fig: 4.1 Trapezoidal Spillway Channel

Bed width= 6m

Bed slope = 1 in 700 from dam

Depth 2.0m

Side slope of the spillway 1:3

Slant side RP = SQ

Area= $\left(\frac{Top\ width\ +bottom\ width}{2}\right) \times height$

$= \underline{(b + 2ny)} + \text{b} \times \text{y}$

2

$= y\ (b + ny)$

$$= 2(6 + \frac{1}{3} \times 2)$$

$$= 13.33m^2$$

4.8

Wetted Perimeter P = RS+RP+SQ

$$= 6 + 2y\sqrt{(1/3)^2 + 1}$$

$$P = 10.22m$$

Hydraulic Radius, R= A/p

$$= \frac{13.33}{10.22} = 1.30\text{m}$$

4.9

Velocity, $V = \frac{1}{n} R^{(2/3)} S^{(1/2)}$

$$V = \frac{1}{0.025}(1.3)^{(2/3)}(1/700)^{(1/2)}$$

$$V = \frac{1}{0.025} \times 1.191 \times 0.0378$$

$= 1.8$m/s

Q = AV

=13.33×1.8

23.99m^3/s

Spillway design using the developed Nigeria Manningcient for sandy clay soil.

$$Q = A.v$$

$$A = \frac{1}{2}.(b + 2.n.y) \text{ -}$$

$$= y.(b + n.y)$$

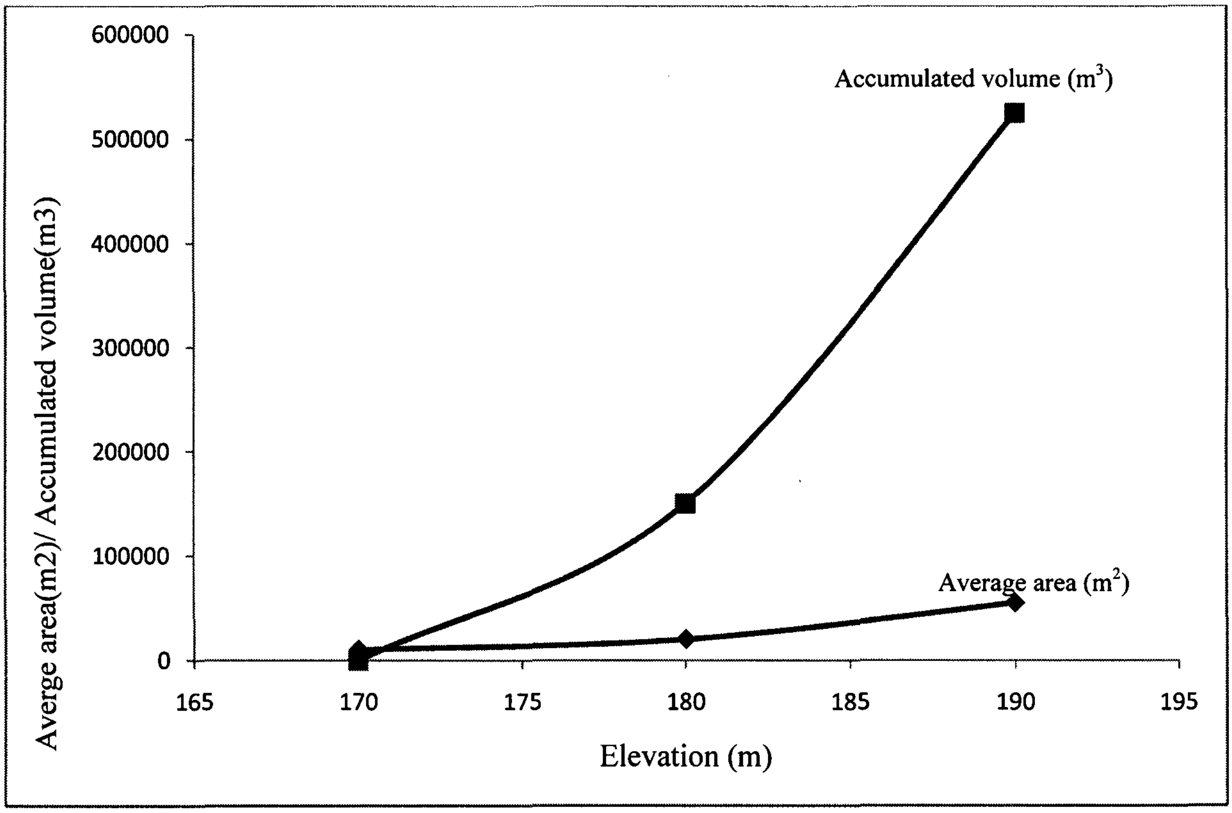


Fig 4.1 Area-capacity curve

Fig. 4.1: Area – Capacity Curve

Total storage is 525,000m^3

Water storage area at full reservoir capacity is 55,000m^2

Catchment area is 90,000m^2

Length of dam calculated from the dam axes is 700m

4.6 Catchment Yield

$Y = Rr \times A \times 1000$

$= 10.4 \times 90 \times 1000$

$= 670800 m^3$

4.7 Preliminary Volume of Earthworks

$V = 0.216HL\ (2C + HS)$

$= 0.216 \times 20 \times 700\ \{(2 \times 7) + (20 \times 5.5)\}$

$= 3024 \times 124$

$= 374{,}976 m^3$

4.8 Seepage Design

Approximate method of computing seepage through an earth embankment

Applying Darcy's law for moisture movement through soil, the seepage distance through the dam is

$$q = \frac{k(h-e)}{l} \times \frac{h+e}{2}$$

$$q = \frac{\frac{k}{2}(h^2 - e^2)}{L}$$

$$q = \frac{4kh^2}{9L} \qquad 4.10$$

Where q is the discharge in m^3/s

k is the hydraulic conductivity of the soil

L is the mean length of seepage path

e is the vertical distance from the impervious foundation to the egress of the seepage line and the vertical distance from the impervious foundation to the water level.(Assume e=h/3)

h is the height of water level

$$l = \left(1.3h + 2z - \frac{e}{2}\right)\cot \propto + w \qquad 4.11$$

Where z is the vertical distance from the head of dam to the top of dam

W is the top width of dam

α is the angle formed by the downstream slope and the foundation surface

From equation 4.11

$$\mathrm{l} = \left(1.3 \times 18 + 2 \times 2 - \frac{18}{6}\right)\cot 2.5 + 7$$

$$\mathrm{l} = 170.95\mathrm{m}$$

Using equation 4.10

$$q = \frac{4kh^2}{9l}$$

K is between 0.13- 0.50cm/hr for sandy clay soils (USDA 2008)

For this calculation, 0.25cm/hr is used

$$q = 4 \times 6.9 \times 10^{-7} \times 18^2/9 \times 170.95$$

$$= 2.76 \times 10^{-6} \times 324/1538.55$$

$$q = 5.81 \times 10^{-7} m^3/sec$$

4.9 Reservoir Dead Storage Allowance

For accurate determination of the dead storage allowance, sufficient sediment data must be available.

Since this was not available, one relay on information derived from other rivers in the region and as well on judgment. The following criteria with regard to sediment area adopted.

i. Mean concentration of suspended matter = 350PPM
ii. Dry weight of bed load 25% of that suspended matter
iii. Dry specific gravity
iv. Reservoir trap efficiency = 90%
v. Life span of reservoir

I. $350 \times 0.9 = 315ppm$
II. $0.25 \times 350 = 87.5ppm$

Total concentration of trapped sediment load

$=314 \times 87.5$

=402.5PPM

Estimated mean flow = 0.636m^3/sec = 20,056,896m^3/year

Specific gravity = 2.55

Estimated mean flow = 0.636m^3/sec = 20,056,896m^3/year

Specific gravity = 2.55

Volume of sediment in 26 years

$= 20.057 \times 26 \times \frac{1}{2.55} \times 402.5$

= 82,312.35 m^3

Volume likely to be taken over by sediments after 26 years (dead storage) is estimated to be 82312.35m^3

Active storage at elevation 190 = 525,000 – 82,312.35

= 442,687.65m^3

Therefore the Active storage or useful storage of the Dam after 26years will be 442.687.65m^3

Using the Gumbel's frequency factor formula, a 26years return period is obtained from the rainfall data.

$$K(T) = -\frac{\sqrt{6}}{\pi}(0.5772 + loglog\frac{T}{(T-1)} \qquad 4.12$$

$$K(26) = -0.7796(0.5772 + lnln\frac{26}{(26-1)}$$

$$K(26) = -0.7796(-2.6614)$$

$$K(26) = 2.075$$

The rainfall data have a mean (x)

X= 104.0mm/hr

And Standard deviation (σ) of 112.09mm/hr

$$\therefore\ x_{26}=x+k\sigma \qquad 4.13$$

=104.0 + 2.075 ×112.09

= 104.0 +232.59

= 336.59mm/hr

Hence, the calculated Rainfall intensity from the rainfall data = 336.59mm/hr. this is seen to be high when compared to the value gotten using formula, therefore the lower value was adopted for safety in calculating the peak runoff rate.

Table 4.2 Results

S/n	Parameter	Equation	Value	Manning Coefficient Value Using (0.05)	Value using Developed Manning Coefficient	
					0.01	0.15
1	Height of Dam	Difference between contour lines 190 and 170	20m			
2	Crest width	$Bt = \left(\frac{H}{5}\right) + 3$	7m			
3	Length of Dam	Estimated from dam axis using topographical map	700m			
4	Total Volume of Dam	From topographical map	525,000m^3			
5	Dam catchment Area	From topographical map	90km^2			
6	Peak Runoff rate	Q = CIA/360	0.636m^3/s			

7	Spillway discharge	$Q = AV$		$23.99m3/sec$	$59.99m3/s$	$4.00m3/s$
8	Spillway Velocity	$V = \frac{1}{n} \times R^{2/3} \times S^{1/2}$		1.8m/sec	4.5m/sec	0.30m/sec
9	Catchment yield	$Y = R_r \times A \times 1000$	$67080m^3$			
10	Volume of earth work	V = 0.216HL (204HS)	$374976m^3$			
11	Seepage through dam	$q = \frac{4kh^2}{9l}$	$5.81\times10^{-7} m^3/sec$			
12	Volume of sediment after 26years	$n\times T\times 1/S_p\times S_l$	$82312.3m^3$			
13	Active storage after 26 years		$442687.m^3$			

estimation of $5.81*10^{-7}m^3$/sec. furthermore, a model was used to estimate the volume of sediments likely to occur after 26 years with the use of rainfall data's which is $82,312.35m^{3.}$ This shows that the dam can still be active for the next 26years or more. The area-capacity curve in Figure 4.1 shows that the reservoir storage capacity and the area of dam can be determined by surveying only a few contours.

Trapezoid spillway channel was designed for the dam in this design work both the manning existing coefficient of 0.025 and the developed manning Nigerian coefficient which have a manning's value of 0.11 and maximum valve of 0.15 and this was used to validate the coefficients gotten, in order to determine which coefficient best suit the sandy clay soil in Gidan Kwano. With the use of manning's coefficient, minimum valve of manning value of manning Nigerian developed co efficient having spillway discharge of 23.99 m^3 /s, 59.99 m^3/s and 4.0 m^3/s respectively and velocities of 1.8 m/s, 4.5 m/s and 0.30 m/s respectively.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 Conclusion:

An Earth Dam was designed for Gidan Kwano using the adjacent materials and Runoff from the catchment area was estimated, A model was also fixed for the 26 years rainfall, Sediments of the area was estimated so as to know if the dam will still be active or useful after a period of about 26years. A Spillway was designed for the proposed dam at Gidan Kwano. An Area-Capacity Curve of the area was developed for future improvement. The spillway discharge of $59.99m^3/s$ approximately $60m^3/s$ is seen to be best convenient for sandy clay soil in Gidan Kwano where this design was carried out because of the larger volume of spill discharge which will help to discharge excess flood water and prevent the dam from failing, hence, elongating the life cycle of the dam.

5.2 Recommendation

The technology for Earth Dam work is a numerous one so only the Hydraulic design of the dam has been considered.

I therefore recommend that further work should dwell on the following:

i. Structural design of the embankment.

ii. Environmental impact assessment of the dam/ Auditing.

iii. The initially proposed dam should be reassessed as they were found not viable from the topographical map available as at the time of this project work.

iv. The storage capacity of the dam can be sufficient to support additional usage.

v. The Rational method which was used for this work is catchment area dependant, it is therefore recommended that flow measuring device such as V-notch weir should be installed along the reservoir.

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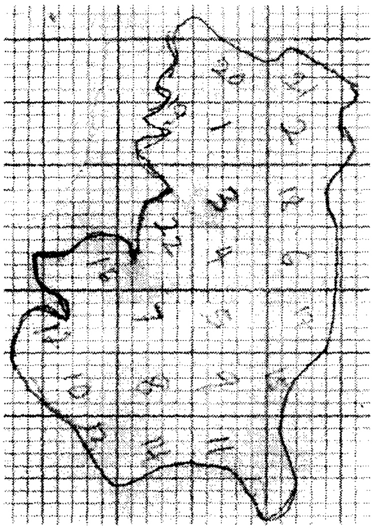
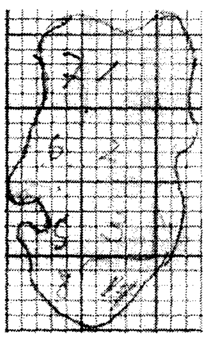
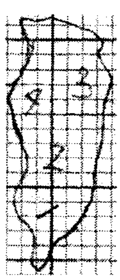
Appendix



Appendix A_1



Appendix A_2



Appendix A_3

Twenty-six years rainfall data of Minna and its environs

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	AVERAGE	STDEV
1985	0.00	0.00	0.00	30.00	127.00	132.00	262.00	205.00	331.00	24.00	0.00	0.00	92.58	0.00
1986	0.00	0.00	13.40	58.80	66.40	186.90	277.60	279.00	350.20	60.10	34.50	0.00	110.58	127.42
1987	0.00	0.00	13.50	44.60	104.50	83.00	143.70	238.50	94.60	100.10	0.00	0.00	68.54	73.92
1988	0.00	0.00	0.00	0.00	81.50	132.00	218.30	350.10	403.60	33.10	0.00	0.00	101.55	146.02
1989	0.00	0.00	5.00	49.50	287.80	193.70	193.70	248.70	202.00	79.00	0.00	0.00	104.95	111.63
1990	0.00	0.00	0.00	177.20	225.20	80.50	256.30	185.80	145.60	110.50	0.00	0.00	98.43	98.15
1991	0.00	0.00	0.00	15.00	334.80	180.00	192.20	269.70	192.00	34.10	0.00	0.00	101.48	123.89
1992	0.00	0.00	0.00	1.20	158.10	177.00	161.20	195.30	231.00	229.40	48.00	37.20	103.20	96.45
1993	0.00	0.00	0.00	0.00	173.60	171.00	189.10	269.70	177.00	62.00	0.00	0.00	86.87	101.13
1994	0.00	0.00	0.00	75.00	114.70	240.00	142.60	365.80	261.00	207.70	0.00	0.00	117.23	126.80
1995	0.00	0.00	0.00	102.00	124.00	144.00	155.00	409.20	189.00	136.40	24.00	0.00	106.97	118.93
1996	0.00	0.00	0.00	48.00	164.30	225.00	260.40	257.30	192.00	127.10	0.00	0.00	106.18	109.41
1997	0.00	0.00	41.60	63.30	191.90	190.10	308.70	271.10	473.20	180.70	0.50	0.00	143.43	152.92
1998	0.00	0.00	1.20	69.10	102.90	185.50	278.60	280.80	194.90	142.10	0.00	0.00	104.59	110.02
1999	0.00	0.00	0.00	36.50	110.50	221.10	200.50	196.10	410.70	181.80	0.00	0.00	113.10	131.187
2000	0.00	0.00	0.00	90.70	112.80	181.60	213.80	364.70	168.20	99.70	0.00	0.00	102.63	114.612
2001	0.00	0.00	0.00	3.00	136.40	162.00	207.70	310.00	303.00	151.90	0.00	0.00	106.17	122
2002	0.00	0.00	0.00	97.60	138.90	159.40	316.70	212.70	360.00	60.10	0.00	0.00	112.12	128.678
2003	0.00	0.00	0.00	58.60	118.10	145.20	208.30	351.50	251.90	187.10	0.00	0.00	110.06	120.042
2004	0.00	0.00	0.00	31.30	99.40	165.70	236.40	215.10	214.80	57.60	0.00	0.00	85.03	96.8482
2005	0.00	0.00	0.00	56.20	81.60	245.70	229.70	174.10	247.10	72.10	0.00	0.00	92.21	103.347
2006	0.00	0.00	0.00	49.10	87.00	207.00	294.20	127.80	226.40	94.80	0.00	0.00	90.53	103.407
2007	0.00	0.00	0.00	218.40	86.50	223.60	214.50	332.30	282.80	80.60	0.00	0.00	119.89	126.346
2008	0.00	0.00	0.00	28.90	128.90	153.90	305.70	373.50	297.60	53.70	0.00	0.00	111.85	139.94
2009	0.00	0.00	0.00	106.50	128.40	105.90	173.90	398.00	254.00	199.20	0.00	0.00	113.83	126.639
2010	0.00	0.00	0.00	46.30	131.50	109.80	260.90	248.30	230.80	161.10	13.00	0.00	100.14	104.56
													104.00	**112.09**

Appendix B