INFLUENCE OF ENGINEERING PROPERTIES OF THE SOIL, GEOLOGICAL AND HYDROMETEROLOGY FACTORS ON FAILURES AND DISTRESS OF EARTH DAMS IN NORTH-EASTERN NIGERIA

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CERTIFICATION

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ABSTRACT

Earth dam failures could result in the loss of lives, damage to properties, health, environmental and social problems. Distressed dams cost a lot of resources and inconveniences to remediate. There is paucity of data on failures and distresses of many earth dams located in the north-eastern part of Nigeria. This study was designed to determine the geological, hydrometeorological, engineering factors and soil properties responsible for the failures and distresses of earth dams.

A total of 42 randomly selected earth dams spread across various geologic formations and constructed with different soil materials in north-eastern Nigeria were studied. Data were obtained on failure modes, design and construction features, operation and maintenance, dam safety instrumentations and operations using the Association of State Dam Safety Officials method. Geological and hydrometeorological data related to dam failures, distresses and functionality were obtained from Upper Benue River Basin Development Authority, States Ministries of Water Resources and Nigerian Meteorological Agency. Soil samples collected were analysed for specific gravity (Gs), particle size distribution, Atterberg limits, compaction, California Bearing Ratio (CBR), permeability, triaxial compression and consolidation tests according to BS1377. The results were analysed using descriptive statistics.

The proportions (%) of failed, distressed, uncompleted and functional dams were 27, 12, 12 and 49 respectively. The failure modes were; seepage (5%), piping (8%), structural (1%), hydraulic (50%) and a combination of two or more modes in a complex manner (36%). The main causes of failure were inadequate maintenance (71%), lapses in design (9%) and poor construction (15%). On the Basement complex formations, 62, 27 and 11% of the dams were functional, failed and distressed respectively. All the dams on Gombe sandstones and Pindiga formations are functional. The status of the dams were affected by peak monthly total rainfall

(327.1–478.8mm) where 75% of the failures and distresses occurred due to high runoff, erosion, siltation and overtopping, while 20% of the failures occurred due to excessive water loss as influenced by peak monthly total evaporation ranging from 354.6-409.7mm coupled with relatively high temperatures (39–43^{o}C). Soil groups for constructing the earth dams in the study area ranged from poorly graded sands to silty/clayey sands. Seventy-nine percent of the failed and distressed dams have embankment materials with Coefficient of uniformity of less than 5. Sixty-five percent of failed and distressed dams have Plasticity Index of 0-7. Eighty percent of functional dams have highly compacted soils with maximum dry density ranging from 1.84 to 2.01Mg/m^3. High permeability ranging from 0.018 to 0.110 m/day influenced 33% of dam failures. Consolidation tests showed a settlement of 1.18mm and 2.29mm for functional and failed dam respectively. The Gs (2.41-2.70) and CBR values (11-46%) as well as cohesion (35-215kN/m^2) and angle of internal friction (3-18^{o}) influenced particular incidents without a trend.

Geologic formations, weather conditions, lapses in design, poor construction, maintenance, operation and poor soil characteristics influenced the status of earth dams in north-eastern Nigeria. Grouting, soil stabilisation, use of rock ripraps, impervious blankets and maintenance scheduling are suggested to minimise failures and ameliorate distresses.

Keywords: Earth dam, Failure modes, Soil properties, Hydrometeorology, Geologic formation

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DEDICATION

This report is dedicated to my late grandfather Modibbo Suleimanu Gombi

ACCRONYMS AND ABBREVATIONS

Symbol	Meaning
GW	Well graded gravels
GP	Poorly graded gravels
GM	Silty gravels
GC	Clayey gravels
SW	Well graded sands
SP	Poorly graded sands
SM	Silty sands
SC	Clayey sands
ML	Inorganic silts with low plasticity
CL	Inorganic clays of low plasticity
OL	Organic silts with low plasticity
MH	Inorganic silts with high plasticity
CH	Inorganic clays with high plasticity
OH	Organic clays with high plasticity
Pt	Peat and highly organic soils
I	Medium plasticity(clay)
	or compressibility(silt)
R.C.	Rolled Compacted
U/P	Upstream slope
D/S	Downstream slope
HE	Homogeneous Embankment
ZE	Zoned Embankment
C	Cohesion
Φ	Angle of Internal friction

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CHAPTER ONE

INTRODUCTION

1.1 Dams in General

A dam is a structure built across a river to create a reservoir on its upstream side for impounding water. The water stored in a reservoir is used for various purposes, such as irrigation, municipal and industrial supply, hydropower and recreation. Dams may also be constructed for flood control, retention of debris, navigation and various other purposes. A dam and a reservoir are complements of each other (Arora, 2001).

It is very difficult to say where and when the first man-made dam was built. Archeological evidences help in estimating that the very first man-made dam is at least 3000 to 5000 years old. Whenever it was built, that first dam was almost certainly an irrigation dam. Its designer might have observed beavers at work or he might have thought it in some other way. Beavers are mammals living under water belonging to the family of rats, mice, squirrels and they produce fur. These creatures create dam type barriers which amount to a place where their family can live in comfortable ponds with the help of trees which they themselves cut. These elegant structures are built out of logs, buttressed with twigs and branches and sealed with mud and stones. A beaver dam accumulates silt brought down by its stream. Whenever a beaver dam is breached, the silted water pours through, and the fertile silt is deposited over a wide area. This creates what farmers call *beaver's meadow*, where crops grow particularly well (Garg, 2008).

Most Engineers recognize seven types of dams. Three of them are ancient in origin, and four have come into general use only in about 100 years or so. The three older types of dams are: the Earth, Rock fill and Solid Masonry Gravity Dams

(1) **Earth Dams**; Earth dams are made of soil that is pounded down solidly. They are built in areas where the foundation is not strong enough to bear the weight of a concrete dam, and where earth is more easily available as a building material compared to concrete or stone or rock. Earth dams can also be further divided into two types viz; homogeneous and zoned dams. Homogeneous dams are constructed entirely or almost entirely of a single embankment material, while zoned dams are constructed of different kinds of materials at different sections of the embankment (Alam, 1978).

(2) **Rock-fill Dams**; Rock-fill are formed of loose rocks and boulders piled in the river bed. A slab of reinforced concrete is often laid across the upstream face of the rock-fill dam to make it water tight (Gopal and Rao, 2007)

(3) **Solid Masonry gravity Dams**; These types of dams are constructed using stone or solid blocks of concrete to hold back the flow of water by sheer weight. These big dams are expensive to build but are more durable than earth and rock dams. They can be constructed on any dam site, where there is a natural foundation strong enough to bear the great weight of the dam. In recent times, 4 other types of dams have come into practice. They are;

(i) Hollow masonry gravity dams; These are essentially designed on the same lines on which solid masonry gravity dams are designed. But they contain about 35 to 40% less concrete or masonry; Generally, the weight of water is carried by the deck of Reinforced Concrete Core (R.C.C) or by arches that share the weight burden. They are very difficult to build and are adopted only if very skilled labor is easily available otherwise the labor cost is too high to build this complex structure.

(ii) Timber dams; these are short lived, since in a few years time rotting sets in. Their life is not more than 30 to 40 years and must have regular maintenance during that time. However, they are valuable in agricultural areas, where a cattle raiser may need a pool for his livestock to drink from and for meeting other such low level needs.

(iii) Steel dams; these are not used for major works. Today steel dams are used for temporary cofferdams needed for the construction of permanent dams. Steel cofferdams are usually reinforced with timber or earth-fill.

(iv) Arch dams; Arch dams are very complex and complicated. They make use of the horizontal arch action in place of weight to hold back the water. They are best suited at sites where the dam must be extremely high and narrow.

1.2 Evidence of Earth Dam Failures around the World

A dam failure is an uncontrolled release of water impounded behind the dam. Dam failures may occur due to a variety of causes. The most common causes of dam failure are leakage and piping (35%), overtopping (25%), spillway erosion (14%), excessive deformation (11%), sliding (10%), gate failure (2%), faulty construction (2%), and earthquake instability (2%) (Lukman *et al*, 2011)

Dam failures and incidences have been taking place all over the world over a long period of time in history. Reports on failure of dams are common things nowadays. Effects of dam's failure on man and environment are well known, and require both preventive and mitigation measures.

Some catastrophic and devastating dam failure around the world were reported by Thandaveswara (2007) to include but not limited to the following;

(a) Kaddam Project Dam, Andhra Pradesh, India; This dam was built in Adilabad, Andhra in 1957/ 58. The dam was overtopped by 46 cm of water above the crest, in spite of a free board allowance of 2.4 m that was provided, causing a major breach of 137.2 m wide that occurred on the left bank. Two more breaches developed on the right section of the dam. The dam failed in August 1958.

(b) Teton Dam, Teton river canyon, Idaho, USA; The dam was designed as a zoned earth and gravel fill embankment, having a height above the bed rock of 126 m, and a 945 m long crest. The embankment material consisted of clayey silt, sand, and rock fragments. The dam failed on June 5, 1976, releasing 308 million m^3 of reservoir water. The time of failure was recorded as 4 hrs. The cause of failure was attributed to piping progressing at a rapid rate through the body of the embankment.

(c) Malpasset Dam; An arch dam of height 66 m, with 22 m long crest at its crown. When the collapse occurred, the dam was subjected to a record head of water, which was just about 0.3 m below the highest water level, resulting from 5 days of unprecedented rainfall. The failure occurred as the arch ruptured, and the left abutment gave way. The volume of water released was 4.94 Mm^3 while 421 lives were lost. The damage was estimated at 68 million US dollars.

(d) Baldwin Dam; This earthen dam of height 80 m, was constructed for water supply, with its main earthen embankment at northern end of the reservoir, and the five minor ones to cover low lying areas along the perimeter. The failure occurred at the northern embankment portion, adjacent to the spillway (indicated a gradual deterioration of the foundation during the life of the structure) over one of the fault zones. The V-shaped breach was 27.5 m deep and 23 m wide. The damages were estimated at 50 million US dollar.

(e) Hell Hole Dam; The Hell Hole (lower) dam was a rock fill dam of height 125 m. The dam failed during construction, when the rains filled the reservoir to an elevation of 30 m above the clay core. The capacity of this multipurpose reservoir after completion was 2.6 M m^3.

(f) Tigra Dam: (Sank, Madhya Pradesh, India, 1917; This was a hand placed masonry (in time mortar) gravity dam of 24 m height and constructed for the purpose of water supply. A depth of 0.85 m of water overtopped the dam over a length of 400 m. This was equivalent to an overflow of 850 m^3/s (estimated). Two major blocks were bodily pushed away. The failure was due to sliding. The dam was reconstructed in 1929.

(g) Machhu II (Irrigation Scheme) Dam, Gujarat, India; This dam was built near Rajkot in Gujarat, India, on River Machhu in August, 1972, as a composite structure. It consisted of a masonry spillway in river section and earthen embankments on both sides and a clay core extending through alluvium to the rocks below. The dam was meant to serve an irrigation scheme. Its, storage capacity is 1.1×10^8 m^3. The dam had a height of 22.56 m above the river bed, a 164.5 m crest length and overflow section, and a total of 3742 m of crest length for the earth dam. The dam failed on August 1, 1979, because of abnormal floods and inadequate spillway capacity. Consequent overtopping of the embankment caused a loss of 1800 lives. A maximum depth of 6.1 m of water was over the crest and within 2 hrs, the dam failed.

1.3 Evidence of Earth Dams Failures in Nigeria

There have been several cases of dam-related disasters in Nigeria displacing thousands of people, plunging them into poverty and destroying properties (Lukman *et al*, 2011). Some of the commonly documented dam failures in Nigeria are as explained in the following sections;

1.3.1 Shiroro Dam; In 1999, at least 7 local government districts in Niger state were flooded when water from the Shiroro Dam was released. Thousands of houses and buildings in the state, including schools and hospitals were either destroyed or damaged in the disaster.

1.3.2 Ojirami Dam; On 30th August 1980, the Ojirami dam failed and inundated the Akuku and Enwan communities. The failure was mainly due to technical breakdown and negligence on the part of the dam official on duty. Moreover, no alarm was installed to give warning to local officials and communities when the water exceeded its limit in the reservoir. The flood destroyed more than 180 houses in the Akuku community and many people lost their houses and other properties worth millions of naira. Although the flood did not directly cause any deaths at the time of the failure, numerous casualties were reported due to the resulting poor conditions. Residents later suffered from housing shortages. Many community members lost their local businesses due to the catastrophe and were left without a means for livelihood (Hope, 2003; Etiosa, 2006).

1.3.3 Tiga and Challawa Dams; In August 2001, over 40 people were feared dead and more than 20,000 people were displaced by the flood resulting from the failure of the Tiga and Challawa dams in Niger and Jigawa States of Nigeria.

1.3.4 Shiroro Dam; Over 26 villages in Kede, Lakpma, and Shiroro Local Governments in Niger State were flooded by the waters from Rivers Niger and Kaduna in 2003. The flood displaced about 10,000 persons in Ketsho and in Kede Local Government who were said to have moved to Kwara State, while other 13,500 person in Lakpam and Shiroro were rendered homeless. In the affected areas, houses, property, farm produce, and animals were destroyed by the flood which struck in the early hours of September 11, 2003. The flood resulted from a downpour and the release of excess water from the Shiroro Hydro-Electric Dam by the then National Electric Power Authority (NEPA). The affected villages include Galadima Kogo, Gofa, Kusasun, Pai, Lagado, Nakpinda, and Karai (Etiosa, 2006).

1.3.5 Obudu Dam; The Obudu Dam spillway was damaged by storm in July 2003 and resulted in fatal disaster that claimed over 200 houses, several farmlands, settlements, and business concerns. The disaster was allegedly caused by the release of excess water from the Lagdo Dam in Cameroun, which overflowed Benue and Niger River banks. Besides the release of excess water from Lagdo Dam, experts attributed the disaster to intensive and non-stop rainfall in Obudu on the fateful day for 16 hrs. The rainfall recorded at the Obudu dam meteorological station was 314.5 mm. This is more than 15 years average rainfall for the peak months of July and September and was not anticipated for when the dam was constructed. The cumulative effect of these events, led to the overflow of all water courses (Etiosa, 2006).

1.3.6 Igabi Dam; Properties worth about N500 million were destroyed while thousands of people were rendered homeless in Kaduna State when River Kaduna overflowed its banks and submerged several streets and housing estates. The flood was caused by the collapse of Igabi Dam. Affected by the flood are Mamman Kotangora Estate, Kirgo Road extension, Kabala area, and parts of Malali Estate. At the Mamman Kotangora Estate, household items including rugs, television sets, fridges, chairs, tables, and other expensive electronics were damaged when water from the river submerged most of the houses there. Several mechanic workshops, grocery stores, and pharmaceutical shops were also submerged. At Kirgo area, apart from household items, maize and sugarcane farms were also destroyed. It was learnt that a manual irrigation system constructed by some farmers in the area made it possible for the river water to submerge places like Mamman Kotangora Estate and Kabala area. Apart from churches and mosques which were destroyed, the Nsukka town hall located at Kirgo Road extension was also affected (Etiosa, 2006).

1.3.7 Alau Dam Maiduguri; The Alau dam was constructed with high hopes: to supply potable water to Maiduguri metropolis and to irrigate the Jere bowl for the production of rice. Several years after its construction, the reservoir behind it failed to fill up to an expected level. The treatment plant has not been completed because there is no sufficient water to run it. No water has been released to flood the jere bowl, and the people of the metropolis could only comment on the dams adverse effects. The reservoir losses water to the unconsolidated sands at its floor, and to the Bama ridge, the outermost beach of the Mega-Chad, on its northeastern margin. There is a topographic divide between the reservoir and the Alau system which has not allowed river Alau to contribute any water to the reservoir. Another issue is the siltation of the reservoir bed owing to increased soil erosion within the basin as a result of increased human activities. The reservoir basin is not adequately confined and a small rise would flood extensive areas in the region (the restraining dykes notwithstanding) thereby compounding the problems of seepage and evaporation both of which are considerable in the area (Olofin, 1985).

On September 13, 1994 due to the flood in Maiduguri, the Alau dam gave way (Odihi, 1996). One of the restraining dykes and the spillway failed and water rushed out of the reservoir in one devastating flood affecting places such as Bulumkutu, Gomari, Gamboru, London Ciki, Bulabulin Gwange, and Gamboru downstream of the dam. The torrential rain was unprecedented and never expected by the planners and designers of the dam.

1.4 Statement of Problem

Earth dam's failure can be catastrophic; involving lives and properties. Earth dams are more susceptible to failure as compared to any other type of dam. There is paucity of data on earth dam's failures and distresses in the study area.

Before the development of the discipline of soil mechanics, earth dams were being designed and constructed on the basis of experience, as no rational basis for their design was available. This probably led to the failure of various such earth embankments. Gravity dams and arch dams require sound rock foundations, but earth dams can be easily constructed on earth foundations. However, earth dams are more susceptible to failures as compared to rigid gravity dams or arch dams. However, in these days, these dams can be designed with a fair degree of theoretical accuracy, provided the properties of the soil placed in the dam are properly controlled. This condition makes the design and construction of such dams thoroughly interdependent. (Garg, 2008)

Earth dams are susceptible to failures as a result of many reasons including; poor feasibility studies, hydrology, geology, design error, construction problems, soil materials, some unforeseen circumstances etc. This study attempts to investigate the reasons for the failures and distresses of earth dams in north-eastern part of Nigeria and come up with engineering solutions to minimize failures and remedy distresses.

1.5 Aim and Objectives

The main aim and objectives of the study are;

(i) to investigate the reasons that led to failures and distresses of earth dams in North-eastern, Nigeria.

(ii) to develop a data base for earth dam failures and distresses.

(iv) to suggest engineering solutions to minimize failures and remedy distresses of earth dams.

1.6 Justification

To keep pace with the ever-increasing demands of water for irrigation, domestic water supply and hydro power generation, more and more earth dams, in preference to other types of dams, are expected to be constructed in Nigeria in times to come. This is due to;

(i) Construction materials like cement and steel which are required in huge quantities for building concrete dams are getting costlier and unaffordable by the day.

(ii) Earth dams are made of locally available materials, like clay, gravel, sand, silt, and boulders.

(iii) Earth dams are cheaper and can be easily constructed.

The two basic requirements to be satisfied by an earth dam are imperviousness and stability under all conditions of operations. Despite these advantages of materials and cost, earth dams are more susceptible to failures as compared to rigid gravity dams or any other type of dam.

1.7 Expected Contributions to Knowledge

A lot of work has been done by many researchers in the area of earth dam failures and the application of soil mechanics in the use of soils to construct earth dams. However, there seem to be little study on earth dam failures and distresses especially in North-eastern Nigeria.

A data base will be generated that can document failures and distresses of earth dams in the study area to aid research and development in the field of earth dam design and construction.

A focus on the engineering properties of soils as they influence failure and distress of earth dams is better than general assessment of the subject.

The study will be of much value to scholars, consultants, governments and international agencies interested in earth dams design and construction especially in Nigeria.

The study will specifically be of immense importance to Agricultural, Civil and Geotechnical Engineers with respect to their various interests in soils, its behavior in the study area as well as the application of such to minimize failures and ameliorate distresses of earth dams.

Safer and more stable earth dams are expected to be designed and constructed with some degree of accuracy using the findings of the study.

CHAPTER TWO

REVIEW OF LITERATURE

2.1 History of Dams

Any discussion on history of dam building will be incomplete without a mention of beavers, the furry animals belonging to the rodents family that build astonishing water impounding structures across streams and rivulets using tree branches, chopped wood, twigs and earth. Beavers are also known for building canals, and their unique homes called lodges. Earliest dam builders might have been inspired by beavers to some extent. Historical evidence of dam building traced to periods as early as 5000 B.C.(Admin, 2009)

Early dam building took place in Mesopotamia and the Middle East. The earliest recorded dam is believed to have been on the Nile River at Kosheish, where a 15 m high masonry structure was built about 2900 BC to supply water to King Menes' capital at Memphis. The earliest known dam is the Jawa Dam in Jordan, 100 km northeast of the capital Amman. This gravity dam featured a 4.5 m high and 1 m wide stone wall, supported by a 50 m wide earth rampart. The structure is dated to 3000 BC. The Ancient Egyptian Sadd-el-Kafara Dam at Wadi Al-Garawi, located about 25 km south of Cairo, was 102 m long at its base and 87 m wide. The structure was built around 2800 or 2600 B.C. as a diversion dam for flood control, but was destroyed by heavy rain during construction or shortly afterwards (Wikipedia, 2012).

Roman dam construction was characterized by "the Romans' ability to plan and organize engineering construction on a grand scale". Roman planners introduced the then novel concept of large reservoir dams which could secure a permanent water supply for urban settlements also over the dry season. Their pioneering use of water-proof hydraulic mortar and particularly Roman concrete allowed for much larger dam structures than previously built, such as the Lake

Homs Dam, possibly the largest water barrier to date, and the Harbaqa Dam, both in Roman Syria. The highest Roman dam was the Subiaco Dam near Rome; its record height of 50 m remained unsurpassed until its accidental destruction in 1305 (Wikipedia, 2012).

Roman engineers made routine use of ancient standard designs like embankment dams and masonry gravity dams. Apart from that, they displayed a high degree of inventiveness, introducing most of the other basic dam designs which had been unknown until then. These include arch-gravity dams, buttress dams and multiple arch buttress dams all of which were known and employed by the 2nd century AD.

The Kallanai is a massive dam of unhewn stone, over 300 m long, 4.5 m high and 20 m wide, across the main stream of the Kayeri river in Tamil Nadu, South India. The basic structure dates to the 1st century AD. and is considered one 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 waters of the Kaveri across the fertile Delta region for irrigation via canals. It is considered to be the oldest dam still in use.

Du Jiang Yau is the oldest surviving irrigation system in China that included a dam that directed waterflow. It was finished in 251 B.C. A large earthen dam, made by the Prime Minister of Chu (state) of, Sunshu Ao, flooded a valley in modern-day northern Anhui province that created an enormous irrigation reservoir (62 miles in circumference), that is still present today.

In Iran, bridge dams such as the Band-e Kaisar were used to provide hydropower through water wheels, which often powered water-raising mechanisms. One of the first was the Roman-built dam bridge in Dezful which could raise 50 cubits of water for the water supply to all houses in the town. Also diversion dams were known. Milling dams were introduced which the Muslim engineer called the *Pul-i-Bulaiti*. The first was built at Shustar on the River Karun, Iran, and many of these were later built in other parts of the Islamic world. Water was conducted from the back of the dam through a large pipe to drive a water wheel and watermill in the 10th century (Jackson, 2008).

In the Nethrlands, 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. Such dams often marked the beginning of a town or city because it was easy to cross the river at such a place, and often gave rise to the respective place's names in Dutch. For instance the Dutch capital Amsterdam (old name Amstelredam) started with a *dam* through the river Amstel in the late 12th century, and Rotterdam started with a *dam* through the river Rotte, a minor tributary of the Nieuwe Maas. The central square of Amsterdam, covering the original place of the 800 year old dam, still carries the name Dam Square or simply *the Dam* (Jackson, 2008).

The age of hydropower and large dams emerged following the development of the turbine. French engineer Benoit Fourneyron perfected the first water turbine in 1832. The era of mega-dam building was initiated after Hoover Dam was completed on the Colorado River in 1936. By 1997, there were an estimated 800,000 dams worldwide, some 40,000 of them over 15 m high (Nicholas, 1998). As at 2010, the tallest dam in the world is Nurek Dam in Tajikistan. Completed in 1980, it reaches 300 m height.

2.2 Classification of Dams

Dams can be classified in various ways depending on the purpose of the classification as follows:.

(1) **Classification According to Material used for Dam Construction**: The dams classified according to materials used for construction are; Solid masonry gravity dams, Earthen

dams, Rock-fill dams, Hollow masonry gravity dams, Timber dams, Steel dams, and R.C.C Arch dams (Brown, 1984).

(2) **Classification According to Use**;

(i) Storage Dams; They are constructed in order to store water during the period of surplus water supply and to be used later during the period of deficient supply. The stored water may be used in different seasons and for different purposes. They may be further classified depending upon the specific use of the water, such as for navigation, recreation, water supply, irrigation, fish, electricity, etc.

(ii) Diversion Dams; these small dams are used to raise the river water level, in order to feed an off taking canal and or some other conveyance systems. They are very useful as irrigation development works. A diversion dam is generally called a weir or a barrage (Venkatramaiah, 2006).

(ii) Detention Dams; They detain flood waters temporarily so as to retard flood runoff and thus minimize the bad effects of sudden flooding. Detention dams are sometimes constructed to trap sediment. They are often called debris dams.

(iii) Coffer Dam; A coffer dam is not actually a dam. It is rather an enclosure constructed around the construction site to exclude water so that the construction can be done in dry conditions.

(3) **Classification According to Hydraulic Designs:**

(i) **Overflow dams**; They are designed to pass the surplus water over their crest. They are often called spillways. They should be made of materials that cannot be eroded by such discharges.

(ii) **Non-overflow dams**; They are those which are designed not to be overtopped. This type of design gives us a wider choice of materials including earth fill and rock-fill dams. Many a times, the overflow dams and the non-overflow dams are combined together to form a composite single structure.

(iii) **Rigid dams and non rigid dams**; Rigid dams are those constructed of rigid materials like masonry, concrete, steel, timber etc while non rigid dams are constructed of earth and rock-fill.

(4) **Classification according to Hazard of Failure**; According to USDA and NRSC Conservation Engineering Division, 3 classes are recognised;

(a) **Low Hazard Class**- Dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.

(b) **Significant Hazard Class**- Dams located in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads, or cause interruption of use or service of relatively important public utilities.

(c) **High Hazard Class**- Dams located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways or railroads (Anonymous, 2005).

(5) **Classification based on Height, Volume of Earth work and Reservoir Capacity**;

(a) **Small dam**; a dam with a height not exceeding 15m above the deepest bed level.

(b) **Large dam**; a dam with a height exceeding 15m above the deepest bed level. Dams between 10m and 15m height may be treated as large dams, provided the volume of earth work exceeds 0.75million m^3 or volume of storage exceed 1million m^3, or the maximum flood discharge exceeds $2000m^3/s$ (Anonymous,2004).

(6) **Classification according to organizations**;

(I) The International Commission on Large Dams (ICOLD) definition of large dams;

(a) All dams with heights of 15m or more measured from the lowest portion of the general foundations area to the crest.

(b) Dams between 10m and 15m can be included if desired provided they comply with at least one of the following conditions, the:

1. length of crest not less than 500m
2. capacity of the reservoir formed by the dam not less than 1million m^3.
3. maximum flood discharge dealt with by spillway not less than 2000 m^3/s
4. dam has special foundation problems
5. dam is of unusual design.

(II) The National Subcommittee on Dams(NSCD)/ Nigerian Committee on Large Dams (NICOLD) definition of medium dams;

(a) All dams with heights between 8 and 10m measured from the lowest portion of the general foundation area to the crest or

(b) Any dam which does not meet the criteria for small or large dam

(III) The NSCD/NICOLD definition for small dams;

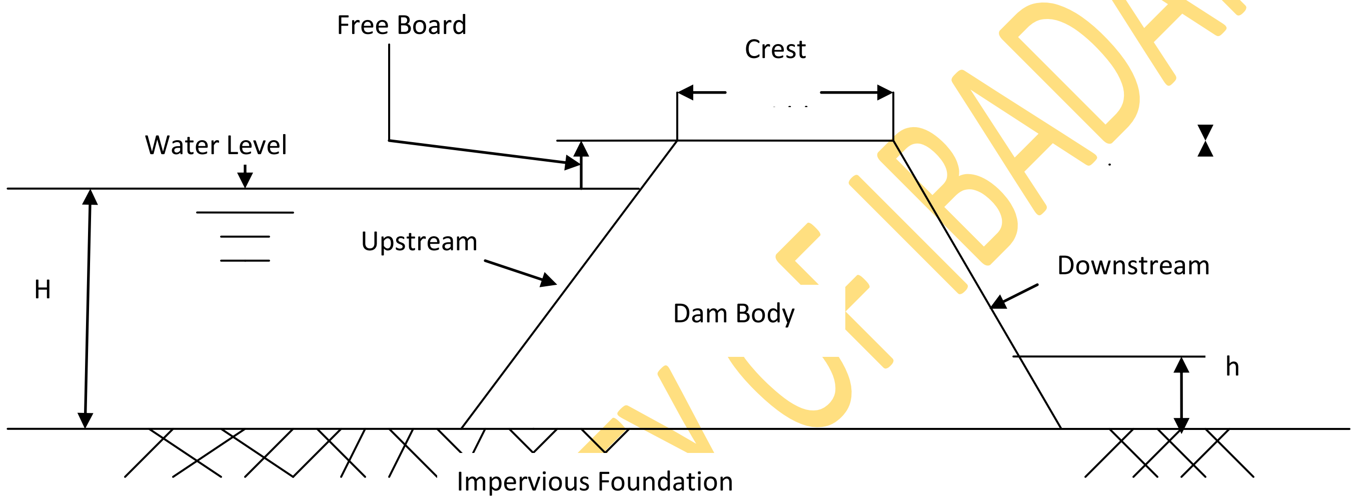
All dams not more than 8m in height measured from the lowest portion of the general foundation area to the crest and impounding not more than 1million $m^3$of water (Anonymous, 2004).

2.3 Earth Dams

Earth dams are constructed mainly from earth or soil. Earth dams for the storage of water for irrigation has been built since early times. Early earth dams were of low heights, as these were designed by empirical methods and their construction was based on experience. Developments in soil mechanics and new construction techniques have been helpful in creating confidence among engineers to build dams of very large heights and configurations.

In terms of composition of materials an earthen dam may be homogeneous or zoned type. A purely homogeneous type of dam is composed of single kind of material (Figure 2.1). The purely homogeneous type of section has now been replaced by a modified homogeneous section in which small amount of carefully placed pervious material control the action of seepage so as to permit much steeper slopes as compared to pure homogeneous dam. A pure homogeneous earth dam can be modified by putting a central impervious core to increase the water tightness and permit steeper slopes as depicted by Figure 2.2. When the foundation is pervious to some extent a cut-off is provided to minimize seepage (Figure 2.3)

The zoned earth dams is composed of a central core flanked by zones of materials considerably more pervious called shells. (Agarwal, 2000). The zoned earth dam is composed of more than one soil type. It usually consists of a central impervious core flanked by shells of pervious materials on the upstream and downstream sides (Figure 2. 4). A transition filter is usually required between the core and the shells to prevent piping and increase stability. The central core checks seepage through the dam. It may be constructed with clay, silt, silty clay or clayey silt. The pervious shells give stability to the dam and may consist of sand, gravel or a mixture of these materials. The upstream pervious zone provides free drainage during sudden drawdown. The downstream pervious zone acts as a drain to control the phreatic line. The pervious zones also give stability to the core and distribute the load over a large area of the foundation. The transition filters prevent the migration of the core materials into the pores of the shell materials.



Figue.2.1 Pure Homogeneous Earth Dam on Impervious Foundation

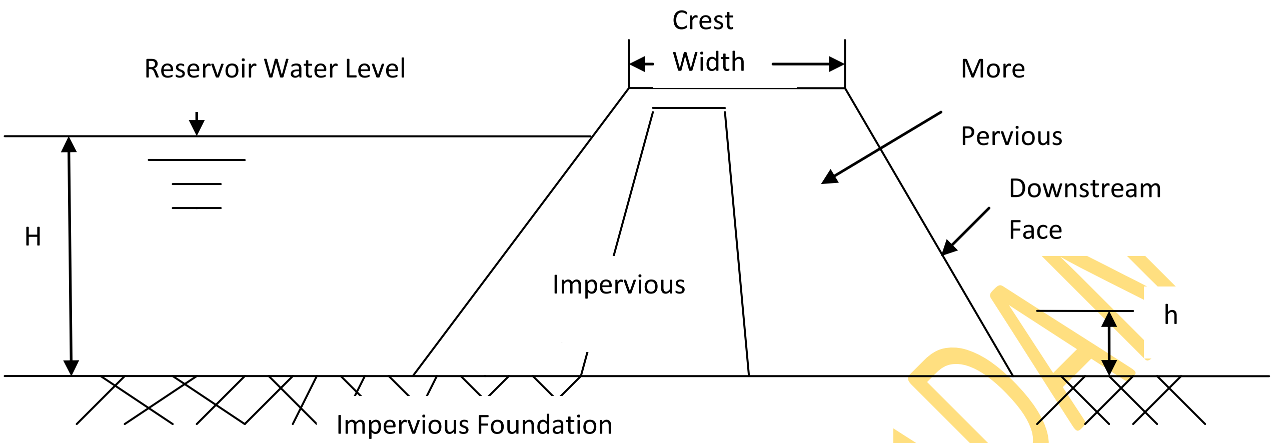


Figure.2. 2 Modified Homogeneous Earth Dam on Impervious foundation

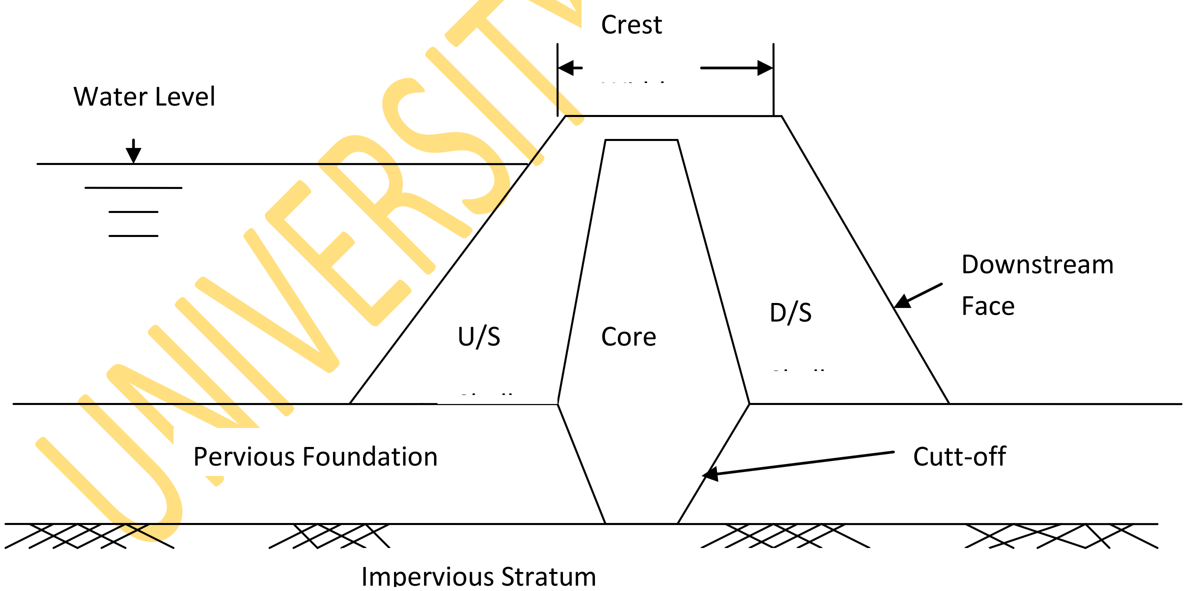


Figure.2. 3 Modified Homogeneous Earth Dam on Pervious Foundation with a Cutt-off

The downstream transition filter is useful during the steady seepage conditions and the upstream filter is useful during the sudden drawdown conditions. However, the transition filters may be omitted if the particle sizes of the core material and the shell material do not differ much or when the seepage gradient line through the dam is quite flat.

If only one type of material is available nearby, a homogeneous section is generally preferred (Figure 2.1) for economic reasons. If the material available is impervious or semi pervious, a small quantity of pervious material is required as casing for protection against cracking. On the other hand, if it is pervious, a thin impervious membrane is required to form a water barrier (Agarwal, 2000)

The various components of the zoned earth dams and their functions are as summarized;

(i) **Shell**: The shell consists of pervious materials. The main function of the shell is to provide structural support to the core and to distribute the loads acting on the dam over a large area on the foundation. It provides stability to the dam.

(ii) **Core**: The core is built of impervious materials. The main purpose of the core is to reduce seepage through the body of the dam.

(iii) **Cut off trench:** When the foundation is pervious to a moderate depth, a cut off trench is provided in the foundation to the impervious stratum. Generally the core is extended down to form a cut off to control seepage.

(iv) **Transition zone**: The transition zones (or transition filters) are provided between the core and the shell when the difference of their particle sizes is quite large. The transition zone prevents the migration of the core material into the pores of the shell material. It is usually built of semi-impervious(SM) materials. However, when the difference in the particle sizes of the core and

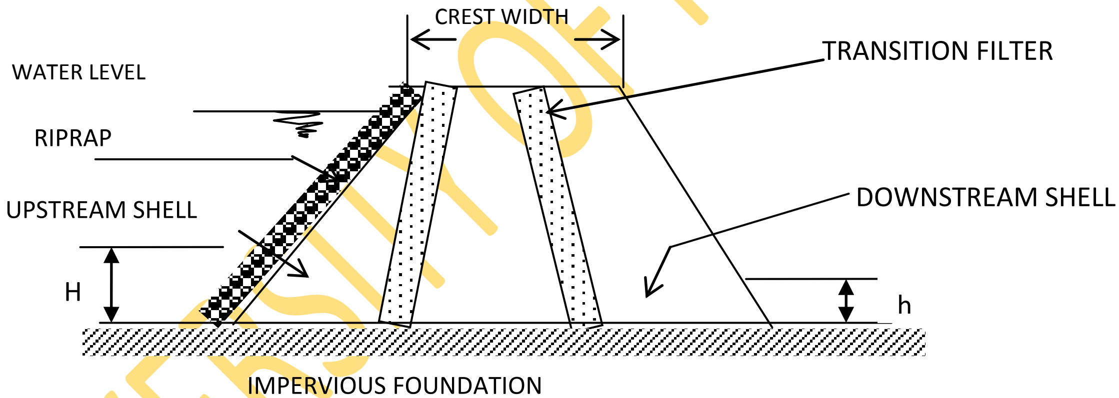


Figure .2. 4 Zoned Earth Dam

shell is not large, the transition zone is omitted. Sometimes, compacted fill is used in the transition zone.

(v) **Drainage system:** An internal horizontal drainage system is provided to carry away the water that seeps through the core or the cut off trench. It also prevents the saturation of the upper part of the downstream shell by rain or water spray. The rock toe is also provided along with the horizontal drainage system. The drainage system prevents sloughing of the downstream face due to seepage or the rain water.

Both the internal drainage and the rock toe require protective graded filters to prevent migration of the soil particles and piping. Due to the provision of the graded filters, the seepage water does not carry the soil particles into the drainage system and clog it or develop seepage erosion.

(vi) **Rip rap:** Riprap is required on the upstream face of the dam to prevent erosion by waves. It generally extends from a level just below the minimum water level to just above the maximum water level or up to crest level. Rip rap is also provided on the downstream face up to the maximum tail water level.

(vii) **Sod or turfing sod:** Sod (or turfing) is provided on the downstream face of the dam above the tail water level to prevent erosion due to rain and wind. In some cases, thin riprap layer is used instead of sod for the same purpose.

(viii) **Surface drainage**: For surface drainage of downstream slope , a system of open paved drains(chutes) along the sloping surface terminating in the longitudinal collecting drains at the junction of berm may be provided to drain the rain water. The section of drain may be trapezoidal having depth of 30cm. collecting drain , the rain water is carried through 15cm diameter pipes placed

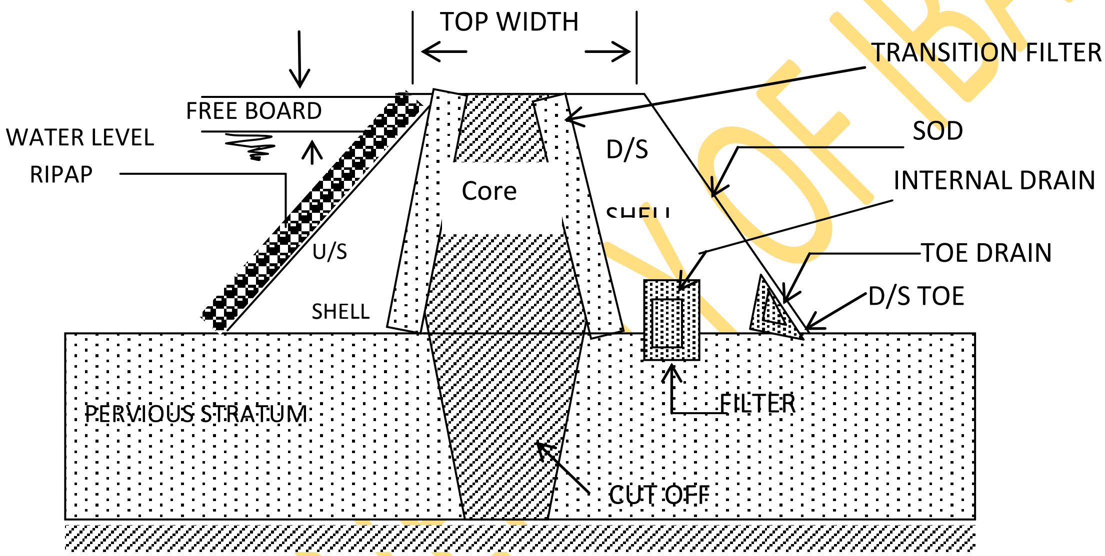


Figure 2.5 Zoning of an Earth Dam

into paved chutes on the downstream slope. Where no berm has been provided, the open paved drains (chutes) should terminate in the downstream rock toe or toe drain.

(ix) **Impervious blanket**: The horizontal impervious blanket is provided to increase the path of seepage when full cut-off is not practicable in pervious foundation. The impervious blanket should be connected to the core of the dam. To avoid formation of cracks the material should not be highly plastic. Reference may be made to IS 12169 – 1987 and IS 1498 – 1970 for suitability of soils for blanket. As a general guideline, impervious blanket with a minimum thickness of 1m and a minimum length of 5 times the maximum water head measured from upstream toe of core may be provided .(Agarwal, 2000).

Recent developments in earth dams construction led to an interesting type of temporary earth dam occasionally used in high altitudes of the cooler regions of the world known as the frozen core dam, in which a coolant is circulated through pipes inside the dam to maintain a water tight region of permafrost within it (Wikipedia encyclopedia, 2007)

2.4 Dam Failures in General

According to ICOLD (1986) a failure is defined as: *"collapse or movement of part of the dam or its foundations, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people and/or property downstream".* To the term "*incident*" is assigned the task of covering all the troubles occurred to a dam, but not degraded in "*failure*", due to the timely recourse to remedial measures. The term "*accident*", even if not officially codified, is used to represent the anomalies of the behavior of the structure that could have been evolved to "*incidents*" or also to "*failures*", but whose timely diagnosis avoids any further negative progress.

The magnitude of recorded damages to earth dams ranges from complete catastrophic failure, resulting in large property damage and loss of life, to relatively minor deterioration which may or may not necessitate remedial work. The worst type of complete failure occurs when the reservoir water suddenly breaks through the embankment and surges downstream in one devastating flood wave. Lesser damages may in the long run lead to complete failure if left unattended and some of which require only maintenance work even under most extreme conditions.

Advancement in the science of soil mechanics has given the engineer powerful analytical tools and rational procedures which have made obsolete many of the older ‘’rules of thumb’’ formally used for earth dams. Knowledge of the principal lessons learned from failures and damages in the past is an essential part of the training of the earth dam designer.

It might be expected that progressive advances in dam design and construction techniques would result in lower incidence of failures. This, however, does not appear to be the case, for two main reasons. First, with any technological advance there are always likely to be unforeseen factors that can produce unexpected problems. Second, most of the easy dam sites around the world have been utilized. This means that future dam construction will be necessary at progressively more difficult and geologically complex dam sites, which increases the probability of dam failure accidents (Wrechein and Mambretti, 2009). Despite the increasing safety of dams due to improved engineering knowledge and better construction quality, a full non-risk guarantee is not possible and an accident can occur, triggered by natural hazards, human actions or just because the dam is loosing strength capacity due to its age.

On a worldwide scale, it is clear that the objective of constructing stable dams is not always achieved. During the 1900–1965 periods, about 1% of the 9000 large dams in service throughout the world have failed, and another 2% have suffered serious accidents (Wrechein and Mambretti, 2009).

The Banqiao dam and Shimantan Reservoir Dam are among the 62 dams in Zhumadian Prefecture of China's Henan Province that failed catastrophically in 1975 during Typhoon Nina.The dam failures killed an estimated 171,000 people; 11 million people lost their homes. It also caused the sudden loss of 18 GW of power , the equivalent of roughly 9 very large modern coal-fired power stations or about 20 nuclear reactors, equalling about 1/3 the peak demand on the UK National Grid.(Wikipedia, 2012).

In August 1975, however, a once-in-2000-years flood occurred, produced by the collision of Super Typhoon Nina and a cold front. More than a year's rainfall fell in 24 hrs, which weather forecasts failed to predict because the typhoon disappeared from radar. Communications to the dam was largely lost due to the collapse of buildings under heavy rain and wire failures. On August 6 of the same year, a request to open the dam was rejected, because of the existing flood in downstream areas. On August 7, however, the request was accepted, but the telegrams failed to reach the dam.

The sluice gates were not able to handle the overflow of water, partially due to sedimentation blockage. On August 7 at 21:30, the People's Libration Army was deployed on the Banqiao Dam, sent the first dam failure warning via telegraph. On August 8, 0:30, the smaller Shimantan Dam, designed to survive a 1-in-500-year flood, failed to handle more than twice its capacity and broke upstream, only 10 minutes after Unit 34450 sent a request that would open the Banqiao Dam by air strike. A half hour later, at 1:00, water at the Banqiao crested at the

117.94 m above sea level, or 0.3 meter higher than the wave protection wall on the dam, and it too failed. This precipitated the failure of 62 dams in total. The runoff of Banqiao Dam was 13,000 m^3/s in against 78,800 m^3/s out, and 701 million m^3 of water were released in 6 hrs, while 1.67 billion m^3 of water were released in 5.5 hrs at upriver Shimantan Dam, and 15.738 billion m^3 of water were released in total. The resulting flood waters caused a large wave, 10 km wide and 3–7 m high in Suiping, to rush onto the plains below at nearly 50 km/hr, almost wiping out an area 55 km long and 15 km wide, and creating temporary lakes as large as 12,000 km^2. Seven county seats were inundated, as were thousands of square kilometers of countryside and countless communities. Evacuation orders had not been fully delivered because of weather conditions and poor communications. Telegraphs failed, signal flares fired by Unit 34450 were misunderstood, telephones were rare, and some messengers were caught by the flood. While only 827 out of 6,000 people died in the evacuated community of Shahedian just below Banqiao Dam, half of a total of 36,000 people died in the unevacuated Wencheng commune of Suipin County next to Shahedian, and the Daowencheng Commune was wiped from the map, killing all 9,600 citizens. Although a large number of people were reported lost at first, many of them returned home later. Tens of thousands of them were carried by the water to downriver provinces and many others fled from their homes. It has been reported that around 90,000 - 230,000 people were killed as a result of the dam breaking. To protect other dams from failure, several flood diversion areas were evacuated and inundated, and several dams deliberately destroyed by air strikes to release water in desired directions. Finally, the Bantai Dam, holding 5.7 billion m^3 of water, was bombed.The Jingguang Railway, a major artery from Beijin to Guanzhou, was cut for 18 days, as were other crucial communications lines. Although 42,618 People's Liberation Army troops were deployed for disaster relief, all communication to and from the cities was cut. Nine days later there were still over a million people trapped by the waters, relying on airdrops of food and unreachable to disaster relief. Epidemics and famine devastated the trapped survivors. The damage of the Zhumadian area was estimated to be about US$ 513 million (Yi Si, 1975).

Thandasveswara, (2007) reported some dam failures in India and America as follws; The Kaila Dam in Kachch, Gujarat, India was constructed during 1952 - 55 as an earth fill dam with a height of 23.08 m above the river bed and a crest length of 213.36 m. The storage of full reservoir level was 13.98 million m^3. The foundation was made of shale. The spillway was of ogee shaped and ungated. The energy dissipation devices first failed and later the embankment collapsed due to the weak foundation bed in 1959.

The Kodaganar Dam, Tamil Nadu, India. was constructed in 1977 on a tributary of Cauvery River as an earthen dam with regulators, with five vertical lift shutters each 3.05 m wide. The dam was 15.75 m high above the deepest foundation, having a 11.45 m of height above the river bed. The storage at full reservoir level was 12.3 million m^3, while the flood capacity was 1275 m^3/s. A 2.5 m free board above the maximum water level was provided. The dam failed due to overtopping by flood waters which flowed over the downstream slopes of the embankment and breached the dam along various reaches. There was an earthquake registered during the period of failure although the foundation was strong. The shutters were promptly operated during flood, but the staff could only partially lift the shutters, because of failure of power. Although a stand-by generator set was commissioned soon, this could not help and they resorted to manual operation of shutters. Inspite of all efforts, water eventually overtopped the embankment. Water gushed over the rear slopes, as a cascade of water was eroding the slopes. Breaches of length 20 m to 200 m were observed. It appeared as if the entire dam was overtopped and breached.

Nanaksagar Dam, situated in Punjab in northwestern India, was constructed in 1962 at Bhakra, with a reservoir capacity of 2.1 x 10^6 m^3. An estimated maximum discharge of 9,711 m^3/s had occurred on August 27, 1967, due to heavy monsoon rains that were heaviest in twenty years. This caused the dam to fail. The water that gushed through the leakage created a 7.6 m breach, which later widened to 45.7 m. The condition of the reservoir had worsened, causing a 16.8 m boil downstream of toe, which was responsible for the settlement of the embankment. The dam was overtopped, causing a breach 150 m wide. A downstream filter blanket and relief wells were provided near the toe but were insufficient to control the seepage. The relief wells each 50 mm in diameter were spaced at a distance of 15.2 to 30.4 m.

The Vaiont Dam is an arch dam, 267 m high. During the test filling of the dam, a land slide of 0.765 Mm^3 volume occurred into the reservoir and was not taken note of. In 1963, the entire mountain slide into the reservoir. The volume of the slide being about 238 Mm^3, and was slightly more than the reservoir volume itself. This material occupied 2 km of reservoir up to a height of about 175 m above reservoir level. This resulted in a overtopping of 101 m high flood wave, which caused a loss of 3,000 lives.

The Khadkawasla Dam, near Pune in Maharashtra, India was constructed in 1879 as a masonry gravity dam, founded on hard rock. It had a height of 31.25 m above the river bed, with a 8.37 m depth of foundation. Its crest length was 1.471 m and had a free board of 2.74 m. The dam had a flood capacity of 2,775 m^3/s and a reservoir of 2.78 x10^3 m^3. The failure of the dam occurred because of the breach that developed in Panshet Dam, upstream of the Khadkawasla reservoir. The upstream dam released a tremendous volume of water into the downstream reservoir at a time when the Khadkawasla reservoir was already full, with the gates discharging at near full capacity. This caused overtopping of the dam because inflow was much above the design flood. The entire length of the dam spilled 2.7 m of water. Vibration of the structure was reported, as the incoming flood was battering the dam. Failure occurred within 4 hrs of the visiting flood waters.

The Panshet Dam, near Pune in Maharashtra India, was under construction when the dam failed. It was zoned at a height of 51 m and having an impervious central core outlet gates located in a trench of the left abutment and hoists were not fully installed when floods occurred at the site of construction. The reservoir had a capacity of 2.70 million m^3. Between June 18 and July 12, 1961, the recorded rainfall was 1778 mm. The rain caused such a rapid rise of the reservoir water level that the new embankment could not adjust to the new loading condition. The peak flow was estimated at 4870 m^3/s . Water rose at the rate of 9 m per day initially, and then up to 24 m in 12 days. Due to incomplete rough outlet surface, the flow through was unsteady and caused pressure surges. Cracks were formed along the edges of the right angles to the axis of the dam causing a subsidence of 9 m wide. An estimated 1.4 m of subsidence had occurred in 2.5 hrs, leaving the crest of the dam 0.6 m above the reservoir level. Failure was neither due to insufficient spillway capacity nor due to foundation effect. It was attributed to inadequate provision of the outlet facility during emergency. This caused collapse of the structure above the outlets.

More recently, Wikipedia (2012) conducted a survey on major dam failures around the world and came out with results as detailed in Table 2.1.

Table 2.1; Major dam failures around the world

Dam/incident	Year	Location	Details
Marib Dam	575	Sheba Yemen	Unknown (possibly neglect)
Pantano de Puentes	1802	Lorca, Spain	608 deaths, 1800 houses and 40000 trees destroyed
Dale Dike Reservoir	1864	South Yorkshire, , England, United Kingdom	Defective construction, small leak in wall grew until dam failed.
South Fork Dam	1889	Johnstown Pennsylvania, United Stated	Blamed locally on poor maintenance by owners; court deemed it an "Act of God". Followed exceptionally heavy rainfall. Caused Johnstown flood.
Walnut Grove Dam	1890	Wickengurg Arizona Teritory, United Stated	Heavy snow and rain following public calls by the dam's chief engineer to strengthen the earthen structure.
McDonald Dam	1900	Texas, United Stated	Extreme current caused failure.
Hauser Dam	1908	Helena Montana, United Stated	Heavy flooding coupled with poor foundation quality
Austin Dam	1911	Austin Pennsylvania, United Stated	Poor design, use of dynamite to remedy structural problems.
Desná Dam	1916	Desna,Austria - Hungry (now Czech Rebublic)	Construction flaws caused the dam failure
Lake Toxaway Dam	1916	Transylvanian Country, North Carolina	Heavy rains caused the dam to give way. Dam was later rebuilt in the 1960s
Sweetwater Dam	1916	San Diego County, California	Over-topped from flooding
Lower Otay Dam	1916	San Diego County, California	Over-topped from flooding; 40 deaths
Gleno Dam	1923	Province of Bergamo, Italy	Poor construction and design
Llyn Eigiau dam and the outflow also destroyed Coedry reservoir dam.	1925	Dolgarrog, North Wales, UK	Contractor blamed cost-cutting in construction but 25" of rain had fallen in preceding 5 days. This was the last dam failure to cause death in the UK to date (2010).
St. Francist Dam	1928	Valencia California, Los Angeles, United	Geological instability of canyon wall that could not have been detected with available technology of the time, combined with

Dam/incident	Year	Location	Details
		Stated	human error that assessed developing cracks as "normal" for a dam of that type.
Nanty Gro Reservoir in Wales	1942	Nanty Gro Valley, Wales	Destroyed during preparation for Operation Chastise in World War II.
Eder, Mohne Dams	1943	Eder Valley, Ruhr, Germany,	Destroyed by bombing during Operation Chastise in World War II.
Vega de Tera	1959	Ribadelgo, Spain	144 deaths
Malpasset	1959	Cote d' Azur, France,	Geological fault possibly enhanced by explosives work during construction; initial geo-study was not thorough.
Baldwin Hills Reservoir	1963	Los Angeles, California, United Stated	Subsidence caused by over-exploitation of local oil field
Spaulding Pond Dam (Mohegan Park)	1963	Norwich, Connecticut, United Stated	6 deaths, more than $6 million estimated damages
Vaion Dam	1963	Italy	Strictly not a dam failure, since the dam structure did not collapse and is still standing. Filling the reservoir caused geological failure in valley wall, leading to 110 km/h landslide into the lake; water escaped in a seiche over the top of dam. Valley had been incorrectly assessed stable.
Mina Plakalnitsa, (Vratsa)	1966	Vratsa Bulgaria, Bulgaria	A tailings dam at Plakalnitsa copper mine near the city of Vratsa failed. A total 450,000 cu m of mud and water inundated Vratsa and the nearby village of Zgorigrad, which suffered widespread damage. The official death toll is 107, but the unofficial estimate is around 500 killed.
Buffalo Creek Flood	1972	Wesy Virginia, United Stated	Unstable loose constructed dam created by local coal mining company, collapsed in heavy rain
Canyon Lake Dam	1972	South Dakota, United Stated	Flooding, dam outlets flooded with debris.
Banqiao and Shimantan Dams	1975	China	Extreme rainfall beyond the planned design capability of the dam
Teton Dam	1976	Idaho, United Stated	Water leakage through earthen wall, leading to dam failure.
Laurel Run Dam	1977	Pennsylvania, United Stated	Heavy rainfall and flooding that over-topped the dam.
Kelly Barnes Dam	1977	Georgia, United Stated	Unknown, possibly design error as dam was raised several times by owners to improve power generation.
Machchu-2 Dam	1979	Morbi, Gujarat,	Heavy rain and flooding beyond spillway

Dam/incident	Year	Location	Details
		India	capacity.
Wadi Qattara Dam	1979	Bengazi, Libya	Flooding beyond discharge and storage capacity damaged the main dam and destroyed the secondary dam in the scheme.
Lawan Lake Dam	1982	Rocky Mountain Nattional Park, United Stated	Outlet pipe erosion; dam under-maintained due to location
Tous Dam	1982	Valentia Spain	
Val di Stava Dam collapse	1985		Poor maintenance and low margin for error in design; outlet pipes failed leading to pressure on dam.
Upriver Dam	1986	Washington state, United Stated	Lightning struck power system, turbines shut down. Water rose behind dam while trying to restart. Backup power systems failed, could not raise spillway gates in time. Dam overtopped(rebuilt).
Peruca Dam detonation	1993	Croatia	Not strictly a dam failure as there was a detonation of pre-positioned explosives by retreating Serb Forces.
Saguenay Flood	1996	Quebec, Canada	Problems started after two weeks of constant rain, which severely engorged soils, rivers and reservoirs. Post-flood enquiries discovered that the network of dikes and dams protecting the city was poorly maintained.
Meadow Pond Dam	1996	New Hampshire, United Stated	Design and construction deficiencies resulted in failure in heavy icing conditions
Opuha Dam	1997	New Zealand	Heavy rain during construction caused failure, dam was later completed
Vodní nádrž Soběnov	2002	Sobenov, Czech Republic	Extreme rainfall during the 2002 European floods
Zeyzoun Dam	2002	Zeyzoun, Syria	Failed 4 June 2002, killing 22 and affecting 10,000.
Ringdijk Groot-Mijdrecht	2003	Wilnis, Netherlands	Peat dam became lighter than water during droughts and floated away
Hope Mills Dam	2003	North Carolina, United Stated	Heavy rains caused earthen dam and bank to wash away
Big Bay Dam	2004	Missippi, United Stated	A small hole in the dam, grew bigger and eventually led to failure.
Camara Dam	2004	Brazil	
Shakidor Dam	2005	Pakistan	Sudden and extreme flooding caused by abnormally severe rain, 70 deaths
Taum Sauk reservoir	2005	Lesterville Missouri, United	Computer/operator error; gauges intended to mark dam full were not respected; dam

Dam/incident	Year	Location	Details
		Stated	continued to fill. Minor leakages had also weakened the wall through piping.
Campos Novos Dam	2006	Compos Novo, Brazil	Tunnel collapse
Gusau Dam	2006	Gusau Nigeria	Heavy flooding
Ka Loko Dam	2006	Kauai, Hawaii	Heavy rain and flooding. Several possible specific factors to include poor maintenance, lack of inspection and illegal modifications.
Lake Delton	9 June 2008	Lake Delton, Wisconsin	Failure due to June 2008 Midwest floods.
Koshi Barrage	2008	Kusha, Nepal	Heavy rain
Algodoes Dam	27 May 2009	Piau, Brazil	Heavy rain
Situ Gintung Dam	2009	Tangerang, Indonesia	Poor maintenance and heavy monsoon rain
Kyzl-Agash Dam	2010	Kazakhstan	Heavy rain and snowmelt
Hope Mills Dam	2010	North Carlina, United Stated	Sinkhole caused dam failure
Delhi Dam	2010	Iowa, United Stated	Heavy rain, flooding.
Ajka alumina plant accident	October 4, 2010	Hungary	Failure of concrete impound wall on alumina plant tailings dam.
Kenmare Resources tailings dam	October 8, 2010	Mozambique	Failure of tailings dam at titanium mine.
Fujimina Dam	March 11, 2011	Japan	Failed after 2011 Tohoku earthquake.
Dam in Campos de Goytacazes, Brazil	January 4, 2012	Rio de Janeiro State, Brazil	Failed after a period of flooding.
Ivanovo	February 6, 2012	Biser, Bulgaria	Failed after a period of heavy snowmelt. A crack in the dam went un-repaired for years. Eight people killed and several communities flooded.
Kopru Dam	February 24, 2012	Adana Province, Turkey	A gate in the diversion tunnel broke after a period of heavy rain; killing ten workers and leaving as many as 5 workers missing.

Source; Wikipedia, (2012)

2.5 Modes of Dam Failures

Anonymous, (2003) pointed out that earth dam failures can be grouped into 3 general categories viz: overtopping, seepage and structural failures. The three types of failure are often interrelated in a complex manner. Uncontrolled seepage for example, may weaken the soil and lead to structural failure. A structural failure may shorten the seepage path and consequently lead to a piping failure while surface erosion may result in structural failure.

One of the most exhaustive surveys of dams which suffered damage or failure was prepared by Middlebrooks (1953) with reasons for the failures listed as shown in Table 2.2

In spite of taking great care in the construction of earth dams, some failures have occurred in the past and in recent times. However, knowledge of the principal lessons learned from failures and damages in the past is an essential part of the training of earth dam designer. (Pumia and Lal, 1992). On the basis of investigation reports on past failures by the same authors, it is possible to categorize the types of failures into three main broad classes namely; Hydraulic (40%), Seepage (30%) and Structural failures (30%).

Investigations carried out by Arora (2001) also showed that about 35% of failures of earth dams are due to hydraulic failures, while about 30% and 20% are attributed to seepage and structural failures respectively. The remaining 7% of the failure are due to other miscellaneous causes such as accidents and natural disasters.

Table 2.2 Frequent Reasons for Dam Failures

Description	% Contribution
Overtopping	30
Seepage(Piping and Sloughing)	25
Slides	15
Conduit leakage	13
Damage due to slope Paving	5
Miscellaneous	7
Unknown	5
Total	**100**

Source; Middlebrooks , (1953)

2.6 Dams in Nigeria

The National Sub-committee on Dams, reported in 1995, that Nigeria, to date, has over 200 large and medium-scale dams, some still at different stages of construction. The report indicates that while 90 % of the dams are multi-purpose in nature, they are also capable of impounding at least 31 billion cubic litres of water for irrigation, hydropower generation; recreation and fishery purposes. "Less than 20 of the dams are functioning at optimal capacity. The rest are either collapsed or abandoned. There are abuses of safety standards and locations," the report further reveals. Analysts say that the history of dams' failure, collapse and dilapidation in the country is quite amazing and unsettling, and they point to many disasters which had occurred as a result of such lapses, lending credence to the report findings (Daily Triumph, 2011)

In the past three decades, over 323 dams have been constructed in Nigeria and many more are under construction in different parts of the country. Between 1970 and 1995, 246 dams were constructed in the country. The effect of the sahelian drought of 1972 – 1975 aggravated the food shortage in the country prompting the various levels of government to embark on a rigorous policy to increase food production. To achieve this, impoundment of river basins was seen as inevitable to provide sufficient water for year-round irrigation which led to the construction of over 246 dams (Imevbore et al, 1986).

In Nigeria, most dams are constructed mainly by the Federal Ministry of Agriculture and Water Resources and about 81% are earth dams (Gundiri, 2004). Earth dams were made even popular since the creation of River Basin Development Authorities (RBDA) under the Federal Ministry of Water Resources and Rural Development by Decree No. 25 of 1976 (Umaru et al, 2010).

2.7 Dam Failures in Nigeria

In Nigeria, earth dam's failure can be catastrophic involving lives and properties. There have been several cases of dam-related disasters in Nigeria displacing thousands of people and plunging them into poverty and destroying properties (Lukman et al, 2011). Instances of such disasters abound and they include the Goronyo Dam in Sokoto State, which failed twice in 1988 and 2010 (a space of 20 years) Bagauda Dam in Kano, which collapsed on Aug. 16, 1988 after two days of intense rainfall, as well as the Cham Dam in Gombe state, which failed in 1998.The collapsed dams also include the Bagoma Dam in Kaduna state, which gave way in 1994 due to a piping through in the dam's foundation; the Obudu Dam in Cross Rivers state that caved in on October 3, 2003, causing the death of 4 persons, aside from the destruction of bridges, roads and homes due to flooding. In Bauchi state, the Wayam Dam also collapsed in 1997 and rendered many members of communities homeless. Nevertheless, experts blame such disasters on the lack of maintenance works on the dams, as well as other forms of structural defects during constructions, inadequate spillway capacities, overtopping of the dams, seepage piping through the dams, poor construction and human errors. In almost all the failures, emergency action plans or warning mechanisms were not provided for in the designs, which could have limit the havoc wreaked on areas downstream of the dams (Daily Triumph, 2011)

Umaru, (2001) gave the general account of failures of earth dams in Nigeria, highlighting this with the mode of construction, mode of failure and reasons for failure of each earth dam (Table 2.3). Table 2.4 summarizes the results on modes of failures of earth dams in Nigeria.

Table; 2.3 Reasons for the failure of earth dams in Nigeria

Name (Mode of Construction)	Mode of Failure	Reasons For Failure
Goronyo Dam (Contract)	Seepage	Foundation and embankment washed away as a result of seepage.
Nasko Dam (Direct Labour)	Structural & Hydraulic	Embankment cracks and subsequent wash away by water.
Obudu Dam (Direct Labour)	Structural & Seepage	Cracks on the Embankment and subsequent failure of the downstream slope.
Bagauda Dam (Direct Labour)	Sturctural & Hydraulic	Borrows on the Embankment due to termite infestation and washing away by water
Yakurr Dam (Direct Labour)	Hydraulic	Siltation of the reservoir and vegetal overgrowth, leads to overtopping.
Ajiwa Dam (Direct Labour)	Hydraulic Structural	Excessive erosion and subsequent cracking of the embankment.
Bagoma Dam (Direct Labour)	Hydraulic	Overtopping by unprecedented flood.
Girei Dam (Direct Labour)	Seepage	Siltation of the reservoir and failure of the spillway. Excessive Seepage and failure of the Spillway.
Cham Dam (Direct Labour)	Hydraulic, Structural & Seepage.	Overtopping, seepage at different parts of the embankment, cracks and slides on the body of the embankment.
Waya Dam (Direct Labour)	Hydraulic	Seepage, Piping, Overtopping, sliding of downstream shell foundation and spillway failure.
Alau Dam (Contract)	Hydraulic	Overtopping and washing away of the dike by an unprecedented flood.
Paki Dam (Direct Labour)	Hydraulic	Collapse of the embankment as a result of deterioration of the spillway and lack of maintenance.
Banki Dam (Direct Labour)	Hydraulic	Siltation of the reservoir, lack of maintenance and subsequent overtopping of the dam.
Garkida Dam (Direct Labour)	Hydraulic	Piping, vegetal over growth and lack of maintenance of the embankment resulting in collapse.
Bagel Zungur Dam (Direct Labour)	Hydraulic	Lack of maintenance and subsequent collapse of the embankment.
Kamal Dam (Direct Labour)	Hydraulic	Overtopping and foundation slide.
Tsohuwar-goram Dam (Direct Labour)	Hydraulic	Overtopping by unprecedented flood.

Surce; Umaru (2001)

Table 2.4 Percentage mode of failure on falied dams.

Mode of failure	Percentage of failed dams
Hydraulic	60
Seepage	11
Structural	0
Hydraulic & structural	16
Structural & seepage	5
Hydraulic, seepage and structural	5
Unknown (natural)	3
TOTAL	100

Source; Umaru, (2001)

Some notable dam failures and incidences in Nigeria are as follows;

(a) Shiroro Dam; Over 26 villages in Kede, Lakpma and Shiroro Local Government in Niger State were flooded by the waters from Rivers Niger and Kaduna in 2003. The flood displaced about 10,000 persons in Ketsho in Kede Local Government who were said to have moved to Kwara State, while other 13,500 person in Lakpam and Shiroro were rendered homeless. In the affected areas, houses, property, farm produce and animals were destroyed by the flood which struck in the early hours of 11th September, 2003. The flood resulted from a downpour and the release of excess water from the Shiroro Hydro-Electric Dam by the National Electric Power Authority (NEPA). The affected villages include Galadima Kogo, Gofa, Kusasun, Pai, Lagado, Nakpinda and Karai. The people suffered for the sacrifice they made by releasing their land for the construction of the Shiroro Dam for the good of the nation. (Etiosa, 2006)

Similarly in 1999 at least seven local government districts in the state were flooded when water from the Shiroro Dam was released. Thousands of houses and buildings in the state, including schools and hospitals were either destroyed or damaged in the disaster.

(b) Ojirami Dam: On 30th August 1980, the Ojirami dam failed and inundated the Akuku and Enwan communities. The failure was mainly due to technical breakdown and negligence on the part of the dam official on duty. Moreover, no alarm was installed to give warning to local officials and communities when the water exceeded its limit in the reservoir. The flood destroyed more than 180 houses in the Akuku community and many people lost their houses and other properties worth millions of Naira. Although the flood did not directly cause any deaths at the time of the failure, numerous casualties were reported due to the resulting poor environmental and sanitation conditions. Residents now suffer from housing shortages, resulting in overcrowded living environments. Many community members lost their local businesses due to the catastrophe and were left without a means of livelihood (Hope, 2003)

(c) Tiga and Challawa Dams: In August 2001, over 40 people were feared dead and more than 20,000 people were displaced by the flood resulting from the failure of the Tiga and Challawa dams in Niger and Jigawa States, Nigeria.

(d) Obudu Dam: The Obudu Dam spillway was damaged by storm in July 2003 and resulted in fatal disaster that claimed over 200 houses, several farmlands, settlements and business concerns. The disaster was allegedly caused by the release of excess water from the Lagdo Dam in Cameroun, which overflowed Benue and Niger River banks. Besides the release of excess water from Lagdo Dam, expert attributed the disaster to intensive and non-stop rainfall in Obudu on the fateful day for 16 hours. The rainfall recorded at the Obudu Dam meteorological station was 314.5mm, more than 15 years average rainfall for the peak months of July and September, and this was not anticipated when the dam was constructed. The cumulative effect of these events, led to the overflow of all water courses.The excessive flood discharge and load on spillway channel led to the failure of the dam. Then, the estimated cost of rehabilitating the dam and completing the outstanding works on the irrigation area was valued at about N350m. (Daily Champion, 2003).

(e) Igabi Dam; Property worth about N500 million ($3.9m) were destroyed while thousands of people were rendered homeless in Kaduna State when River Kaduna overflowed its banks and submerged several streets and housing estates. The flood was caused by the collapse of Igabi Dam. Affected by the flood are Mamman Kotangora Estate, Kirgo Road extension, Kabala area and parts of Malali Estate. At the Mamman Kotangora Estate, household items including rugs, television sets, fridges, chairs, tables and other expensive electronics were damaged when water from the river submerged most of the houses there. Several mechanic workshops, grocery stores and harmaceutical shops were also submerged. At Kirgo area, apart from household items, maize and sugar cane farms were also destroyed. It was learnt that a manual irrigation system constructed by some farmers in the area made it possible for the river water to submerge places like Mamman Kotangora Estate and Kabala area. Apart from churches and mosques which were destroyed, the Nsukka town hall located at Kirgo Road extension was also affected (Etiosa, 2006).

In Nigeria it was observed that earth dams fail due to, human error in feasibility studies, design and construction defects, overtopping, piping, cracking of embankment, slides, excessive erosion, termite infestation, vegetal overgrowth, siltation, general lack of maintenance and lack of dam safety monitoring teams (Umaru, 2001).

2.8 Reasons for Dam Failures

One of the most serious causes of failure and damage of earth dams can be attributed to lack of adequate application and mastering of the engineering properties of soils during design and construction of such dams.

In the early times, Terzaghi in his experience in geotechnical engineering encountered many cases of failure of dams, that resulted mainly to inability to predict and control groundwater. Among the frequently occurring type of failures were; piping, slope failures, bearing capacity failures and excessive settlements.(Burland, 2006)

Muhunthan and Schofield (1999) disagree with Casagrande (1975) that liquefaction occurs when the soil is at the dry side of critical states(near zero effective stress) and in the presence of high hydraulic gradients. Their work refers to some aspects of the failures of Fort

Peck, Baldwin Hills, and Teton dams in America to support their argument. Casagrande (1975) held an opposite view that liquefaction occurs by a chain reaction among sand grains on the wet side of critical states. A model for ductile stable yielding and deformation of an aggregate of grain wetter than critical states is provided by Cam-clay. A layer of such sediment can form folds during deformation. If a soil aggregate is more dense (dry) than critical states, it can fail with fault plains on which gouge material dilates and softens, or it can fracture and crack into a clastic debris, or develop pipes and channels. The critical explanation of rapid failure is rapid transmission of pore water pressure through such opening cracks or channels.

The Baldwin Hills and Teton dam failures in America were manifested with cracks and pipes. In the case of the Fort Peck dam failure also in America, Muhuntan and Schofield (1999) observed that high pore pressures from the core hydraulic fill was transmitted in the layer beneath the part of the dam that failed. Casagrande's view of the failure as evidence of a "chain reaction" was thus questioned. Hajime and Kurashima, (2003) found out that diversion dam structures break due to local scouring of the riverbed caused by local flow around them.

Sherard *et al,* (1963) have attempted to provide a summary of the most instructive experiences of dam failures and damages. Of necessity such a summary prepared by a small group cannot draw upon all the experiences which exist. Many factors in addition to their own natural reluctance to publicize their troubles cause owners and engineers to withhold the details about unsatisfactory performances of dams. Experiences with failures remain the exclusive knowledge of a few people, and in other cases the information given to the profession is not complete or wholly correct. As a consequence there are many misconceptions about the frequency, details, and importance of the failures which have occurred. The summary is as follows;

2.8.1 Embankment and Foundation Piping; Piping or progressive erosion of concentrated leaks, has caused a larger number of catastrophic failures than any other action except overtopping, and many of the modern techniques of earth dam design and construction have developed to prevent it. For example, the present stringent requirements for uniformly compacted embankments with emphasis on control of compaction water content and density have been developed to provide dense and homogeneous cores which reduce the incidence of concentrated leaks and resist piping when leaks do develop. Because of such requirements, and because of the introduction of graded filters in the downstream portions of dams, there have been extremely few piping failures in important modern dams.

2.8.2 Mechanics of Piping; As water seeps through the compacted soil embankment or the natural soil of the foundation, the pressure head is dissipated in overcoming the viscous drag forces which resist the flow through the small soil pores. Conversely, the seeping water generates erosive forces which tend to pull the soil particles with it in its travel through and under the dam. If the forces resisting erosion are less than those which tend to cause it, the soil particles are washed away and piping commences. The resisting forces depend on the cohesion, the interlocking effect, and the weight of the particles, as well as the action of the downstream filter if any.

2.8.3 Leaks and Piping; When first observed, the leaks which have led to piping failure have varied considerably in size, and the rates of development have been widely different. At some dams the leak was seen after the first filling of the reservoir; in others it appears only after many years of leak-free operation. In some cases the leakage water first emerge as a small seep which to the naked eye, ran clear for years and then increase gradually until rapid failure occurred. In other cases, a large and muddy leak preceded complete failure by only a few days or hours (Lane and Wohlt, 1961).

The most common cause of embankment leaks has been poor construction control, which can result in inadequately compacted or pervious layers in the embankment, inferior compaction adjacent to concrete outlet pipes or other structures, or poor compaction or bond between the embankment and the foundation or abutment. Embankment leaks through differential settlement cracks have also been a major source of trouble (Lane and Wohlt, 1961).

2.8.4 Resistance to piping; Records of dams which have developed concentrated leaks demonstrate a very wide range of susceptibility to piping. In one study of leaks in 31 dams the influence of the soil properties and the embankment construction method on the piping resistance (that is, on resistance to piping after a condition of leakage exist) was analyzed by Sherard (1959). It was concluded that the embankment soil properties, particularly the plasticity of fines, had a larger influence on piping resistance than the method by which the embankment had been compacted. Results of the study i.e. Sherard, (1959) shows that, embankments constructed of clay with plasticity index greater than 15 demonstrated the highest resistance to piping, while embankments constructed of fine uniform cohesion less sand had the lowest resistance (Sherard, 1959).

2.8.5 Sloughing; Progressive sloughing (or raveling) is a type of damage closely related to piping which have occurred in a few older homogeneous dams. The process begins when a small amount of material at the downstream toe erodes and produces a small slump of miniature slide. It leaves a relatively steep face, which becomes saturated by seepage from the reservoir and slumps again, forming a slightly higher and more unstable face. This raveling process can continue until the remaining portion of the dam is too thin to withstand water pressure and complete failure occurs suddenly as the reservoir breaks through (Sherard, 1953).

Failure of this type has taken place only when the whole downstream portion of the dam has been saturated. In sloughing failures concentrated leaks may or may not develop, but it is possible for the total quantity of leakage to remain small until just before failure.

2.8.6 Differential Settlement Cracks; While the danger of cracking has not been widely publicized or understood by engineers, it is possible that a larger number of leaks which have led to piping failures have originated from embankment cracks than from any source (Casagrande, 1950, Sherard, 1953 and Peterson, 1957).

Although many of these failures have been in small and cheaply constructed dams, a considerable number of larger well-constructed dams have developed alarming cracks in recent years. When a slope slide occurs in an embankment its presence is obvious even to the casual observer and it cannot be easily hidden; but an open crack, which may be potentially more dangerous than a slide, often cannot be discovered except by close observation (Narain, 1962).

A large group of failures which have occurred when reservoirs were filled for the first time have been attributed to piping through leaks along the outlet conduit or at the abutment or foundation contacts. Actually, piping in many of these cases undoubtedly started in embankment cracks. Many such failures took place without witnesses, but even if there had been reliable records of the event leading to failure, it would still have been difficult in many cases to establish the cause with certainty (Marsal, 1960).

2.8.7 Mechanics of Cracking; Cracking develops because portions of the embankment are subjected to tensile strains when the dam is deformed by differential settlement. Depending upon the geometry and relative compressibility of the foundation, abutment, and embankment, earth dams may be twisted in different ways which result in quite different cracking patterns. Cracks may open parallel or transverse to the axis of the dam and may form in vertical or horizontal planes or in any intermediate direction. They may be either localized or continuous for great distances through the impervious core. Cracks may be transverse, longitudinal, or interior cracks not visible on the dam surface or a combination of one or more these (Field, 1923 and Hinderlider, 1923).

2.8.8 Influence of Embankment Properties; The amount of cracking which will develop at a given dam depends on the magnitude of the strain imposed and on the deformability of the embankment. There exist no reliable guides, either from field observation or laboratory tests, for estimating the maximum amounts of embankment settlement which can take place at a given site without the development of cracks, cumulative records of embankment cracking, perhaps supplemented by laboratory research on the stiffness of compacted impervious soils, may in the future provide some definite criteria for the designer (Sherard, 1953).

2.8.9 Embankment and foundation Slides; Slides which are one of the frequent causes of failure occur in earth dams in the same way that landslides develop in natural earth slopes-when the average stress along any sliding surface becomes greater than the average strength. Because earth movements are particular, and phenomena, and because they lend themselves to analytical treatment, the mechanics of these types of failure have received considerable attention by the profession. Present methods of stability analysis have been developed largely as a result of studies of actual landslides, and therefore the designer must understand the mechanics of failure which have occurred in order to ascertain the reliability of his analytical procedures (Middlebrooks, 1953).

Slides can be grouped in to three categories;

1. Slides during construction involving the upstream or downstream slope (or both);
2. Slides on the downstream slope during reservoir operation;
3. Slides on the upstream slope after reservoir sudden drawdown.

Relatively few slides have occurred on rolled-earth dams during construction compared with the number which have developed during the operation of the reservoir. Of these few none have threaten loss of life or damage to property other than to the dam itself (Peterson et al., 1957). Two distinct types of downstream slides have occurred; deep slides which usually pass through the clay foundation, and shallow surface slides. Deep slides nearly always take place during full or almost full reservoir and frequently reduce freeboard by extending further upstream than the upstream edge of the crest. The internal pore water pressures that cause deep slides are the result of seepage from the reservoir through or under the dam. After a slide takes place, there is no relief in this pressure. The unstable vertical slide scarp left standing often slough or slides again until it breaches the dam and releases the flood water in one great flood wave. Many dams have been saved from complete failure after downstream slides only by around the clock emergency action.

Shallow slides, most of which follow heavy rainstorm, do not as a rule extend into the embankment in a direction normal to the slope more than 1 to 1.5 m. Some take place soon after construction, while others occur after many years of reservoir operation.

Although it could be conceivable that upstream slopes could take place during full reservoir, all of the cases (except upstream slopes during construction) none have occurred following reservoir drawdown. Upstream slides have not caused complete failure or loss of water form the reservoir, although they have occasionally blocked the entrances to outlet conduits and made these useless for further reducing the reservoir, sometimes creating an awkward and dangerous situation. Following an upstream slope slide caused by reservoir drawdown, the excess pore water pressure within the embankment soil adjacent to the surface of sliding are dissipated to a large extent. Consequently, there is a lesser continued sloughing and sliding than there is in the case of downstream slides, in which the pore pressures are not likely to be diminished. Since the slide comes to equilibrium at a stage of low reservoir, there is small likelihood of catastrophic failure even though a large earth movement has taken place (Peterson et al., 1957).

2.8.10 Influence of Soil Type; Almost all slide during construction and all deep upstream downstream slides after construction have occurred in dams underline by foundations of clay relatively high in plasticity and natural water content. In addition, a strong correlation existed between the incidence of slides and the use of fine-grained and highly plastic soil in the embankment. From experiences common to many engineers, there seems to be justification for the statement that rolled earth fill dam embankments have not failed by sliding unless the embankments or the foundations consisted of relatively fine-grained soils (Sherard, 1953).

2.8.11 Reservoir Wave Action and Upstream Slope Protection; The erosive action, which caused most of the trouble at earth dams, occurs only at relatively infrequent intervals during unusually bad storms. It normally last for short periods, and since considerable time is required for wave to erode completely through an earth dam even if there were no slope protection, damage from wave action have not caused a serious threat of complete failure except in rear cases. Usually it has necessitated repairs rather than emergency action. Only a few poorly constructed earth dams with completely inadequate freeboard or excessively deep upstream slopes have been in danger of failure from wave action (Sherard, 1953 and Boyce, 1958).

2.8.12 Slope Protection Failures; The upstream slope of most earth dams, have been protected with one of the following materials (in decreasing order of frequency);

a) Dumped rock riprap.
b) Hand- placed rock riprap.
c) Articulated pavement consisting of individual slabs.
d) Monolithic reinforced concrete pavement. (Betram, 1951).

A few dams have been faced with asphalt layers of various types or protected with floating log-booms, but such dams retain small reservoirs which have little or no wave action. The few dams constructed with steel plate on the upstream face have been completely resistant to wave action (Betram, 1951).

2.8.13 Dumped and Hand Placed Riprap; The fact that layers of dump rock riprap are more successful than layers of equivalent carefully hand-placed rock has been suspected for a number of years, but it was not confirmed definitely until the U.S. Corps of engineers comprehensive study by Middlebrooks (1953). The primary reason for the superiority of dumped riprap is the moderate movement of any individual rock has little influence on the integrity of the protective layer. In contrast, if one large rock in a tightly knit, hand placed blanket is moved, the filter is exposed and progressive erosion starts to undercut adjacent rocks. Hand- placed riprap is particularly vulnerable to damage by floating trees and ice layers, which can gauge one or two rocks out of place. Individual rocks in a dumped rock layer are only slightly jostled when rammed by trees or ice, and the layer remains intact.

During a heavy storm the waves on the surface of the reservoir beat repeatedly against the slope just above the reservoir water level, and their energy is dissipated in turbulent action on and within the rocks of a riprap layer. As a wave strikes the slope, the water rushes upward into the riprap and filter layer and then, in the lull before the next wave strikes, tumbles back downward. This action may damage dump rock riprap in two main ways. First if the filter material is too fine, the wave water moving in and out of the riprap may gradually wash the filter out; in an extreme case where the filter is completely removed, the individual rock in the riprap layer settle and expose the embankment to wave erosion. Second, if the average size of rock comprising the riprap is not heavy enough to resist the hydraulic forces generated by the waves, rocks may be literally washed out of the layer (Holtz, 1961).

2.8.14 Damage due to Borrowing Animals; Borrowing animals have been responsible for piping failures in a number of small earth dams and dykes but have not caused trouble in major dams because animal holes do not penetrate to great depth. In the U.S the worst pests have been muskrats and ground squirrels. Muskrats burrow into embankments either to make homes or to dig passages from one pond to another (Dawson, 1950).

2.8.15 Damage Caused by water Soluble Chemicals; The leaching of natural deposits of water soluble materials from abutments and foundations has caused difficulty in some dams. Gypsum which is gradually dissolved by seepage water from the reservoir has been particularly troublesome in this respect (Anonymous, 1976).

2.8.16 Soluble Materials in Embankment Soils; It was thought in previous years that a small percentage of water soluble salts in the embankment were potentially dangerous, but there was never a record of failure or damage from this case. Most engineers today do not think it necessary to test embankment soils for salts except in extreme cases when the soil has an odd light color or when some other suspicious characteristics indicate that large fraction may be water soluble (Field, 1923 and Hinderlider, 1923).

2.8.17 Flow Slides Due to Spontaneous Liquefaction; One of the most difficult problems faced by earth dam designer is the analysis of loose sand foundations against the possibility of liquefaction or flow slides. The performance of existing dams give practically no assistance, since a few major dams have been founded on loose sand foundations and no failures of rolled earth dams have occurred from liquefaction (Cleary, 1914, Hazen and Metcalf, 918).

A number of major earth dams constructed by the hydraulic fill method have developed construction flow slides which were due primarily to liquefaction of the outer granular shells of the embankment. In all of these failures the mechanics of movement were similar. When the high fluid pressure acting in the upstream- downstream direction caused sufficient shear strains in the outer shells of the embankment during construction, flow slides resulted (Anonymous, 1909 and Anonymous, 1910).

2.8.18 Damage Caused by Downstream Deflection in Rock fill Dams with Central Core; No matter how steeply the slopes are constructed, rock fill dams with thin central cores of concrete or earth on rock foundations never develop slides of the type which develop in earth dams. If the downstream slope is too steep, however, the dam crest deflects an excessive amount when the reservoir is been filled for the first time, and it may continue to move gradually downstream with a creep like action. No theoretical methods are available for analysis of this phenomenon, and the few records of movement available give little guidance in the problem of determining the critical downstream slope of a dam of this type (Noetzli, 1932).

2.8.19 Damage Due to Surface Drying; Surface drying cracks have caused a considerable maintenance problems on a few low dams constructed with homogeneous sections of clayey soil. Usually the main cracks, which in extreme cases have been several inches wide, develop near the top of the dam parallel with the crest. They appear to be aided by the tensile stresses at the top of the embankment slopes. The worst conditions develop when combination of the following three factors occurs; (1) hot, dry climates during which the reservoir remains empty for long periods; (2) embankment construction materials of highly plastic or extremely fine silty soil; and (3) embankments not compacted to high densities (Creager, 1939).

The higher the content of clay fines in a poorly constructed embankment, the more the embankment can be expected to shrink and crack. On the other hand, it is the cohesion less, silty materials which are most susceptible to erosion. Some of the worst cases have been in the arid southwestern part of the United States, where, in some cases, homogeneous dams of very fine clayey silt have been badly eroded with concentrated gullies, starting in drying cracks, that they have to be almost completely reconstructed. This type of reconstruction is very awkward and expensive (Creager, 1939).

2.8.20 Drying Cracks During Construction; If the construction surface of an embankment of fine-grained soil is allowed to dry in the sun, drying cracks can generally increase the overall permeability of the material (Fucik, 1952). This happen even on dams constructed in accordance with good modern practice. Because of such problems USBR often recommends that contractors protect completed portions of the embankment against drying out by sprinkling with water or covering with loose earth (Weyerman, 1960).

2.9 Soil and Dam Construction.

A close look at the problems of most failures showed that, the situation may be rescued partly by strict application of engineering properties (physical and hydraulic) of soils and hydrology in the design and construction of the dams.

The engineering properties of soils are those properties that indicate the behavior of the soils during construction and under loading. The laws of mechanics and hydraulics are essentially applied (Geotechnical Engineering) to soil aggregates to arrive at the engineering properties. These properties include, bulk density, porosity, permeability, submerged density, particle size distribution, friction, cohesion and water content among others (Murthy, 2008).

Since soil materials are pervious to smaller or larger degrees, seepage has to take place through earth dams and their foundations. The water seeping under pressure through the soil voids is accompanied by mechanical drag on the soil particles, when these forces exceed the resistive forces of the soil grains, the movement of grains or heaving of the soil at exit end may result.

The adverse effect of seepage also include migration of soil particles resulting in piping failure, excessive pore pressure may result in slope failure, saturation of the downstream slope which may lead to progressive sloughing. These have to be controlled by proper embankment zoning/ or draining of the embankment.

Control of seepage through the embankment as well as the foundation is affected by two approaches, generally used in combination; the first approach involves reduction of the quantity of seepage or keeping the water out as far as possible. In the embankment this requires provision of the impervious zone, generally called the 'core'. The second approach involves providing a safe outlet to the seeping water which still enters the embankment or the foundation in spite of the measures taken in the first category. This requires provision of such drainage arrangements downstream of the seepage barriers so that the seepage forces will not be able to cause soil migration and their magnitude and direction would be such that they cannot cause embankment sliding or sloughing. If the outer zones of the dam are sufficiently pervious to be considered as free -draining, these will therefore serve as drains and no other drainage arrangement may be required. (Oskoorouchi , 1988).

2.9.1 Soils and Design Parameters for Earth Dams Construction

The important soil properties to be considered are permeability, compacted density, shear strength, compressibility, settlement and erosion resistance.

Terzaghi was quoted by Arora, (2001) on the recommended side slopes and soil types for embankment dams as shown in Table 2.5

For maximum economy in the usage of materials, the slopes should be as steep as possible. However, from stability considerations, the slopes should not be excessively steep. Therefore compromise is made. The stability of the slope depends mainly on the shear and deformation characteristics of the materials. The coarse grained materials can have steeper slopes as compared to the fine grained materials. In the case of zoning the slopes are relatively steeper as compared to a homogeneous section of the same material, because stronger materials (coarse - grained materials) are placed in shells where they are most effective in resisting shear stresses. Moreover, there is a better drainage control and reduction in the pore water pressure in a zoned dam than in a homogeneous dam.(Arora, 2001). Signh, (2001) suggested the slopes and soils for small zoned earth dams on stable foundations as in Table 2.6. According to IS: 12169 – 1987, Agarwal, (2000) suggested a general guidelines for embankment sections as shown in Table 2.7 and also suggested the suitability of soils for use in the construction of earth dams as shown in Table 2.8.

Table 2.5 Recommended soil types and slopes for earth dams

Type of section	Type of material	Upstream slope (U/S)	Downstream slope (D/S)
Homogeneous section	Well-graded material	2.5:1	2:1
	Coarse silt	3:1	2.5:1
	Silty clay or Clay		
	(a) Height < 15m	2.5:1	2:1
	(b) Height ≥ 15m	3:1	2.5:1
Zoned section	Sand or gravel shells with clay core	3:1	2.5:1
-do-	Sand or gravel shells with R.C. Core	2.5:1	2:1

Source: Arora (2001)

Table 2.6 Recommended soils and slopes for small zoned earth dams.

Type	Purpose	Subject to rapid drawdown	Shell material	Core material	U/S slope	D/S Slope
Zoned with minimum core 'A'	Any	Not critical	Rock fill,	GC,GM,SC	2:1	2:1
		(Note1)	GW,GP,	SM,CL,ML		
			SW(gravelly or SP(gravelly)	CH, or MH		
Zoned with maximum core	Detention or Storage	No	-do-	GC,GM,	2:1	2:1
				SC,SM,	$2^{1/}{}_{4}$:1	$2^{1/}{}_{4}$:1
				CL,ML,	$2^{1}/_{2}$:1	$2^{1}/_{2}$: 1
				CH,MH	3:1	3:1
Zoned with maximum core	Storage	Yes	-do-	GC,GM	$2^{1}/_{2}$:1	2:1
				SC,SM	$2^{1}/_{2}$:1	$2^{1/}{}_{4}$:1
				CL,ML	3:1	$2^{1}/_{2}$:1
				CH,MH	$3^{1}/_{2}$:1	3:1

Note; Rapid drawdown will not affect the u/s slope of a zoned embankment which has a large u/s pervious shell.

Source: Singh, (2001)

Table 2.7 General Guidelines for Embankment Sections

S.No	Description	Height up to 5 m		Height above 5 m and up to 10m		Height above 10 m and up to 15m	
1.	Types of section	Homogeneous/Modified homogeneous section		Zoned/Modified homogeneous/Homogeneous section		Zoned/modified homogeneous/ homogeneous section	
2.	Side slopes	U/S	D/S	U/S	D/S	U/S	D/S
a)	Coarse grained soil						
	(i)GC, GP, SW, SP	Not Suitable		Not Suitable		Not suitable for core, suitable for casing zone	
	(ii) GC, GM SC, SM	2:1	2:1	2:1	2:1	Section to be decided based upon stability analysis	
b)	Fine grained soil						
	(i) CL, ML, CI, MI	2:1	2:1	2.5:1	2.25:1	-do-	
	(ii) CH, MH	2:1	2:1	3.75:1	2.5:1	-do-	
3.	Hearting zone	Not required		May be provided		Necessary	
	a) Top width	--		3m		3m	
	b) Top Level	--		0.5m above MWL		0.5m above MWL	
4.	Rock toe height	Not necessary up to 3m height. Above 3m height, 1m ht. of rock toe may be provided		Necessary. H/5, where H is height of embankment		Necessary. H/5, where H is height of embankment	
5.	Berms	Not necessary		Not necessary		The berm may be provided as per design. The minimum berm width shall be 3m.	

Source: Agarwal, 2000 as adopted from IS 12169 - 1987

Table 2.8 Suitability of soils for construction of earth dams

Relative Suitability	Homogenous Dykes	Zoned Dams		Impervious Blanket
		Impervious core	Pervious casing	
Very Suitable	GC	GC	SW,GW	GC
Suitable	CL, CI	CL, CI	GM	CL, CI
Fairly suitable	SP, SM, CH	GM, GC, SM, SC, CH	SP, GP	CH, SM, SC, GC
Poor	-	ML, MI, MH	-	-
Not suitable	-	OL, OI, OH, Pt	-	-

Source; Agarwal, 2000 as adopted from IS 12169 - 1987

Brink *et al* (1982) suggested that the engineering properties of soils used for the construction of the zones of composite earth dams should include; grade of the soil, clay content, hydraulic conductivity(permeability), cohesion and angle of internal friction, liquid limit, plasticity index, optimum moisture content, linear shrinkage and dry density. The acceptable ranges are shown in Table 2.9

Yohana et al (2003) tested the engineering properties of anthills and found that the properties are similar to what Brink *et al* (1982) suggested. They recommended its use with mixtures of sand and gravel for the control of seepage in earth dams.

An attempt at approximate classification of core materials on the basis of resistance to concentrated leakage was proposed by Sherard (1953) as;

(1) Very Good Materials (2) Good Materials (3) Fair Materials (4) Poor Materials and

(5) Very poor Materials.

Oskooruchi and Mehdibeigi, (1986) suggested that the selection of soil parameters for designing an earth and rockfill dam should be based on the following;

(i) Visit site and pay attention to the source of soil formation and geological origin of construction materials.
(ii) Compare the above information with similar sites and constructed projects.
(iii) A complete set of physical and classification test on borrow materials should be made.
(iv) Run a limited number of engineering test on the selected construction materials
(v) Make selection of soil base on information obtained from steps (i) to (iii) and close to (±30%) the minimum values of step (iv) provided that the factor of safety ($F.S._{eq}$(min)) be kept close to unity.
(vi) In their specific investigations there was no need to choose F.S' s greater than 1.5 for the stability of the d/s and u/s slopes.

Table 2.9 Engineering properties of soils with acceptable ranges for the zones of a composite earth dam.

Soil Parameter	Acceptable limits for the different Zones		
	Impermeable Core	Semi-permeable transit	Permeable shell.
Grading	Fine	Medium	Coarse
Clay (%)	10-30	5-10	5
Liquid limit(%)	25-60	25	20
Plasticity Index	10-30	10	5
Linear Shrinkage(%)	6-14	5	2
Optimum moisture content (%)	12-25	10-15	8-12
Dry Density(Kg/m^3)	14-16.5	15.5-17.5	16.5-17.0
Angle of shearing resistance(^{0})	20-30	30-35	35
Cohesion(*Kp*)	25-50	25	25
Permeability(m/s)	1x10^{-9}	1x10^{-7}	1x10^{-5}

Source : Brink et al (1982)

Apart from the soil parameters, hydrological parameters are also needed for the design and operation of small earthen dams. Suhr et al (1999) tried to generate these parameters in lower Shiwaliks of India by constructing three water harvesting structures(core-wall type of earthen dams) having catchment areas of 77.2, 66 and 17.3ha.. The study showed that 73%, 77%, and 85% of the total summer monsoon rains could produce runoff with runoff coefficients of 0.22 ±0.03, 0.37 ±0.04, and 0.35 ±0.05, at the respective sites. On the average 1211, 2712 and 2769m^3 of water was harvested per hectare in the structures. From the harvested water, 79%, 78%, and 46% was lost through evaporation and seepage. The major mode of water loss was seepage which varied from 61- 86% at those sites. The water harvesting structures lost their gross storage capacity by 1.30%, 1.08%, and 1.16% per year with siltation rate of 31, 37, and 47t/ha of catchment area at respective sites. All the studies mentioned so far test the materials for dam's construction from borrow pits. Only Oskorouchi and Mehdibeiji (1986) used test results obtained from dams constructed on similar sites with the aim of minimizing costly experiments and increasing the reliability of the data.

This study attempts to go further in that direction by using the design and soil test results from performing, distressed and failed earth dams in the study area, in order to arrive at those properties that most influence failures and induce distress of those dams.

This study is not limited to soil and hydraulic parameters only as they affect earth dam failure. All other relevant factors will also be considered as they influence failure and induce distresses of earth dams in the study area.

2.10.1 General Stability Analysis

The design of embankment dam sections may be divided into the following three categories based upon the height of the embankment in its deepest portion.

(a) where the height of embankment is 5m or less

(b) where the height of embankment is greater than 5m but less than10m.

(c) where the height of embankment is greater than 10m but less than 15m.

For small dams under category (a) and (b) the stability analysis may not be necessary. General guidelines and the recommended side slopes are given in Table 2.7 for guidance of the designer. The minimum top width may be kept at 4.5m. However the designer with his experience and judgment may decide the adequate side slopes where special technical or economic considerations may have to be taken into account.

Stability analysis may be carried out based upon the detailed foundation and borrow area investigation and laboratory testing if the soil strata below the dam seat consist of weak foundation and / or the height of embankment is more than 10m.

Weak foundation conditions include fissured clay, expansive soils, shales, over consolidated highly plastic clays, soft clays, dispersive soils etc. within the substratum in the dam seat.

Main problem of silt and clay foundations is stability. In addition to the obvious danger of bearing failure of foundations of silt and clay, the design must take into account effect of saturation of the foundations of the dam and appurtenant works by the reservoir. The following are methods of treatment for the above problems;

(i) Remove soils of low shearing strength

(ii) Provide drainage of foundation to permit increase of strength during construction

(iii) Reduce magnitude of average shearing stress along potential surface of sliding by flattening slopes of embankment.

Pockets of material substantially more compressible or lower in strength than the average, are usually removed.

The most practicable solution for foundation of saturated fine-grained soils is to flatten the slope of the embankment.

Soils of low density are subjected to large settlements when saturated by the reservoir, although these soils have high dry strength in natural state. If proper measures are not taken to control excessive settlement, failure of dam may occur by differential settlement and foundation settlement. The required treatment of low-density foundation will be dictated by the compression characteristics of the soil. Foundation consolidation will be achieved during construction (Agarwal,2000).

One of the methods of stability analysis is the circular arc method proposed by Sharma and Sharma, (2002). It is also known as the Swedish or Slip Circle method. In this method, the surface of rupture is assumed as cylindrical or in the cross section by an arc of a circle. The method is generally applicable for analyzing slopes of homogeneous earth dams and dams resting on thick deposits of fine grained materials. The assumptions made in this method are;

(i) No shearing stresses act across the plane of the cross section and the analysis is treated as two dimensional,

(ii) Section of dam analysis is of unit thickness,

(iii) The sliding mass is divided into a number of convenient slices and each slice is assumed to act independently of its adjoining slices and the forces acting on the sides of a slice have no influence on the shear resistance which may develop on the bottom of the slice, and

(iv) Shear strength of the various zones along the potential failure surface is mobilized simultaneously.

In the method, a possible circular failure surface through the embankment and foundation (if it is not firm and through which failure is expected) is assumed. The trial sliding mass is divided into a number of vertical slices, usually 10 to 15, of preferably equal width, depending on the width and profile of the sliding mass, number of various zones included in the sliding mass and the accuracy required. For zoned embankment and stratified foundation with different properties, where an arc of the potential failure surface passes through more than one type of material, the vertical ordinates of the slices for each zone or part of foundation are obtained by locating the slice at each such dividing point. Trial surface computations are made of the shear force needed for equilibrium and the strength forces available. Figure 2. 6 depicts an assumed failure surface.

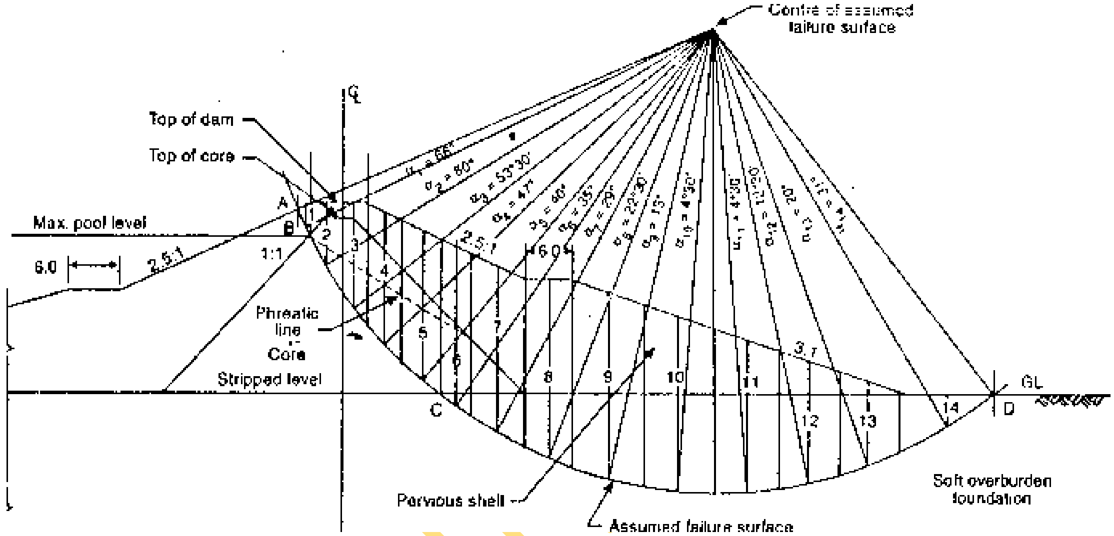


Fig 2.6. Assumed failure surface by Circular arc method.

2.10.2 Stability at Junctions

According to Agarwal (2000) junctions of embankment dam with foundation, abutments, masonry structures like overflow, non-overflow dams and outlets need special attention with reference to one or all of the following criteria:

(i) Good bond between embankment dam and foundations

(ii) Adequate creep length at the contact plane

(iii)Protection of embankment dam slope against scouring action and

(iv)Easy movement of traffic.

2.11 Seepage Control and Safety against Internal Erosion

In the case of seepage through an earth dam the upper boundary or the uppermost flow line is not known. The upper boundary is a free water surface and is referred to as the line of seepage or phreatic line. The seepage line may therefore be defined as the line above which there is no hydrostatic pressure and below which there is hydrostatic pressure. In the design of all earth dams the following factors are important;

(i) The seepage line should not cut the downstream slope.

(ii) The seepage loss through the dam should be the minimum possible.

The two most important problems that are required to be studied in the design of earth dams are:

(a) The prediction of the line of seepage in the cross-section and

(b) The computation of the seepage loss.

If the line of seepage is allowed to intersect the downstream face much above the toe, more or less serious sloughing may take place and ultimate failure may result. This mishap can be prevented by providing suitable drainage arrangements on the downstream side of the dam (Murthy, 2008)

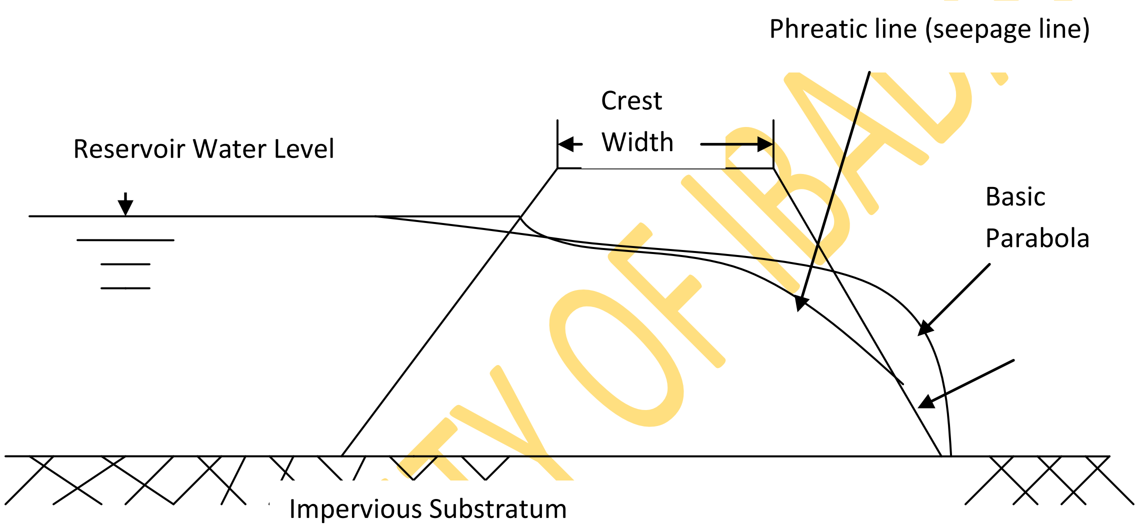


Fig.2. 7. Seepage line in a Homogeneous Earth Dam

The seepage through the dam embankment and foundation should be such as to control piping, erosion, sloughing and excessive loss of water. Seepage control measures are required to control seepage through dam and foundation.(Agarwal,2000).

Nelson (2000) argued that seepage is related not so much to the presence of pervious soils as it is to the prevailing ground water conditions at the site. This can be illustrated by considering an excavated tank dug into sandy soil. If the water table is well below the bottom of the tank, water will seep through sandy soil down to the water table. But, given the same tank in the same pervious soils but with a higher water table, there will be no seepage out of the tank. In fact, if the water were pumped out of the tank seepage water will move in to replace it, thus resulting in seepage gain. This, of course, is the principle of the soakage tank, which supplies many farms in Australia. Nevertheless, many reservoirs are well above water table levels and consequently must be located in relatively impervious soils.

Seepage through a levee is similar to what is happening through earth dams. A steady state two-dimensional unconfined flow through a homogeneous levee with a horizontal toe drain resting on impervious base was analyzed by Mishra and Singh (2005). The shape of the phreatic line is similar to what obtains in the body of a homogeneous earth dam. Unlike in the Kozney method, the hydraulic resistance of the soil in a levee bounded by an equipotential parabolic surface was considered in the computation of the seepage and locating the phreatic line. The appropriate position of a filter is suggested to contain capillary rise well within the downstream sloping surface.

Umani et al (2003) formulated an optimal hydraulic design problem regarding an earth dam cross section as an inverse problem for the steady model of saturated-unsaturated seepage flows in porous media. The choice of soil material to be used in each point of the dam cross sectional domain was considered as control variable to be identified. The performance index used to evaluate the appropriateness of the design is defined as the sum of two squire integral norms, which represent a reduction of the saturated zone and minimum cost of materials. A numerical scheme including pseudo-unsteady terms was developed to calculate the optimal solution in an earth dam cross section to be designed utilizing two different types of soil. The result showed that an inclined clay core of less hydraulic conductivity should be located on the upstream side of the cross section.

In a study conducted by Rengasamy et al (1996) on a red-brown earth (Natrixeralf) to find the effectiveness of spontaneously dispersed clay from sodic soils and mechanically dispersed clay (by puddling) from calcic and sodic soils in reducing the seepage loss of water from a series of small dams(pits), the effect of inoculating algae in the pits on reducing seepage was also investigated. A plastic lined pit was used for water balance control to measure incoming rainfall and evaporation loss.

The result showed the effectiveness of dispersed clay in sealing the surface soil materials in the banks and beds of the pits. The dispersed clays from sodic soils were very effective in reducing the seepage to zero. When the clay concentration was above 8 g/l the sealing was complete, irrespective of spontaneous or mechanical dispersion from sodic soils. The mechanically dispersed clay from calcic soils were less effective in sealing because of the deposition of flocculated materials in the pore systems formed domains and generated micro porosity. In calcic pits, the inoculation of algae reduced the seepage by 13 to 23% and increase in biopolymer (chlorophyll and polysaccharide) production was only small.

2.12 Deliberate Dam Failures

Not all dam failures are accidental as reported by Wikipedia, (2012). Notable case of deliberate dam failure was the British Royal Air force Dambursters raid on Germany in World War II, in which three German dams were selected to be breached in order to impact on German infrastructure, manufacturing and power capabilities deriving from the Ruhr and Eder rivers.

2.13 Dam Break Analyses

Recent studies in dam's failure are geared towards hydraulic analysis of dam failure (dam break analysis). This requires an evaluation of the downstream propagation of a flood hydrograph (wave), which determines the movement of the flood wave as a function of time, so as to provide peak water surface elevation, peak discharges and timing of the peak elevations and discharges at various locations downstream of the dam. Forecasting downstream flash floods due to dam failures is an application of flood routing that has received considerable attention (Larry, 2005).

2.14 Consequences of Dam Construction

The consequence of building dams on the hydro-ecology and socio-economic activities at the dam site is highlighted by Graciela et al, (2006) in their work on Ibera wetlands in Argentina .These consequences include increased ground water inflow, many changes in the ecosystem of the wet land, loss of productive land and changes in the socio-economic activities of the residents.

In Zimbabwe, Tafangenyasha, (1997) showed that the construction of Beiji dam brought environmental degradation in the Beiji catchment which is also threatening the viability of the water reservoir itself as a result of siltation. The cause of siltation at Beiji dam is catchments degradation due to declining wood land, stampeding and overgrazing by over concentration of animals at the water hole and catchments. Catchments erosion has resulted in heavy siltation of Beiji dam. Again, dredging the dam pose an environmental challenge as to the disposal of the silt. The silts may eventually be disposed in some other water bodies or on the land surface; this can obviously silt other water courses and cause loss of valuable land.

Due to the numerous consequences of dam construction, the field of "Dam Removal" is generating a lot of interest among environmentalists and academics all over the world. Dam removal is an increasingly viable approach to watershed restoration from geomorphologic, economic, and ecological perspectives, with over 400 dams already removed, breached, or otherwise taken out of service in the US in recent decades. Owners remove dams for a number of reasons; to avoid expensive repairs required for re-licensing; to preclude liability for hazards associated with antiquated dams; to discontinue maintenance of obsolete structures and to improve fish passage. Environmental damage by dams include channel deterioration, habitat loss, native-species decline and decrease dissolve oxygen levels downstream of dams among other impacts.(Kuby et al, 2005).

In the 5000 or so years that humans have been building dams, millions have been constructed globally, especially in the last 100years. If dams have successfully met human needs, why is there a growing call for their removal? The answers to this question require an appreciation of societies changing needs for, and concerns about dams, including the emerging recognition that dams can impair river ecosystems. But decisions about dam removal are complex in no small part, because great scientific uncertainty exists over the potential environmental benefits of dam removal. More fundamentally, however, a scientific framework is lacking for considering how the tremendous variation in dam and river attributes determines the ecological impacts of dams and the restoration potential following removal.

In the work carried out by Poff and Hart, (2002) they developed a conceptual foundation for the emerging science of dam removal by; (a) reviewing the ways that dams impair river ecosystems, (b) examining criteria used to classify dams and describing how the criteria are of limited value in evaluating the environmental effects of dams, (c) quantifying patterns of variation in some environmentally relevant dam characteristics using governmental databases, (d) specify a framework that can guide the development of an ecological classification, and (e) evaluating the ways that dam characteristics affect removal decisions and future of dam removals.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Description of the Study Area

Nigeria, a West African Nation lies between Latitude 4°16'N and 13°52'N, and between Longitude 2°49'E and 14°37'E. The principal physiographic feature is the Niger and Benue River system which separates three highland blocks (Oke and Ismail, 2012a). The country can be divided into two major geographical zones namely the North and South. Furthermore, both the North and South South are segmented into three regions each, making a total of six geopolitical zones in the country. These six geopolitical zones are; North East, North West, North Central, South West, South East and South. Policies, resources allocations, sites of infrastructures and even political appointments are mostly considered by zoning (Oke and Ismail, 2013).

North Eastern Nigeria (Fig. 3.1 and 3.2) encompasses Borno, Yobe, Bauchi, Gombe, Adamawa and Taraba states. It is the home of a rapidly growing population of some 6.5 million Nigerians. Characterized by water scarcity, the climate of the region ranges from Sahel to Sudan Savannah (Adeniji, 2003). The area has an annual rainfall of between 234mm and 1600mm and has between 3-6 months of rainfall a year, with August and September as the wettest months, while the driest months are February and March. The relative humidity range from 9% to 82%. The choice for the study area is the availability of dams and their collapse and distressnes.

In this study three states were selected out of the six states that make up North Eastern Nigeria(Adamawa, Gombe and Bauchi states). The choice of the sampling is guided by the fact that there are very few dams in Borno and Yobe States and for logistic reasons.

Adamawa state lies between Latitude 7° 28'N and 10 ° 55'N of Equator and Longitude 11° 30'E and 13° 45'E of the Greenwich Meridian, with a population of 3,168,101, in the National Population Census of 2006 (Federal Republic of Nigeria Official Gazette, 2007). The State has an annual rainfall of between 700mm and 1600mm and has between 3-6 months of rainfall a year, with August and September as the wettest months, while the driest months are February and March when relative humidity is about 20% (Adebayo and Umar, 1999). Adamawa State is bounded in the north by Borno State, to the east by the Republic of Cameroon, to the south by Taraba and in the west by Gombe and Borno States (Fig. 3.1).

Gombe State is located in the centre of the north east of the country as shown in the map (Fig. 3.1) with a land area of 20,265 km^2 and a population of 2,353,879 according to year 2006 Census (NPC, 2006). Rainfall in the state has an annual average of 850 mm with temperatures ranging from 41 to 42^oC during the months of March-May considered to be the hottest months.

Bauchi State, on the other hand has a population of 4,706,909 in the National Population Census 2006 (Federal republic of Nigeria Official Gazette, 2007). It is located in the western part of North-East as shown in map (Fig. 3.1).

Agriculture is the main occupation of the majority of the population through subsistence traditional farming (Ismail and Oke; 2012a; b and c; Oke and Ismail; 2013).

Fig. 3.2 and Fig. 3.3 show the locations of the selected dams and the geology of the study area respectively.

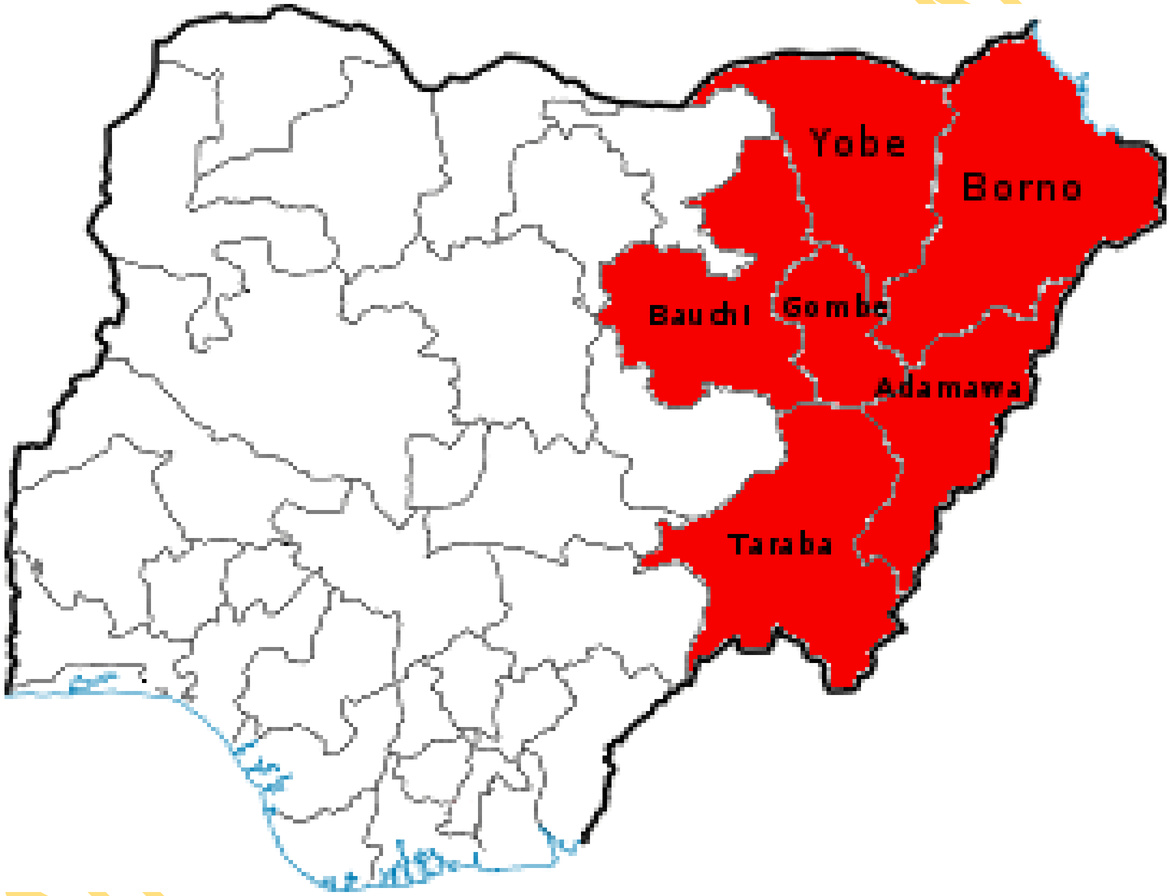


Fig 3.1; Map of Nigeria Showing the North Eastern States

Source (Wikipedia, 2012)

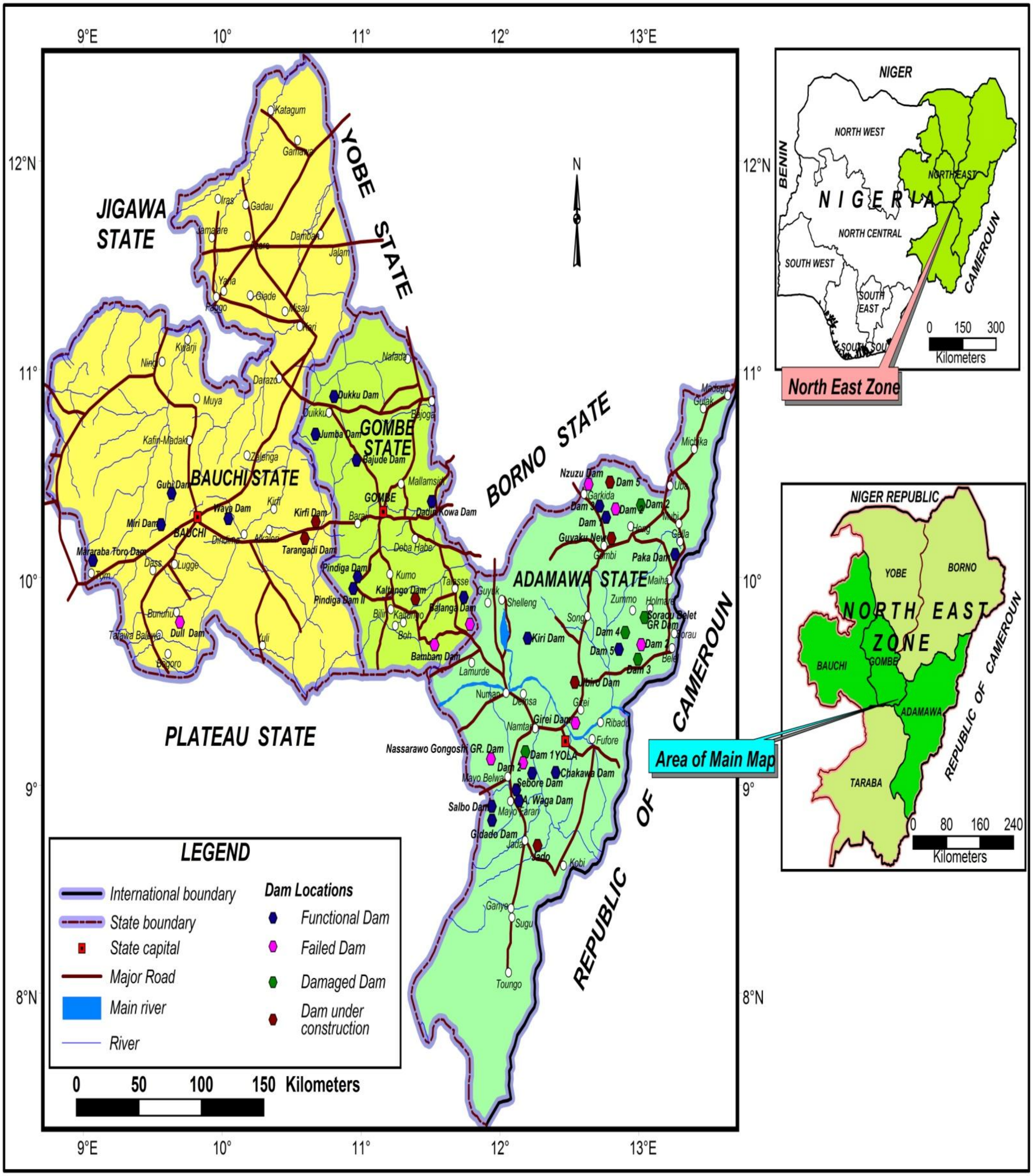


Fig 3.2; Map of Northeastern Nigeria showing dam locations and status

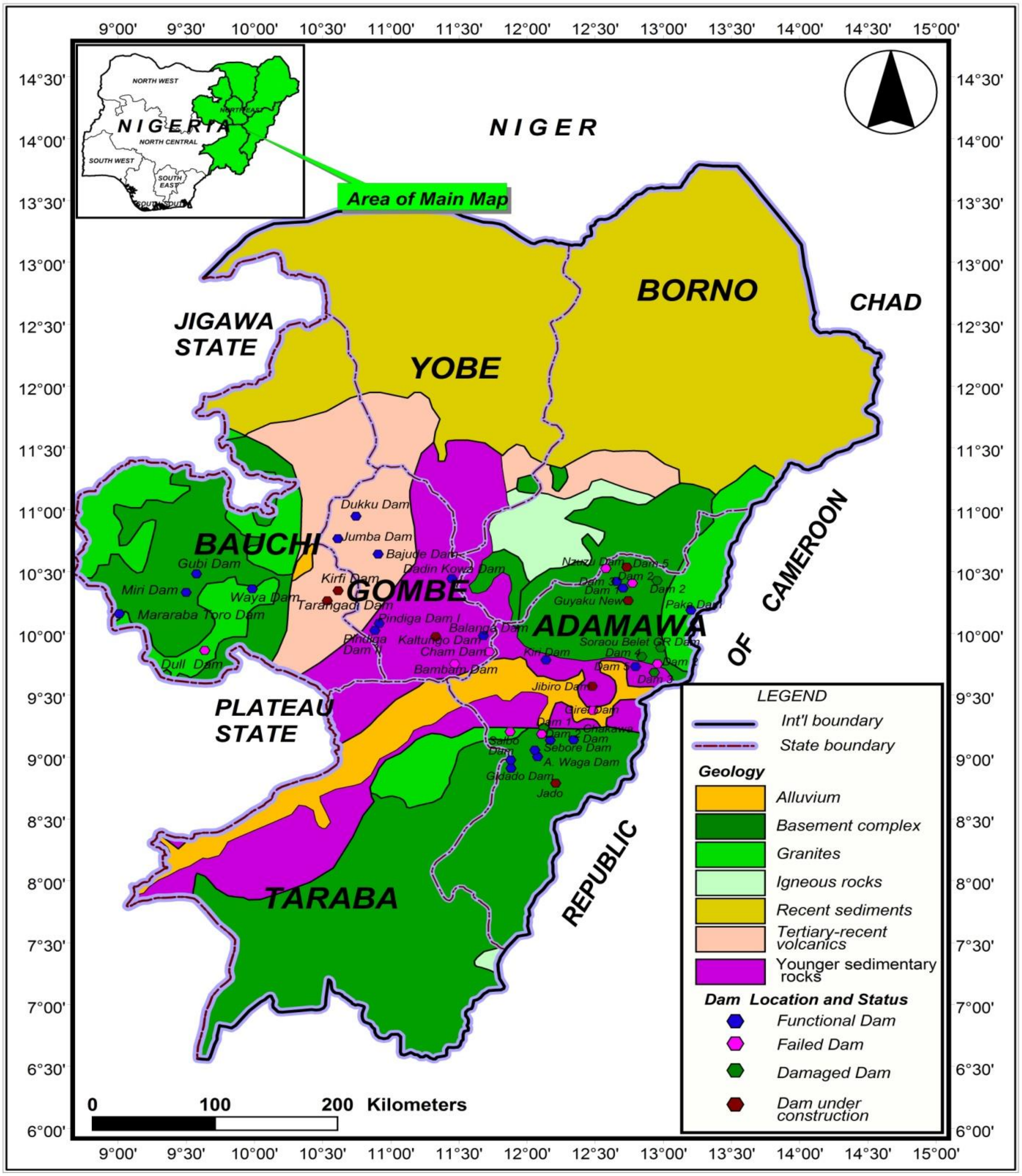


Fig 3.3; Geologic map of Northeastern Nigeria showing dam locations and status

The methodologies adopted for carrying out this research follows the Association of State Dam Safety Officials (ADSO) guideline, (2011) as follows;

3.2 Desktop Research

Desktop research was used to establish a holistic database on the Geology and Hydrometeorology of the study area and relevant information on the earth dams. Materials including, fact sheets, Nigeria register of dams and past reports (archives) were consulted. The following information about the study area and earth dams was sourced from the relevant organizations;

1. Geology and Hydrometreology
2. Design drawings of embankment crossections.
3. Geologic maps.
4. Reports on past dam incidents.

3.3 Field Work

The field work was carried out in form of visitations to selected dams in the study area for observations, measurements, photographs and picking of soil samples where required. The materials used were; measuring tape, Global Positioning System (GPS), camera, soil auger, hoe, shovel and scoop for removing and picking of soil samples and cement bags for carrying and transporting the samples to the laboratory.

For each of the dams visited, a structured questionnaire was administered to source for information based on the following modules so as to standardize desktop information;

Module A was used to source for general information about each of the dams visited; these information include; name of dam, owner, year of construction, mode of construction, embankment type, condition/status of dam, year of failure, loss of life or property, mode of failure, associated causes of failure, the most likely causes of the failure and what could have been done to avoid the failure.

Module B focused on information about design and construction; information requested included; height of dam, length of dam, crest width, reservoir capacity, design life of reservoir, predominant embankment soil material, upstream slope, downstream slope, number of zones and the soil materials in the zones, construction methods, periods of construction, compaction density per layer, thickness of layers for compaction, type of equipment used for compaction and number of passes to achieve desired level of compaction per layer.

Module C dealt with issues of operation and maintenance**;** where the sought information were; how well is the reservoir water utilized, condition/status of the spillway, operation of the spillway, how often is the embankment cleared of shrubs, trees, termites, ants, rodents etc, presence of any maintenance schedule for the reservoir, presence of any safety instrumentations in place, types of safety instrumentations in place, condition of safety instrumentations, presence of any dam safety and monitoring team in place, how equipped is the monitoring team and how often does the team go for training to update skills?. (See Appendix I).

According to the Nigeria register of dams, there are a total of 9 dams in the study area. However, a total of 42 dams were visited for investigation in this study. The Nigeria register of dams is therefore not comprehensive. The maiden edition was produced in 2004, and up till the time of conclusion of this study, there is no review or update of the register.

The 42 dams visited were randomly selected across the study area. They are distributed as follows; 25 dams in Adamawa state, 10 in Gombe state and 7 in Bauchi state. Taraba state was not visited for logistics reasons. Borno state and Yobe state were also not visited because of their arid nature as a result of which they have very few dams.

3.4 Laboratory Experiments

Soil samples were collected where appropriate, placed in cement bags and transported to the laboratory for the necessary tests. Soil tests were carried out in the soil mechanics laboratory of the Civil Engineering programme at the Abubakar Tafawa Balewa University of Technology (ATBU), Bauchi, Nigeria. Methods of test for soils for civil engineering purposes BS 1377(1990) was adopted for the sampling, soil test and analysis.

Based on observation and inspection, soil samples were picked at the appropriate point (failure point, damaged point, stockpiled leftovers) and at different parts (embankment, core, shell, reservoir, spillway etc) of the dam for the tests and analysis. The different types of scientific and engineering tests that the samples were subjected to were; Specific gravity (Gs), Sieve analysis, Atterberg limits (PL, LL, PI), Compaction test (MDD & OMC), California Bearing Ratio (CBR), Permeability (K), Triaxial test (C and ϕ) and Consolidation test. The soil samples were picked from specific functional, distressed and failed dams across the study area for comparison.

Based on the preliminary investigations carried out, specific scientific and engineering tests were recommended and carried out on the collected samples as summarized in Table 3.1. All the data were analysed using Analysis of Variance (ANOVA) and descriptive statistics.

Table 3.1; Recommended soil tests on samples

S/N	Name	Status	Sampling Points	Tests on samples
1.	Girei	Failed	• the Spillway area, • embankment and • the reservoir area	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • Consolidation • Triaxial test
2.	Dam1	Functional		
3.	Dam2	Failed	• The breached section of the dam • The dam site • Initial part of the gulley in the reservoir	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
4.	Dam3	Functional		
5.	Dam4	Distressed		
6.	Dam5	Failed	• Soil samples were taken at the initial part of the gully in the reservoir area. • Soil sample was also taken at the section of the embankment where the gully cut across the embankment at the left abutment. • Soil sample was taken from the surrounding dam site for analysis.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
7.	Guyaku New	Under Construction		
8.	Jibiro New	Under Construction		
9.	Paka	Functional		
10.	Nzuzu	Failed	• Soil sample was taken at a section where the spillway failed.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability

S/N	Name	Status	Sampling Points	Tests on samples
				• Consolidation • Triaxial test
11.	Dam1	Distressed	• Soil sample was taken in the reservoir (silt characteristics). • Soil sample was also taken at the point where excessive erosion has taken place on the embankment	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
12.	Dam2	Failed	• Soil sample was taken in the reservoir	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits
13.	Nasarawo Dam3	Failed		
14.	Shakawa	Functional		
15.	Sebore	Functional		
16.	Musa Nyako	Functional		
17.	Ali Walga	Functional	• Soil sample was taken on the embankment where erosion sets in.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
18.	Salba	Functional		
19.	Sallau Gidao	Functional		
20.	Dam1	Distressed		
21.	Dam2	Failed		
22.	Dam3	Distressed	• Soil sample is taken from the reservoir.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits
23.	Dam4	Distressed	• Soil sample is taken from the reservoir. • Soil sample is also taken from the	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction

S/N	Name	Status	Sampling Points	Tests on samples
			embankment.	• CBR • Consolidation • Triaxial test
24.	Dam5	Functional		
25.	Kiri	Functional		
26.	Dadinkowa	Functional	• Soil sample was taken from the stockpiled leftover which was used for the core	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
27.	Cham	Failed	• Soil at the cracked crest. • At the landslide. • Dam body at section of breach. • At the toe leakage.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
28.	Bambam	Failed	• Soil sample was taken from the embankment	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
29.	Kaltingo	Under Construction		
30.	Pindiga I	Functional	• Soil sample was taken form the Reservoir/Embankment.	• Specific Gravity (Gs) • Sieve analysis • Permeability • Compaction • CBR • Consolidation • Triaxial test
31.	Pindiga II(Madagaska)	Functional	• Soil sample was taken from the Reservoir/Emb	• Specific Gravity (Gs) • Sieve analysis • Permeability • Compaction

S/N	Name	Status	Sampling Points	Tests on samples
			ankment.	• CBR • Consolidation • Triaxial test
32.	Bojude	Functional	• Soil sample was taken from the reservoir/embankment.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
33.	Jombo Dam Dukku	Functional	• The embankment/reservoir Soil sample was taken	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
34.	Dukku Dam (Kogin Dole)	Functional	• Soil sample was taken from the embankment.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
35.	Balanga	Functional		•
36.	Waya	Repeated Failure but Rehabilitated	• Soil from the borrow pit • Filter materials from shells	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
37.	Gubi	Functional	• Shell soil sample from the stockpiled left over was collected	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR

S/N	Name	Status	Sampling Points	Tests on samples
			• Core soil sample from the embankment was also collected	• Consolidation • Triaxial test
38.	Miri	Functional	• Soil sample was taken from the embankment and reservoir	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
39.	Marraba Ganye Toro Dam	Functional	• Soil sample was taken from the reservoir and embankment	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation • Triaxial test
40.	Tarangadi	Under Construction		
41.	Kufan Abba Rima	Under Construction		
42.	Dull Dam	Failed	• Soil samples were taken from the embankment/ reservoir and the spillway area.	• Specific Gravity (Gs) • Sieve analysis • Atterberg Limits • Permeability • Compaction • CBR • Consolidation

Blank under Sampling Points and Test on Samples indicate that Samples were not picked at some dams due to logistic reasons or non cooperation of owners.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Preliminary Investigations

Preliminary investigations on the selected dams are detailed in Table 4.1. Status of the dams in the study area and number of dams visited per State are presented in Figs 4.1 and 4.2 respectively.

Table 4.1; Dams visited with names, owner, location, embankment type and status

S/N	Name	Owner	Location	Type	Status
1.	Girei	UBRBDA	Girei, AD State	HE	Failed
2.	Dam1	Guyaku Grazing Reserve	Guyaku,AD State	HE	Functional
3.	Dam2	GuyakuGrazing Reserve	Guyaku, AD State	HE	Failed
4.	Dam3	Guyaku Grazing Reserve	Guyaku, AD State	HE	Functional
5.	Dam4	Guyaku Grazing Reserve	Guyaku, AD State	HE	Distressed
6.	Dam5	Guyaku Grazing Reserve	Guyaku, AD State	HE	Failed
7.	Guyaku New	Guyaku Grazing Reserve	Guyaku, AD State	HE	Under Construction
8.	Jibiro New	Guyaku Grazing Reserve	Jabbi Lamba, AD State	HE	Under Construction
9.	Paka	Maiha LGA	Maiha, AD State	HE	Functional
10.	Nzuzu	Garkida Community	Garkida, Gombi AD State	HE	Failed
11.	Dam1	Nasarawo/Gongoshi Grazing Reserve	M. Belwa LGA, AD State	HE	Distressed
12.	Dam2	Nasarawo/Gongoshi Grazing Reserve	M. Belwa LGA, AD State	HE	Failed
13.	Nasarawo Dam	Nasarawo/Gongoshi Grazing Reserve	M. Belwa LGA, AD State	HE	Failed
14.	Shakawa	Sebore Farms	M. Belwa LGA, AD State	HE	Functional
15.	Sebore	Sebore Farms	M. Belwa L.G.A,AD State	HE	Functional
16.	Musa Nyako	Musa Nyako	M. Belwa L.G.A,AD State	HE	Functional
17.	Ali Walga	Ali Walga	M. Belwa L.G.A,AD State	HE	Functional
18.	Salba	Salba Nig. Ltd	M. Belwa L.G.A,AD State	HE	Functional
19.	Sallau Gidao	Sallau Gidado	M. Belwa L.G.A,AD State	HE	Functional

S/N	Name	Owner	Location	Type	Status
20.	Dam1	Sarau/Belel Grazing Reserve	Maiha, AD State	HE	Distressed
21.	Dam2	Sarau/Belel Grazing Reserve	Maiha, AD State	HE	Failed
22.	Dam3	Sarau/Belel Grazing Reserve	Maiha, AD State	HE	Distressed
23.	Dam4	Sarau/Belel Grazing Reserve	Maiha, AD State	HE	Distressed
24.	Dam5	Sarau/Belel Grazing Reserve	Maiha, AD State	HE	Functional
25.	Kiri	UBRBDA	Kiri Shelleng, AD State	ZE	Functional
26	Dadinkowa	UBRBDA	Dadinkowa, Gombe State	ZE	Functional
27.	Cham	UBRBDA	Cham, Gombe State	ZE	Failed
28.	Bambam	UBRBDA (CP)	Bambam Balanga, Gombe State	HE	Failed
29.	Kaltingo	UBRBDA (CP)	Kaltingo, Gombe State	HE	Under Construction
30.	Pindiga I	UBRBDA(MDGS)	Pindiga, Gombe State	HE	Functional
31.	Pindiga II(Madagaska)	UBRBDA (CP)	Pindiga, Gombe State	HE	Functional
32	Bojude	UBRBDA (MDGS)	Bojude Kwami, Gombe State	HE	Functional
33.	Jumbo Dam Dukku	UBRRBDA(MDGS)	Jumbo Dukku, Gombe State	HE	Functional
34.	Dukku Dam (Kogin Dole)	UBRBDA	Dukku, Gombe State	HE	Functional
35.	Balanga	GSMWR	Balanga, Gombe State	ZE	Functional
36.	Waya	UBRBDA	Waya, Bauchi State	ZE	Repeated Failure but Rehabilitated
37.	Gubi	BSMWR	Gubi, Bauchi State	ZE	Functional
38.	Miri	UBRBDA (MDGS)	Miri, Bauchi Bauchi State	HE	Functional
39.	Marraba Ganye Toro	UBRBDA (MDGS)	Mararraba Ganye Toro, Bauchi State	HE	Functional

S/N	Name	Owner	Location	Type	Status
	Dam				
40.	Tarangadi	HJRBDA(CTPRJC)	Tarangadi Alkaleri, Bauchi State	HE	Under Construction
41.	Kufan Abba Rima	HJRBDA(CTPRJC)	Kufan Abba Rima Alkaleri, Bauchi State	HE	Under Construction
42.	Dull Dam	ADP/NFRA/MDGS	Dull Tafawa Balewa Bauchi State	HE	Failed

Key; HE = Homogeneous Embankment, ZE = Zoned Embankment, AD = Adamawa

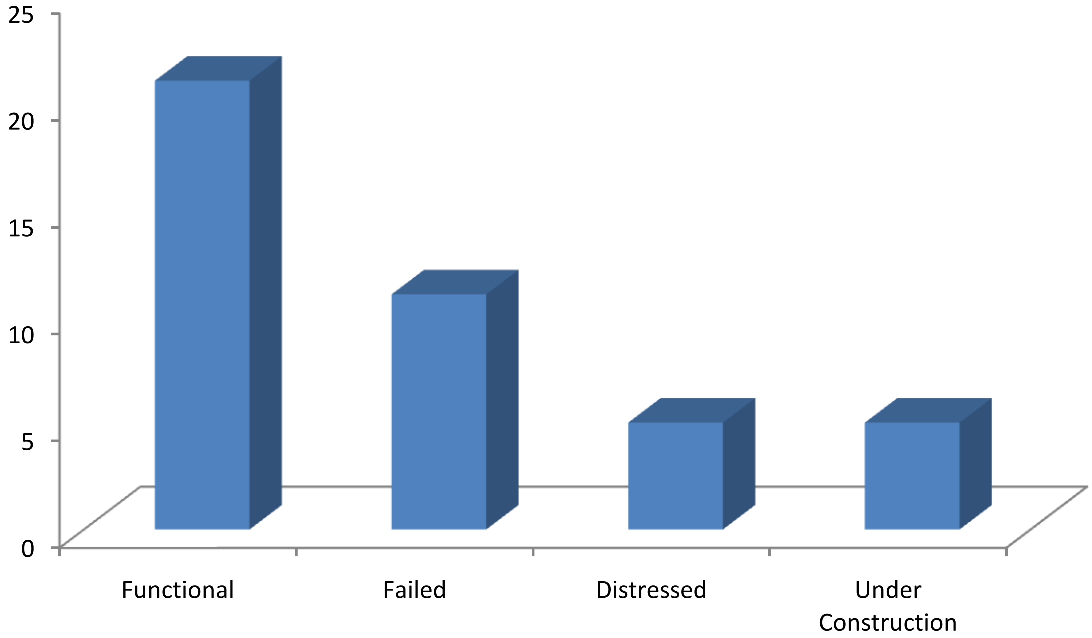


Fig 4.1;Status of the Dams in the study Area

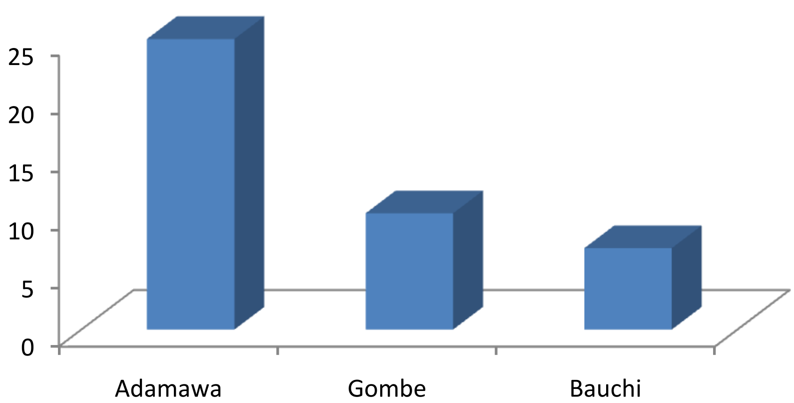


Fig 4.2; Number of Dams Visited per State

4.2 Design Information

When designing a building or other structure on land, it is important to take into consideration the structural properties of the ground that supports the project. Adequate knowledge of ground conditions is very essential for analysis, design and construction of geotechnical systems. Project delays, failures and cost over-run are the results of inadequate and inappropriate sub-soil investigations. Geotechnical investigation is an integral component of any civil engineering project. (Adejumo *et al*., 2012)

The study revealed that the geology of the study area is composed of Basement complex, Alluvium, Tertiary to recent Volcanics, Bima Sandstones, Yolde formations, Gombe sandstones, Pindiga formations, Younger granites and Kerikeri formations (Figure 3.3). The dams are located on different geological formations in the study area (Table 4.2).

In Nigeria, the basement complex consists of a wide variety of rock types which are classified into three broad groups:

• The first group is the older granites. These rocks account for most of the rugged relief and rocky landscape found in the Northeast.

• The second group of basement complex rocks consists of the quartzose metamorphic rocks, notably, the quartz schists and feldspathic quartz schists. Pegmatites are associated with these rocks in many occurrences. These rocks have undergone some weathering and usually, have a thin covering of stony soils also found in Northeast.

• The third group is the basic igneous and metamorphic rocks such as diorite, hornblende schists, biotite schists, and gneisses also found around the Northeast.(Lukman et al., 2011).

Table 4.2: Dams Location on Geological formations

S/N	Name	Location	Formation
1.	Girei	Girei, AD State	Alluvium
2.	Dam1	Guyaku,AD State	Basement Complex
3.	Dam2	Guyaku, AD State	Basement Complex
4.	Dam3	Guyaku, AD State	Basement Complex
5.	Dam4	Guyaku, AD State	Basement Complex
6.	Dam5	Guyaku, AD State	Basement Complex
7.	Guyaku New	Guyaku, AD State	Basement Complex
8.	Jibiro New	Jabbi Lamba, AD State	Alluvium
9.	Paka	Maiha, AD State	Tertiary to Recent Volcanics
10.	Nzuzu	Garkida, Gombi AD State	Basement Complex
11.	Dam1	M. Belwa LGA, AD State	Basement Complex
12.	Dam2	M. Belwa LGA, AD State	Basement Complex
13.	Nasarawo Dam	M. Belwa LGA, AD State	Basement Complex
14.	Shakawa	M. Belwa LGA, AD State	Basement Complex
15.	Sebore	M. Belwa L.G.A,AD State	Basement Complex
16.	Musa Nyako	M. Belwa L.G.A,AD State	Basement Complex
17.	Ali Walga	M. Belwa L.G.A,AD State	Basement Complex
18.	Salba	M. Belwa L.G.A,AD State	Basement Complex
19.	Sallau Gidao	M. Belwa L.G.A,AD State	Basement Complex
20.	Dam1	Maiha, AD State	Tertiary to Recent Volcanics
21.	Dam2	Maiha, AD State	Tertiary to Recent Volcanics
22.	Dam3	Maiha, AD State	Tertiary to Recent Volcanics
23.	Dam4	Maiha, AD State	Tertiary to Recent Volcanics
24.	Dam5	Maiha, AD State	Tertiary to Recent Volcanics
25.	Kiri	Kiri Shelleng, AD State	Bimma Sandstone and Yolde formations
26	Dadinkowa	Dadinkowa, Gombe State	Gombe Sandstone
27.	Cham	Cham, Gombe State	Tertiary to Recent Volcanics
28.	Bambam	Bambam Balanga, Gombe State	Tertiary to Recent Volcanics

S/N	Name	Location	Formation
29.	Kaltingo	Kaltingo, Gombe State	Bimma Sandstone and Yolde formations
30.	Pindiga I	Pindiga, Gombe State	Pindiga formation
31.	Pindiga II(Madagaska)	Pindiga, Gombe State	Pindiga formation
32	Bojude	Bojude Kwami, Gombe State	Keri- keri formation
33.	Jumbo Dam Dukku	Jumbo Dukku, Gombe State	Keri- keri formation
34.	Dukku Dam (Kogin Dole)	Dukku, Gombe State	Keri- keri formation
35.	Balanga	Balanga, Gombe State	Bimma Sandstone and Yolde formations
36.	Waya	Waya, Bauchi State	Basement Complex
37.	Gubi	Gubi, Bauchi State	Basement Complex
38.	Miri	Miri, Bauchi Bauchi State	Basement Complex
39.	Marraba Ganye Toro Dam	Mararraba Ganye Toro, Bauchi State	Younger granites
40.	Tarangadi	Tarangadi Alkaleri, Bauchi State	Keri- keri formation
41.	Kufan Abba Rima	Kufan Abba Rima Alkaleri, Bauchi State	Keri- keri formation
42.	Dull Dam	Dull Tafawa Balewa Bauchi State	Younger granites

Key; HE = Homogeneous Embankment, ZE = Zoned Embankment.

All these classes of the basement complex posses the stability as well as the water tightness for sound foundation of dams and their reservoirs in the study area. Of the 19 dams sited on the Basement complex, 10 were functional, 6 failed, 2 were distressed and 1 was under construction.

Basement complex rocks are subdivided into migmatite-gneiss complexes; the older metasedi- ments; the younger metasediments; the older gran-ites; and the younger granite alkaline ring complex-es and volcanic rocks. The migmatite gneiss complex is the commonest rock type in the Nigerian Basement complex. It comprises two main types of gneisses: the biotite gneiss and the banded gneiss. Very widespread, the biotitic gneisses are normally fine-grained with strong foliation caused by the parallel arrangement of alternating dark and light minerals. These gave them the strength, stability as well as imperviousness to support dams and reservoirs.

The banded gneisses show alternating light-coloured and dark bands and exhibit intricate folding of their bands. The migmatite gneiss complex is the oldest basement rock, and is believed to be of sedimentary origin but was later profoundly altered into metamorphic and granite conditions. The older metasediments were also among the earliest rocks to form on the Nigerian Basement Complex. Initially of sedimentary origin, with a more extensive distribution, the older metasediments underwent prolonged, repeated metamorphism; and now occur as quaitzites (ancient sand- stones), marble (ancient limestones), and other calcareous and relics of highly altered clayey sediments and igneous rocks (Adefila, 1975). These characteristics of the basement complex gave it the ability to be strong as a foundation material that can support dams without any risk of failure and also the impermeability to retain water in the reservoirs of the dams.

The basement complex rock areas are mainly granitic in composition and in different stages of metamorphism, either as gneisses, migmatites, schists. There are older and younger granite. These rocks are hard, with low permeability and generally not water bearing. Most of the area covered by this formation fall within the semi-arid part of the country; where surface water is either seasonal or nonexistent. Most crystalline rock areas are located in areas of high relief. As a result run off is high and infiltration rates very low (Offodile, 1992). These behaviors encourage reservoir performance and stability of dams in the study area.

The oldest rock formation is the basement complex rocks which, as already indicated, is a crystalline and poor water yielding formation. The rocks underline most areas of Bauchi, Adamawa and Sardauna provinces. The rocks are mainly gneiss, and quartzite. Schist, marbles and calsilicates make up the metasedimentary areas (Thompson, 1956). Therefore giving the formation the ability to support dams and retain the water in the reservoir. In a study of Basement complex rocks employing secondary resistivity parameters in Northeastern Nigeria, Solomon and Samaila (2011) conclude that the third geologic layer indicates a higher conduction zone along the eastern parts which consist probably of the fine grained materials/weathered materials. Intermediate conductance striking from the north to the south constitutes a horizon with increased weathered materials, while the lower conductive value is underlined by fractured bedrock area. Contour values of the transverse resistance horizon increase from the west towards the east. Majority of the porosity contour levels fall within the range reasonable for weathered bedrock aquifer, however some of the resistivity derived porosities are slightly lower, which depict high concentration of clay matrix in the aquifer zone.

The Bima Sandstone, found in parts of Gombe, consist of essentially feldspathic sandstones, grits, pebble beds and clays. It is highly crystalline and cemented. Under this condition it presents the hydrogeological characteristics of Basement complex rocks. Secondary permeability is only developed by means of fracturing, weathering and solution (Okafo, 1982). It is generally a good foundation material due to its poor permeability. Of the 3 dams sited on the Bima sandstone, 2 are functional and one is under construction.

Also according to Okafo, (1982) the Yolde formation overlies the Bima sandstone and consist of about 152 m of thinly bedded sandstone, followed by alternating mudstones. The formation underlies the two sedimentary sub-basins of upper Benue found in Gombe and Numan. It is a weaker load bearing formation than the Bima formation.

Pindiga formation, the Dukkul, Jessu, Sekule and Numanha formations are found in the Gombe basin. These formations which overlay the Yolde formation within the basin, consist of black shells, limestone's and a number of inter bed sands (Reyment, 1965), hence giving it stability and relative impermeability to support earth dams and their reservoirs.

The overlaying Kerri-Kerri formation is a sequence of fine grained sandstones, clays, silts, with some thin coal bands. The lithology changes rather rapidly, both vertically and laterally (Okafo, 1982). Due to its looseness and coarseness, the Kerri-Kerri formation is stable with good bearing capacity. Sixty percent of the dams on Kerikeri formation are fuctional while the remaining 40% are under construction.

Gombe sandstone consists of a series of brownish well-bedded fine to medium grained sandstones, sandy and silty micaceous shales and mudstone. It occupies much of the highland areas marking the western area of Gombe and its surrounding countryside overlies the prindiga formation (Okafo, 1982). The formation is generally impervious to some extent and fairly stable under loading. The dominant argillaceous materials further reduce the permeability considerably. All the dams on Gombe sandstones and Pindiga formations are functional.

Of the dams sited on the Basement complex, 61, 27 and 11% were functional, failed and distressed respectively. The only completed dams sited on Alluvium has failed. Seventy-five percent of the dams found on Tertiary to recent Volcanics have either failed or distressed, while the remaining 25% are functional. About 80% of the dams located on Bima sandstone and Yolde formations are functional while the remaining 20% are under construction. All the dams on Gombe sandstones and Pindiga formations are functional. Sixty percent of the dams on Kerikeri formation are fuctional while the remaining 40% are under construction. For the dams on Younger granites; 50% failure and 50% functionality were recorded.

4.3 Hydrometeorology of the Dam Sites

The Nigerian climate is controlled by latitude pressure belts which generate the south west and north east trade winds. The zone of convergence of these two trade winds is sometimes described as the Inter Tropical Discontinuity (ITD). The pressure belts sweep across the country in an almost north-south direction and shift the ITD along. The location of the ITD at any one time determines the climate at that region as pointed out by Adefolula (1986). Appendix II (Tables A1 to A12) gives the weather of the study area.

4.3.1 Rainfall (mm)

The main characteristics of rainfall in the study area are its seasonal nature and its variability from year to year. Similar studies (Ishaku et al., 2010, 2011 and 2013) in different parts of Nigeria show different patterns and variability. Rainfall is determined by the movement of the intertropical convergence and all is derived from the monsoon air masses. Change and variability in rainfall are important determinants of the need for dam construction, reason for construction and period of construction. This also suggest likely flooding seasons and therefore the risk of failure or distressnes when the reservoirs are being threatened by excess water from floods resulting from heavy downpours.

Table 4.3 highlights the period of failures and distressnes of the dams. Most of the dam failures and distress occur during peak rainfall months as a result of flooding, erosion, siltation and overtopping, while a few happen during peak dry season months due to excessive dryness coupled with high evaporation losses.

Tables 4.4 - 4.6 show statistical summary of rainfall in Adamawa, Bauchi and Gombe over a period of 1982 – 2010, obtained from Upper Benue River Basin Development Authority, Yola (UBRDA) and Nigerian Meteorological Agency, Abuja (NIMET). It was observed that rainfall season of the study area sets in properly in March and ends in October/November each year. The peak of the season occurs between the months of July and September. The dry season sets in properly in October/November to February every year. These results agree with Offodile (1990) Adebayo and Umar (1999) and Lukman *et al.* (2011),

Most of the dams studied are small dams that exploit run off from surrounding hills and seasonal streams in the reservoir catchment. Construction of earth dams in the study area helps to conserve excess water that is obtained during peak rainy season to be utilized during lean periods. The short period of rainy season calls for proper harnessing of the resource by damming. The long dry periods allows for larger periods of convenient construction on the sites. Construction during peak seasons is very difficult and expensive. Most of the dam failures occur during the period of peak rainfall as a result of flooding by heavy downpours. The status of the dams were affected by peak rainy (Monthly total of 327.1 to 478 mm) season, where most (75%) of the failures and distresses happened due to erosion, siltation and subsequent flooding among other reasons.

Table 4.3; Period of Failures and Distresses

S/N	Name	Month of Incidence	Type	Status
1.	Girei	April	HE	Failed
3.	Dam2	March	HE	Failed
5.	Dam4	August	HE	Distressed
6.	Dam5	September	HE	Failed
10.	Nzuzu	September	HE	Failed
11.	Dam1	April	HE	Distressed
12.	Dam2	August	HE	Failed
13.	Nasarawo Dam	September	HE	Failed
20.	Dam1	September	HE	Distressed
21.	Dam2	August	HE	Failed
22.	Dam3	September	HE	Distressed
23.	Dam4	September	HE	Distressed
27.	Cham	August	ZE	Failed
28.	Bambam	August	HE	Failed
36.	Waya	September	ZE	Repeated Failure but Rehabilitated
42.	Dull Dam	August	HE	Failed

4.3.2 Evaporation (mm)

Evaporation in the study area is generally high. A close examination of the statistical summary of the evaporation in the study area as presented by Tables 4.4 to 4.6 shows values of evaporation between 1982 and 2010 being higher than rainfall values.

As expected from the climate, mean daily evaporation values are slightly high for the dry season months (October – February) with the highest values occurring within the month of February to April. This is when the influence of the moisture laden south- westerlies is greatest.

This scenario results in high losses of water from the reservoirs through evaporation and can contribute to dam failures around the catchments due to absence of water in the reservoir. During the dry season months when evaporation quantities are highest (Monthly total of 354.6 to 409.7 mm), coupled with relatively high temperatures (39^0C - 43^0C), the 20% of the failures were attributed to loss of reservoir water through evaporation among others. This agrees with Ishaku and Maji (2010).

Table 4.4; Statistical summary of monthly total rainfall (mm) in Yola over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(mm)	-	-	19.0	36.8	112.5	128.8	244.2	202.3	182.4	59	7.4	-
Maximum(mm)	3.9	-	54.0	89.3	217.0	246.9	1991.1	437.8	355.2	192.6	15.4	-
Minimum(mm)	3.9	-	1.7	0.3	34.6	21.2	93.6	83.3	78.7	5.7	0.3	-
Median(mm)	3.9	-	14.9	37.6	115.3	115.1	193.4	199.7	183.2	49.9	6.95	-
SD	-	-	18.69	23.75	48.19	49.47	34.50	64.07	68.22	47.31	7.34	-
Skewness	-	-	1.15	0.58	0.27	0.65	5.15	1.44	0.57	1.68	0.15	-
Kurtosis	-	-	1.05	-0.09	-0.75	0.44	27.31	5.88	0.13	2.80	-4.56	-
Variance	-	-	349.66	564.19	2322.80	2447.39	115941.2	4106.17	4654.77	2238.55	53.95	-

Table 4.5; Statistical summary of monthly total rainfall (mm) in Bauchi over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(mm)	-	0.25	10.06	33.07	87.48	156.24	217.88	266.92	163.20	25.20	-	-
Maximum(mm)	-	0.5	33.2	202.0	168.5	340.3	396.3	478.8	265.5	57.4	-	-
Minimum(mm)	0	0	0	0	27.3	77.8	77.6	31.3	31.2	1.7	0	0
Median(mm)	-	0.25	6.2	18.7	95	129.5	181.2	272.5	163.2	20.3	0	0
Kurtosis	-	-	3.29	9.51	-0.74	1.10	-0.86	0.57	1.16	-0.55	-	-
Skewness	-	-	1.767	2.957	0.164	1.066	0.541	-0.342	-0.395	0.525	-	-
SD	-	-	12.198	48.139	39.499	64.721	93.453	101.709	53.197	15.362	-	-
Variance	-	0.125	148.810	2317.40	1560.17	4188.82	8733.62	10344.9	2829.99	235.991	-	-

Table 4.6; Statistical summary of monthly total rainfall (mm) in Dadinkowa(Gombe) over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(mm)	-	-	6.3	30.7	93.8	121.9	190.9	202.0	152.7	48.6	6.8	-
Maximum(mm)	0.0	0.0	12.3	101.7	229.7	299.0	327.1	319.0	303.9	153.6	12.5	0.0
Minimum(mm)	0.0	0.0	1.0	0.0	11.2	28.7	61.9	82.2	47.2	0.0	1.0	0.0
Median(mm)	-	-	4.3	27.0	93.9	117.1	182.4	192.7	149.9	44.1	6.8	-
Kurtosis	-	-	-3.112	1.745	0.327	1.558	-0.576	-0.979	-0.344	1.570	-	-
Skewness	-	-	0.399	1.213	0.639	1.127	-0.004	0.101	0.409	1.344	-	-
SD	-	-	5.563	24.442	51.103	59.872	65.002	62.601	63.039	37.333	8.131	-
Variance	-	-	30.953	597.443	2611.51	3584.67	4225.26	3918.95	3973.93	1393.78	66.12	-

Table 4.7; Statistical summary of monthly total evaporation (mm) in Yola over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
Average(mm)	206.62	243.59	293.61	267.38	211.38	154.61	136.47	118.35	126.10	150.92	181.76	195.94	2216.73
Kurtosis	15.15	5.55	1.22	0.58	0.44	2.57	9.66	3.76	0.25	0.37	2.17	0.98	7.83
Maximum(mm)	250.47	354.64	339.41	335.6	298.8	233.78	255.8	195.92	177.8	177.93	218.45	241.69	2492.05
Median(mm)	214.2	246.955	299.3	270.21	208.125	150.21	135.4	115.16	126.47	153.61	182.82	198.175	2314.69
Minimum(mm)	22.9	167.9	220.32	181.92	146.43	107.44	86.14	79.42	91.2	107.8	125.36	146.45	944.43
Skewness	-3.601	0.621	-1.091	-0.211	0.774	0.825	2.442	1.339	0.426	-0.945	-1.140	-0.655	-2.615
SD	42.705	33.285	29.737	35.244	38.895	26.083	30.519	23.769	20.863	18.743	21.612	22.285	332.347
Variance	1823.8	1107.9	884.3	1242.1	1512.8	680.3	931.4	564.9	435.2	351.3	467.1	496.6	110455.1

Table 4.8; Statistical summary of monthly total evaporation (mm) in Bauchi over the period of 1980 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(mm)	285.7	360.0	403.3	340.3	227.2	146.2	85.7	70.5	90.6	171.5	279.8	261.9
Maximum(mm)	438.0	471.0	477.0	459.0	348.0	207.0	123.0	93.0	114.0	258.0	522.0	315.0
Minimum(mm)	0	276	336	249	114	99	0	27	69	102	150	0
Median(mm)	288.0	354.0	402.0	318.0	216.0	147.0	90.0	69.0	90.0	171.0	273.0	270.0
SD	66.33	46.24	42.13	56.52	49.29	24.05	21.20	13.88	11.29	32.57	68.71	54.77
Skewness	-2.362	0.128	0.181	0.651	0.437	0.383	-1.948	-0.699	0.096	0.528	2.186	-3.814
Kurtosis	11.93	-0.222	-1.277	-0.549	0.445	0.246	8.487	1.722	-0.605	0.863	6.896	18.109
Variance	4400.	2138.40	1774.66	3194.86	2429.38	578.38	449.26	192.52	127.45	1060.85	4721.38	2999.51

Table 4.9; Statistical summary of monthly total evaporation (mm) in Dadinkowa(Gombe) over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(mm)	226.6	257.5	314.3	303.1	517.6	227.1	201.9	180.6	244.2	164.1	210.0	210.2
Maximum(mm)	295.2	295.4	409.7	404.0	3740.0	334.6	333.7	395.5	1995.6	233.5	329.6	291.7
Minimum(mm)	151.6	179.0	200.8	191.6	182.1	158.3	108.8	115.6	129.1	62.0	115.9	161.3
Median(mm)	231.0	267.6	316.0	310.8	305.9	212.9	189.1	165.8	163.3	162.2	208.5	215.3
Kurtosis	0.166	1.03	-0.05	-0.05	13.76	0.01	1.42	5.73	24.52	1.06	1.64	0.87
Skewness	-0.347	-1.192	-0.394	-0.203	3.654	0.759	1.044	2.096	4.932	-0.544	0.425	0.463
SD	34.966	30.153	56.319	51.759	810.986	45.296	48.608	61.487	366.417	39.863	47.198	33.322
Variance	1222.67	909.22	3171.93	2679.06	657699.	2051.80	2362.74	3780.68	134261.	1589.13	2227.71	1110.42

4.3.3 Temperature (^oC)

The temperature in the study area is relatively high. It is shown by the recording from Yola, Bauchi and Dadinkowa (Gombe) as shown in Tables 4.10 to 4.12. The mean daily maximum ranges from 39^{o}C to 40^{o}C in March and April in Bauchi, and 43^{o}C in March and April in Yola. The minimum for the study area is recorded in Bauchi with 25^{o}C - 31^{o}C in December and January.

These high temperatures encourages loss of water from the reservoirs through evaporation. The situation it difficult for reservoirs to conserve water into the drier seasons. This also contributes in the failure of some dams in the study area as can be seen from the later part of this work.

4.3.4 Relative humidity (%)

Relative humidity (RH) can be simply defined as the amount of water in the air relative to the saturation amount the air can hold at a given temperature. Mean monthly relative humidity is generally low with no month experiencing values greater than 90%. (Tables 4.13 to 4.15). As expected, the mean monthly relative humidity values are slightly high for the wet season months (June to October) with the highest values occurring during the months of June to September. The maximum for the study area recorded in Yola is in the range of 82 - 90% in the months of June and July, while the minimum for the same station was 14 - 15% in February and March. Bauchi has a maximum range of 87 - 88% in August to September with a minimum of 15 - 18% in February and March. The maximum for Gombe was recorded in the range of 80 - 81% in August to September and minimum of 15% in February to March. This phenomenon is controlled by the influence of the moisture-laden south westerlies and moisture deficient north-easterlies. Offodile (1990) reported a similar trend of events for the entire country. This phenomenon again, subject the reservoirs to water losses especially during the dry season; when the recharging rains and seasonal streams contribute little or nothing to the reservoir storage. This, and low Relative Humidity, (14%- 18%), coupled with high temperatures and high evaporation rates contributed to dam failures and distresses in the study area.

In summary, the status of the dams were affected by two opposite scenarios, first during peak (Monthly total of 478.8 mm- 327.1 mm) rainy season (August to September), where most (75%) of the failures and distresses happened due to siltation and subsequent flooding among other reasons. Second; during the dry season months (October to February) when evaporation quantities are highest (Monthly total of 354.6 mm- 409.7 mm), coupled with relatively high temperatures (39^0C - 43^0C), where 20% of the failures were attributed to loss of reservoir water through evaporation among others. The low Relative Humidity, (14%- 18%) during the dry season further intensifies the loss of water through evaporation.

Table 4.10; Statistical summary of monthly mean maximum temperature (^oC) in Yola over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(^oC)	33	36	38	39	36	33	31	30	31	32	34	33
Maximum(^oC)	39	41	43	43	41	36	37	34	35	37	38	38
Minimum(^oC)	30	29	32	33	30	28	29	27	27	29	28	29
Median(^oC)	34	37	39	40	36	33.5	32	31	31	33	35	34.5
Kurtosis	-1.33	1.34	0.74	-0.26	-0.05	3.15	1.38	1.17	0.88	0.87	1.30	0.07
Skewness	0.146	-1.175	-0.727	-0.716	-0.162	-1.329	0.996	-0.125	0.084	-0.280	-1.022	-0.577
SD	2.707	2.785	2.610	2.737	2.731	1.624	1.852	1.411	1.759	1.667	2.300	2.096
Variance	7.332	7.758	6.812	7.495	7.463	2.638	3.432	1.992	3.096	2.780	5.290	4.396

Table 4.11; Statistical summary of monthly mean maximum temperature (°C) in Bauchi over the period of 1980 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average	30.8	33.7	36.8	37.7	35.8	32.9	30.6	29.6	30.8	32.7	33.3	31.4
Maximum	34	38.1	38.5	40	38.4	34.9	33	32.2	33.2	34	34.7	33.7
Minimum	24.9	30	34.5	29	32.2	29	28.8	28	29.8	31	30.8	28.9
Median	30.9	33.6	36.7	38.0	35.9	32.7	30.3	29.6	30.6	32.9	33.3	31.4
SD	2.23	1.96	0.98	1.84	1.28	1.17	1.03	0.86	0.78	0.78	1.03	1.25
Skewness	-0.743	0.001	-0.340	-3.659	-0.374	-0.815	0.527	0.652	1.312	-0.803	-0.663	-0.069
Kurtosis	0.121	-0.320	0.013	17.618	1.137	3.017	0.076	1.385	1.996	0.158	0.018	-0.895
Variance	4.968	3.827	0.953	3.385	1.634	1.377	1.060	0.746	0.613	0.611	1.055	1.558

Table 4.12; Statistical summary of monthly mean maximum temperature (^oC) in Dadinkowa over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(^oC)	33	34	38	40	37	34	32	31	32	34	35	34
Maximum(^oC)	37	39	42	43	41	36	34	33	34	37	38	38
Minimum(^oC)	26	28	33	35	33	30	28	27	28	30	29	25
Median(^oC)	33	35	38	40	38	35	32	31	33	35	36	35
Kurtosis	0.034	2.040	0.039	0.270	-0.407	2.837	0.996	1.486	2.838	1.019	3.275	6.757
Skewness	-0.706	-0.741	-0.208	-0.686	-0.436	-1.421	-0.896	-1.069	-1.393	-0.861	-1.727	-1.962
SD	2.863	2.333	2.349	1.978	2.000	1.396	1.468	1.412	1.343	1.562	2.206	2.553
Variance	8.198	5.443	5.519	3.913	4.000	1.951	2.156	1.993	1.804	2.440	4.867	6.5199

Table 4.13; Statistical summary of monthly relative humidity (%) in Yola over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(%)	30	26	32	45	58	68	73	75	77	67	40	32
Maximum(%)	45	44	76	88	79	90	82	86	87	80	52	53
Minimum(%)	18	15	14	27	46	48	58	37	63	48	18	25
Median(%)	28	23.5	30.5	44.5	60	68	75	79	78	69	39	30.5
Mode(%)	27	21	33	47	49	68	76	79	77	70	39	26
SD	7.123	8.532	13.459	14.089	8.861	8.240	5.452	9.637	5.409	8.510	8.480	7.033
Skewness	0.30	0.84	1.33	1.23	0.30	0.06	-1.11	-2.80	-1.13	-1.01	-0.76	1.43
Variance	50.74	72.80	181.14	198.52	78.53	67.91	29.72	92.88	29.25	72.42	71.92	49.47
Kurtosis	-0.509	-0.427	2.914	2.264	-0.303	1.920	1.499	10.056	1.632	0.568	0.729	2.237

Table 4.14; Statistical summary of monthly relative humidity (%) in Bauchi over the period of 1981 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average	32.2	27.2	27.8	42.7	59.0	69.1	76.3	79.0	73.6	58.6	37.8	34.3
Maximum	70.3	51.9	57.4	66.5	75.3	82.8	85.7	88.0	86.6	76.6	69.5	63.3
Minimum	21.4	15.6	18.0	20.6	41.2	59.5	69.5	66.4	35.7	32.2	29.2	25.7
Median	28.6	24.6	24.5	42.3	59.0	69.5	76.5	79.6	74.9	58.6	35.8	32.6
SD	9.66	8.56	9.47	9.21	7.72	5.78	3.38	4.12	8.12	9.49	8.49	7.89
Skewness	2.527	1.587	1.720	0.291	-0.137	0.526	0.181	-0.637	-3.559	-0.190	2.246	2.054
Kurtosis	7.729	2.431	2.958	1.203	0.348	0.079	1.096	2.600	17.070	1.465	6.041	5.368
Variance	93.405	73.323	89.686	84.891	59.663	33.435	11.458	16.983	65.973	90.141	72.014	62.327

Table 4.15; Statistical summary of monthly relative humidity (%) in Dadinkowa(Gombe) over the period of 1982 to 2010

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Average(%)	29	25	27	40	55	61	68	73	73	64	40	32
Maximum(%)	52	51	45	55	73	72	77	81	80	74	55	53
Minimum(%)	20	15	15	20	40	48	56	65	58	47	26	19
Median(%)	28	25	27	42	58	62	70	75	74	64	38	31
Kurtosis	4.58	3.59	0.19	-0.47	-0.96	-0.42	0.08	-0.53	4.09	0.85	-0.37	1.08
Skewness	1.744	1.494	0.795	-0.329	0.155	-0.382	-0.667	-0.447	-1.553	-0.738	0.4719	0.827
SD	7.008	8.200	8.091	10.035	9.817	6.855	5.204	4.146	4.762	6.565	7.757	7.832
Variance	49.114	67.252	65.465	100.71	96.383	46.996	27.090	17.194	22.683	43.110	60.183	61.350

4.4 Engineering Factors of Failures and Distresses

Statistcal Package for the Social Sciences (SPSS) was used to analyse the data in this section and the results are presented using bar charts and tables. Most of the dams in the study area are small with few medium ones and fewer large dams according to dam heights (Fig. 4.3). This also agrees with the reservoir capacities of the sample dams as shown in Fig. 4.4. Most of the dams have reservoir capacity less than 1 million m^3 indicating that they are small while very few have capacities greater than 1 million m^3 indicating that they are large. In most cases small dams do not require more stringent measures regarding their stability analysis and other engineering performances that may be the reason why most engineers and contractors do haphazard work when it comes to dealing with small dams which resulted into their frequent failure. Of the dams investigated, 27% were found to have failed, 12% were distressed, 12% were uncompleted and 49% are functional. Most of the failed and distressed dams are small dams. Engineering estimates and standards must be adhered to in executing dam projects irrespective of the sizes of the dams. Ethics of engineering profession should be strictly applied and enforced in order to reduce the failures to the barest minimum.

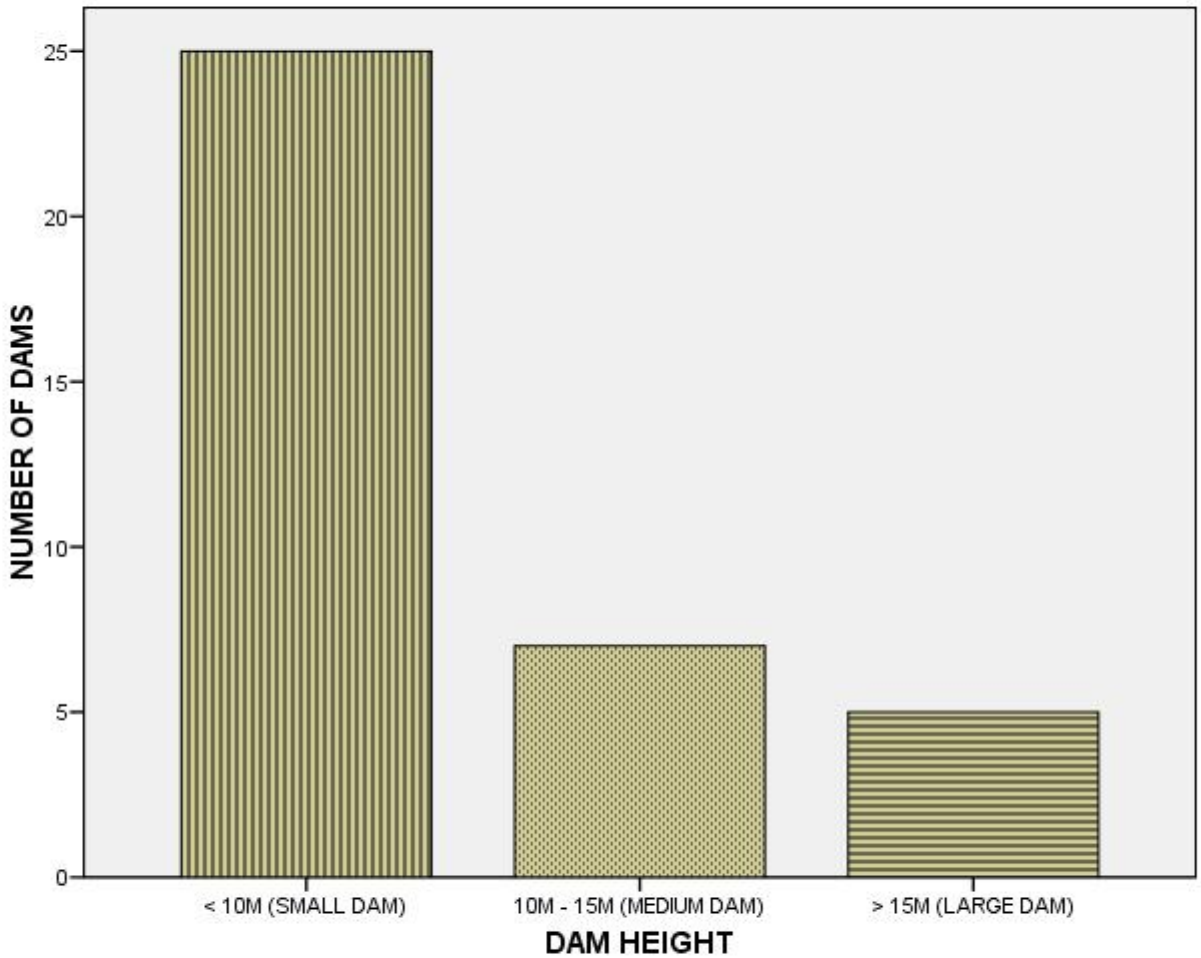


Fig 4.3; Heights of the dams

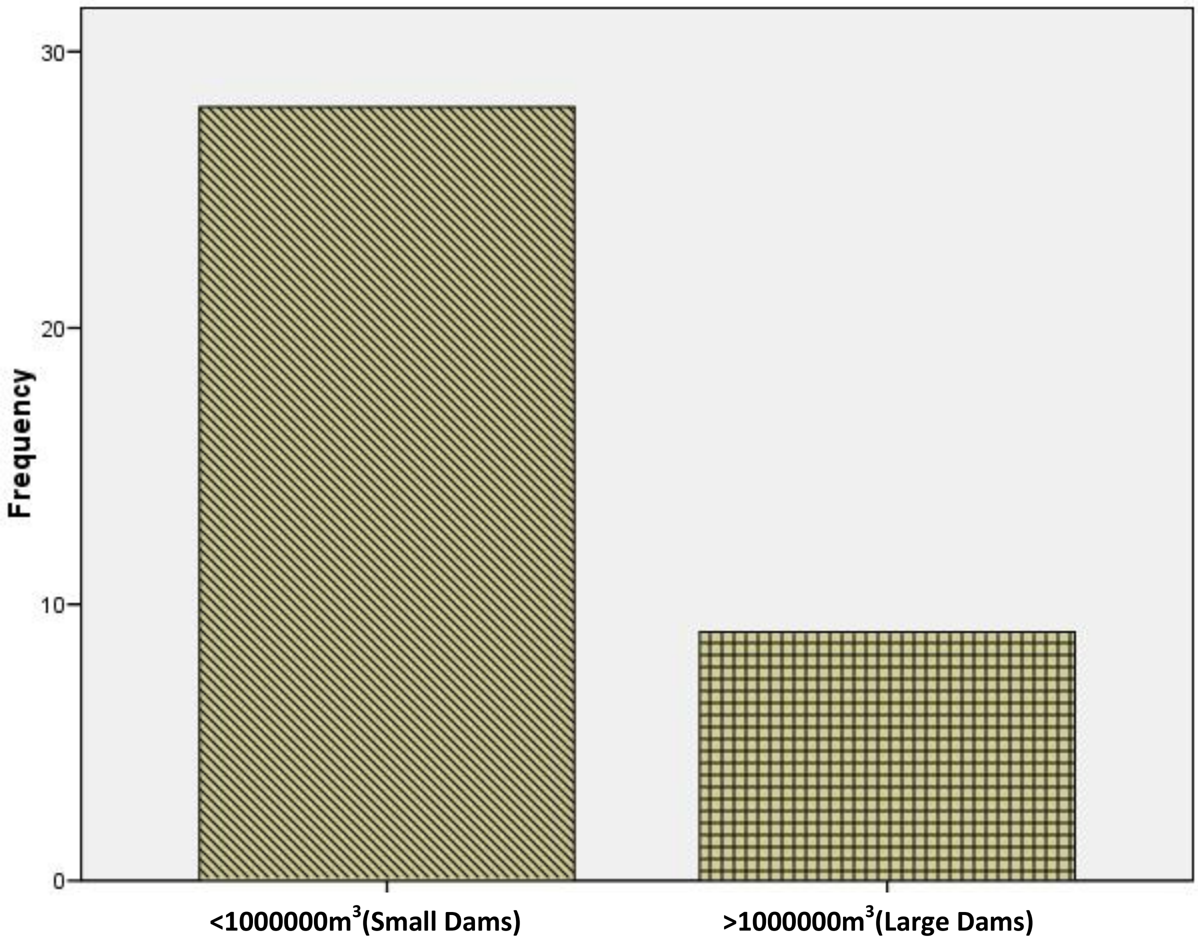


Fig 4.4; Reservoir capacities of the dams

4.4.1 Mode of Failure

The failure modes include; seepage (5%), piping (8%), structural (1%), hydraulic (50%) and a combination of two or more modes in a complex manner. Of the failures recorded, a combination of the hydraulic, seepage and structural modes accounted for 36% among others. The failure modes often interact in a very complex manner whereby at times, a seepage failure may result to piping and the dam may give way hydraulically leading to a structural failure. The failure mode of most of the dams is a combination of seepage, hydraulic and structural failure (Figure 4.5)

4.4.2 Causes of Failures and Distresses

The main causes of failures of the dams in the study area are attributed to (i) poor construction (ii) poor design and construction (iii) poor maintenance (iv) poor feasibility studies, design and construction. Inadequate maintenance is the main cause of failure with (71%). Others are lapses in design (9%) and poor construction (15%) among others. (Figure 4.6)

Embankment type also seems to have influence on the failure, distresses and performance of earth dams. Almost all the dams that failed have homogeneous embankments with very few zoned embankments among them. The rate of failure on homogeneous embankments (90%) was compared to zoned embankments (10%). All the distressed dams are of homogeneous embankment type. Of the functional dams majority are of homogeneous type with few zoned embankment type (Figure 4.7)

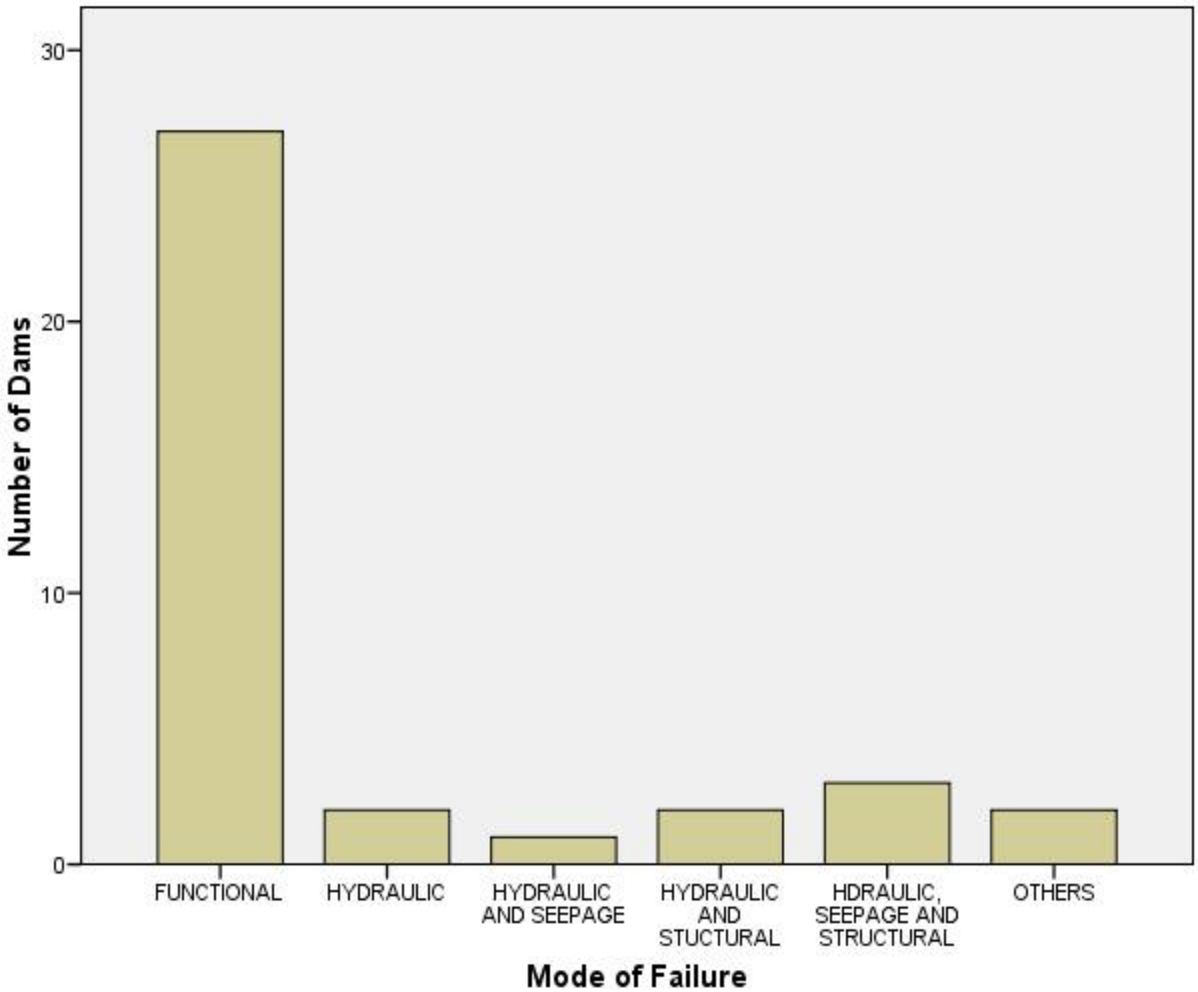


Fig 4.5; Modes of failure

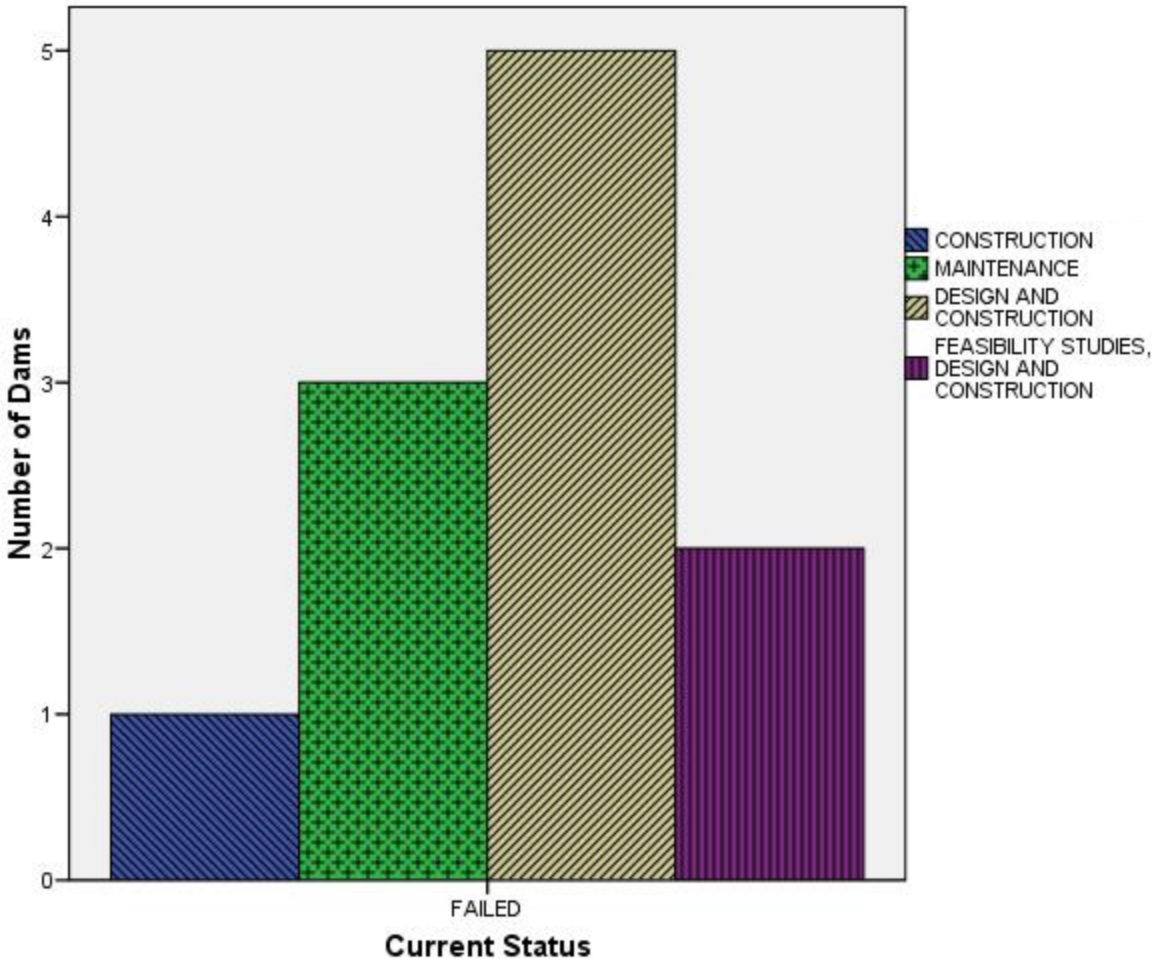


Fig 4.6; Main causes of failures

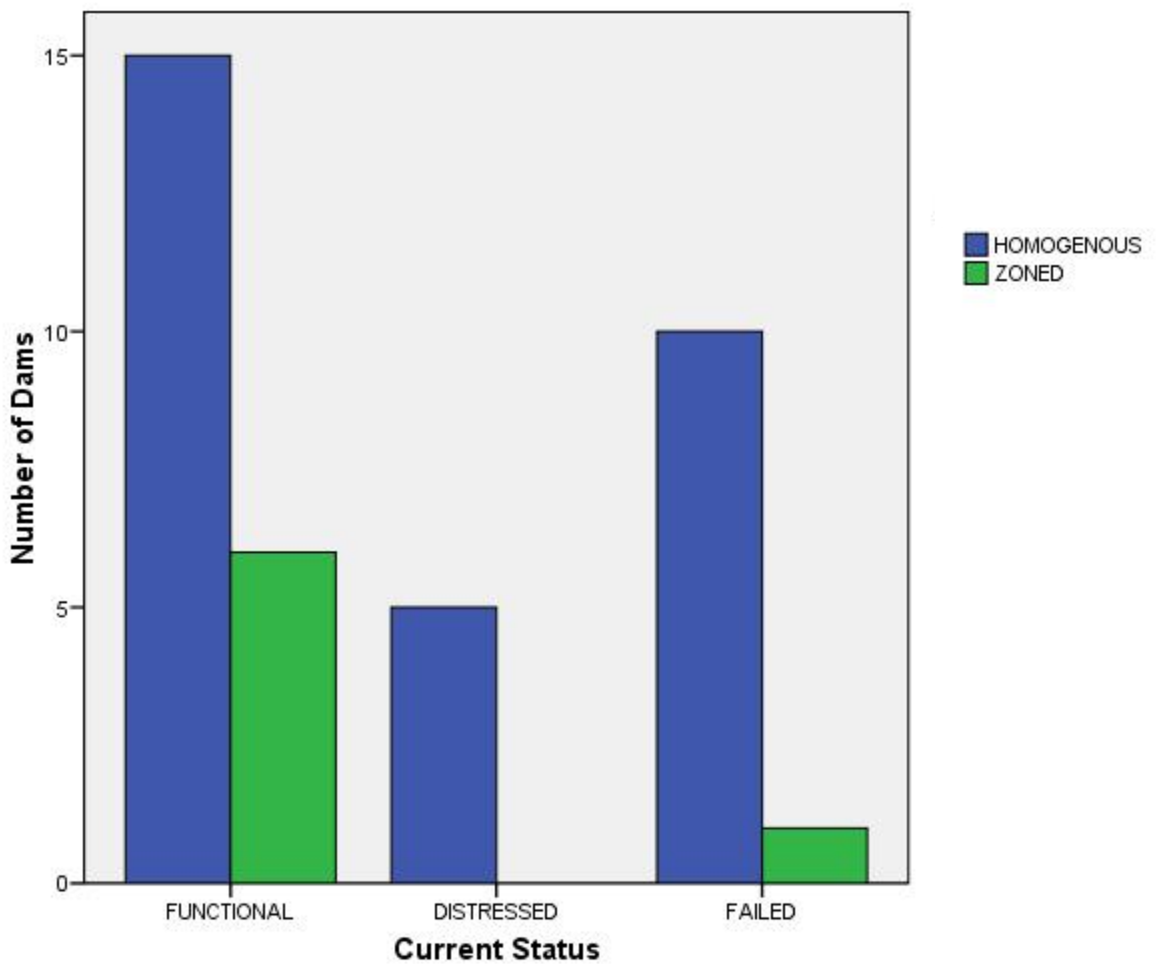


Fig 4.7; Effects of embankment type on status of dams

The causes of failure can also be associated with the modes of construction. Those dams constructed using direct labor seems to fail the most, followed by those dams constructed using contractors and only very few fail using both direct labor and contract mode of construction. Most of the functional dams were contracted out and only few of them were constructed using direct labor by the organizations or owners of the dams. Most of the distressed dams were constructed using direct labor and only very few of these dams were constructed using contractors (Figure 4.8).

Embankment maintenance has some influence on failures, distresses and performance of earth dams in the study area. Most of the dams that failed were not maintained and very few were maintained. The distressed dams show a similar trend with very few dams. The functional dams were better maintained (Figure 4.9).

A clearer picture is obtained when embankment maintenance schedule is being related to failure, distresses or functionality of the dams. Of the dams that failed, most were not maintained at all. Where they were maintained, the exercise was not regular. Only very few were maintained regularly. None of the distressed dams were being maintained regularly. Majority of the functional dams were not maintained regularly, few were maintained regularly and some were not maintained at all. Regular maintenance increases the chance of functionality. (Figure 4.10).

The Presence of safety instrumentation in form of piezometers, stilling wells and basins seem to have influence over functionality, distressnes and performance of earth dams in the study area. Most of the dams that fail have no safety instrumentation in place. Only few of them have. All the distressed dams do not have safety instrumentation in place. Majority of the functional dams also have no safety instrumentations in place but some of them have (Figure 4.11).

Conditions of safety instrumentation (such as piezometers, observation wells, stilling basins) also play some role on the status of the dams. Of the failed dams, most of them have no safety instrumentation, only very few have safety instrumentations which are damaged, functional or a mixture of both. Again, of the functional dams most of them do not have safety instrumentations and few have safety instrumentation that are either damaged or a mixture of both damaged and functional ones on the same embankment. Very few have damaged safety instrumentations in place (Figure 4.12).

The presence of dam safety monitoring teams also has some influence on the functionality, distresses and failure of earth dams. Majority of the failed dams do not have dam safety and monitoring teams at site while very few of them do. A similar trend was observed for distressed dams. Dam safety and monitoring teams were not present in most of the functional dams (Figure 4.13). Table 4.16 summarizes the site visits, inspection and justification of sampling points for soils.

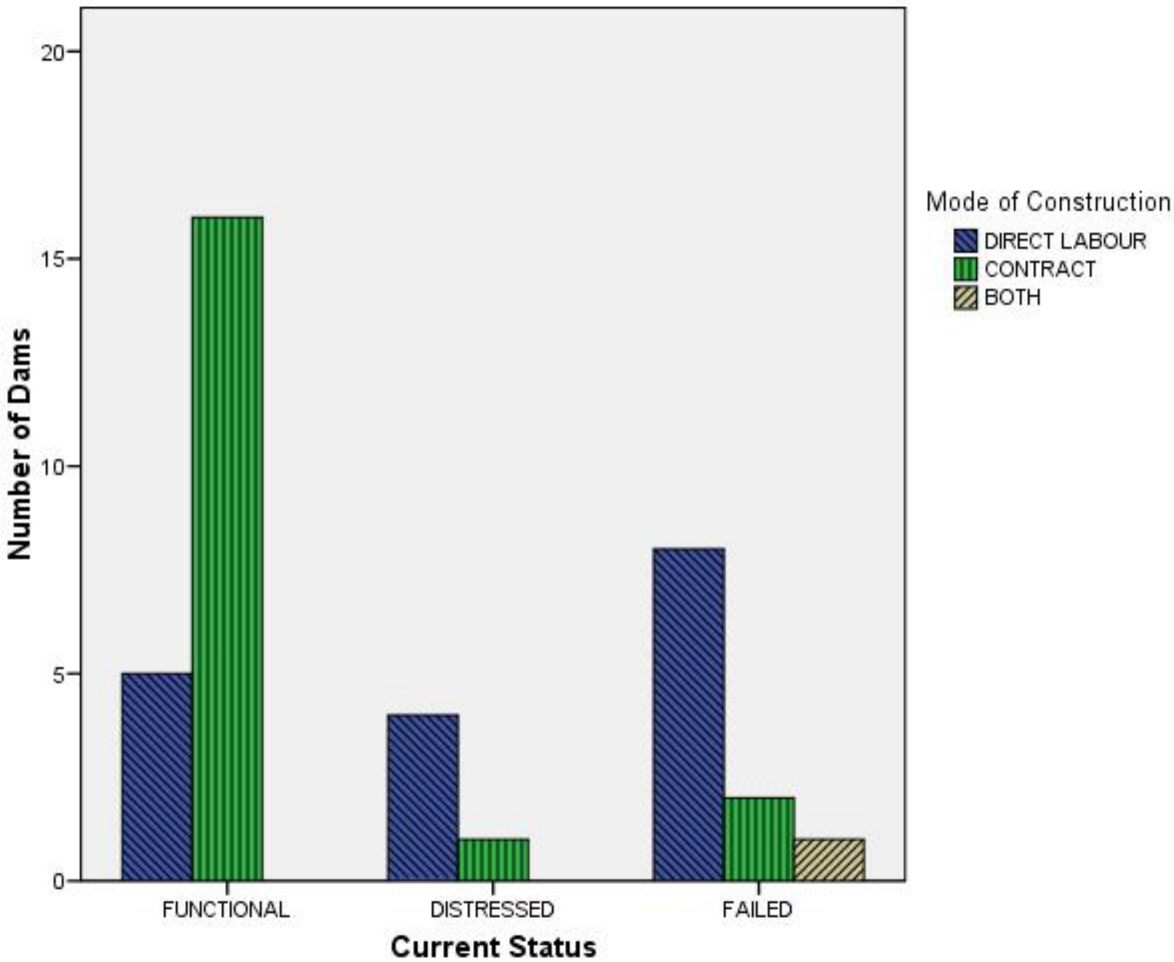


Fig 4.8; Effects of mode of construction on status of dams

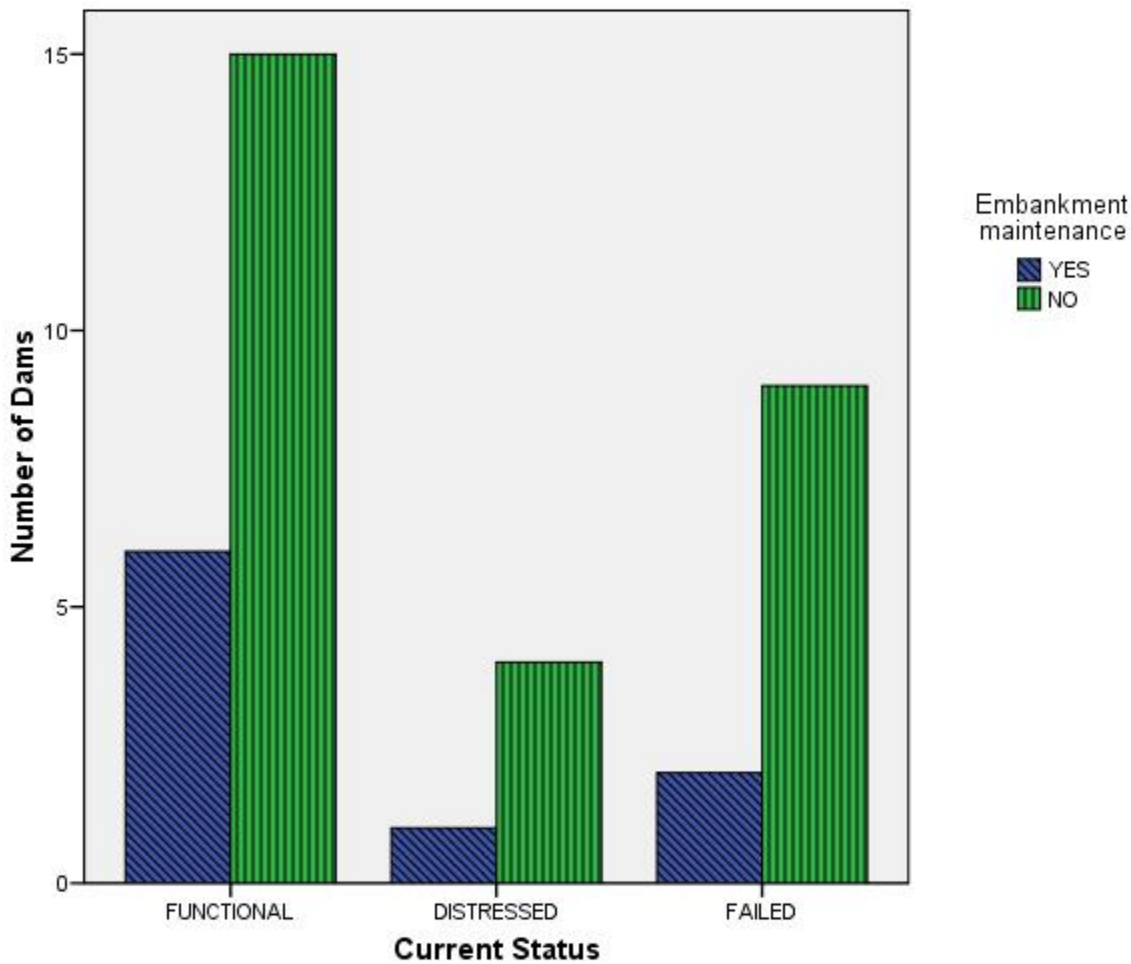


Fig 4.9; Effects of embankment maintenance on status of dams

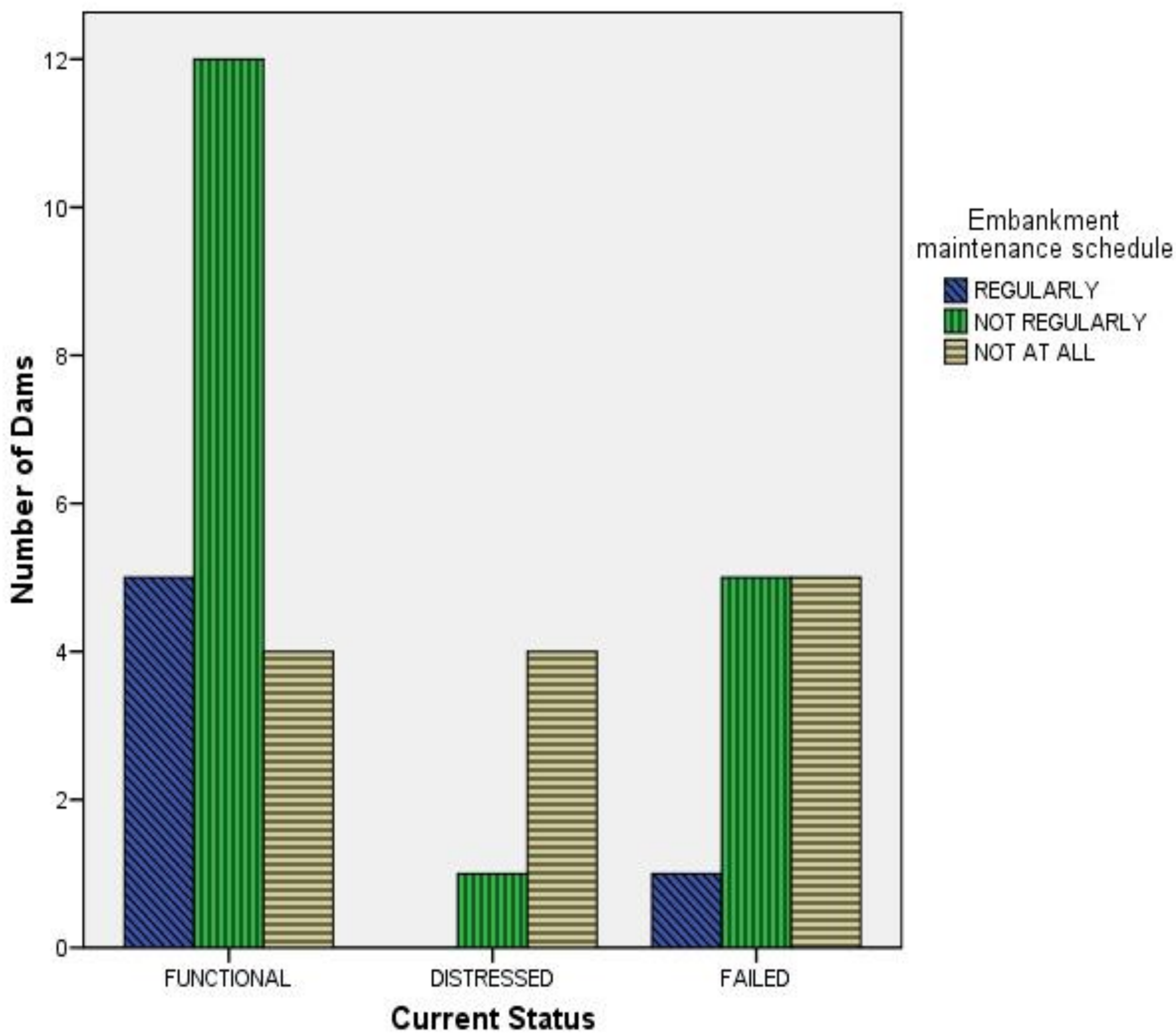


Fig 4.10; Effects of maintenance schedule on status of dams

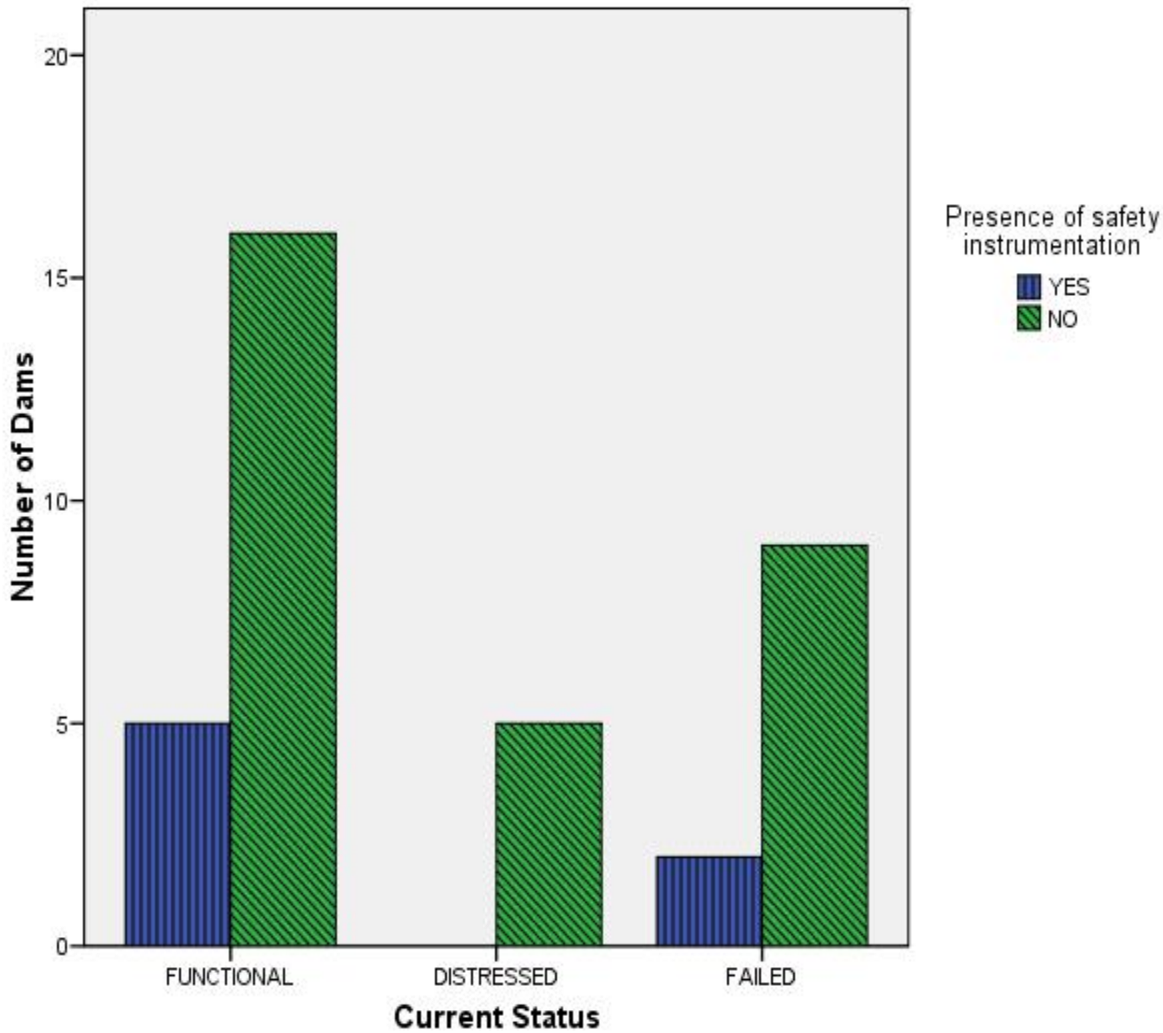


Fig 4.11; Effects of presence of dam safety instrumentations on status

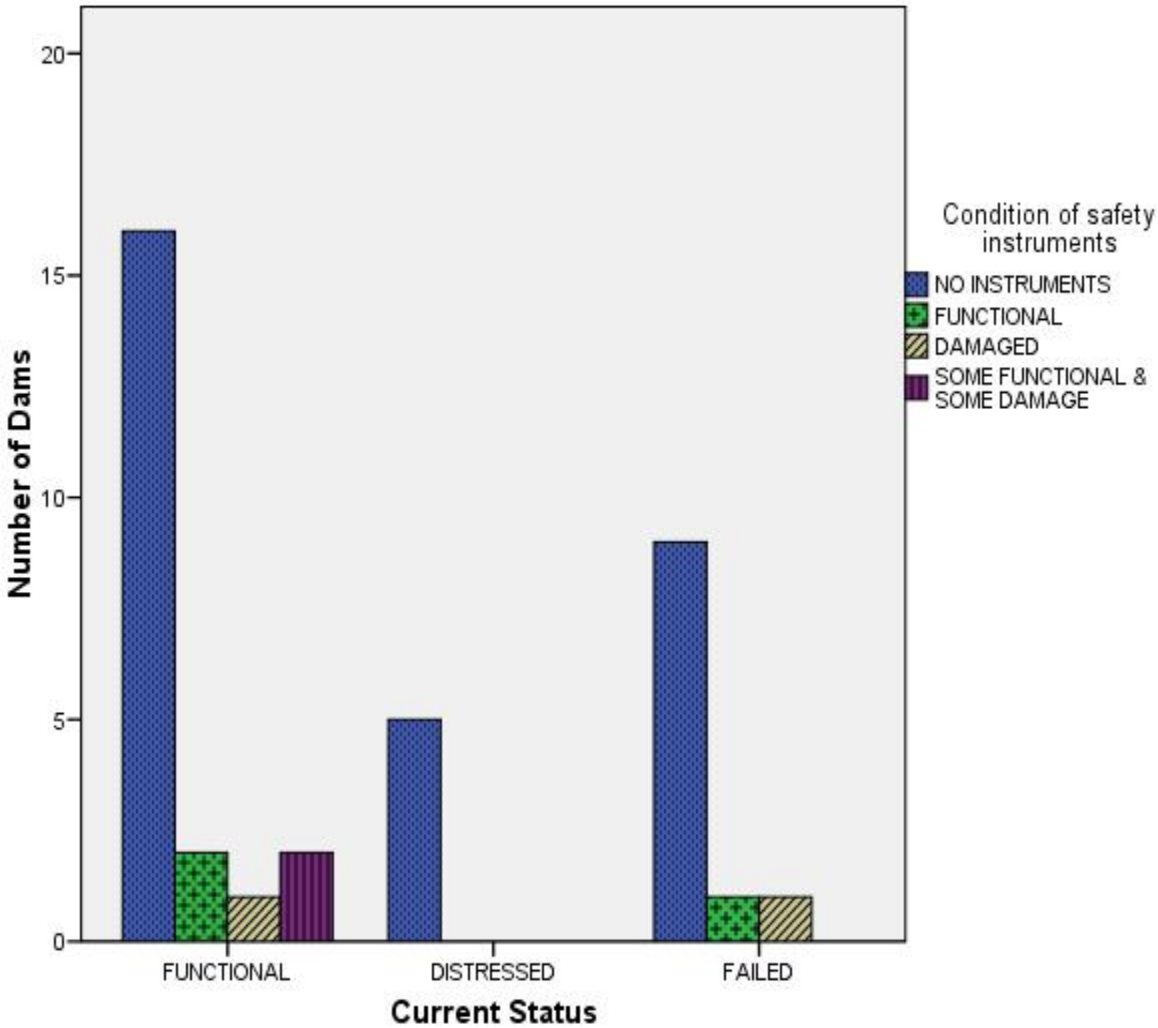


Fig 4.12; Effects of conditions of safety instrumentations on status

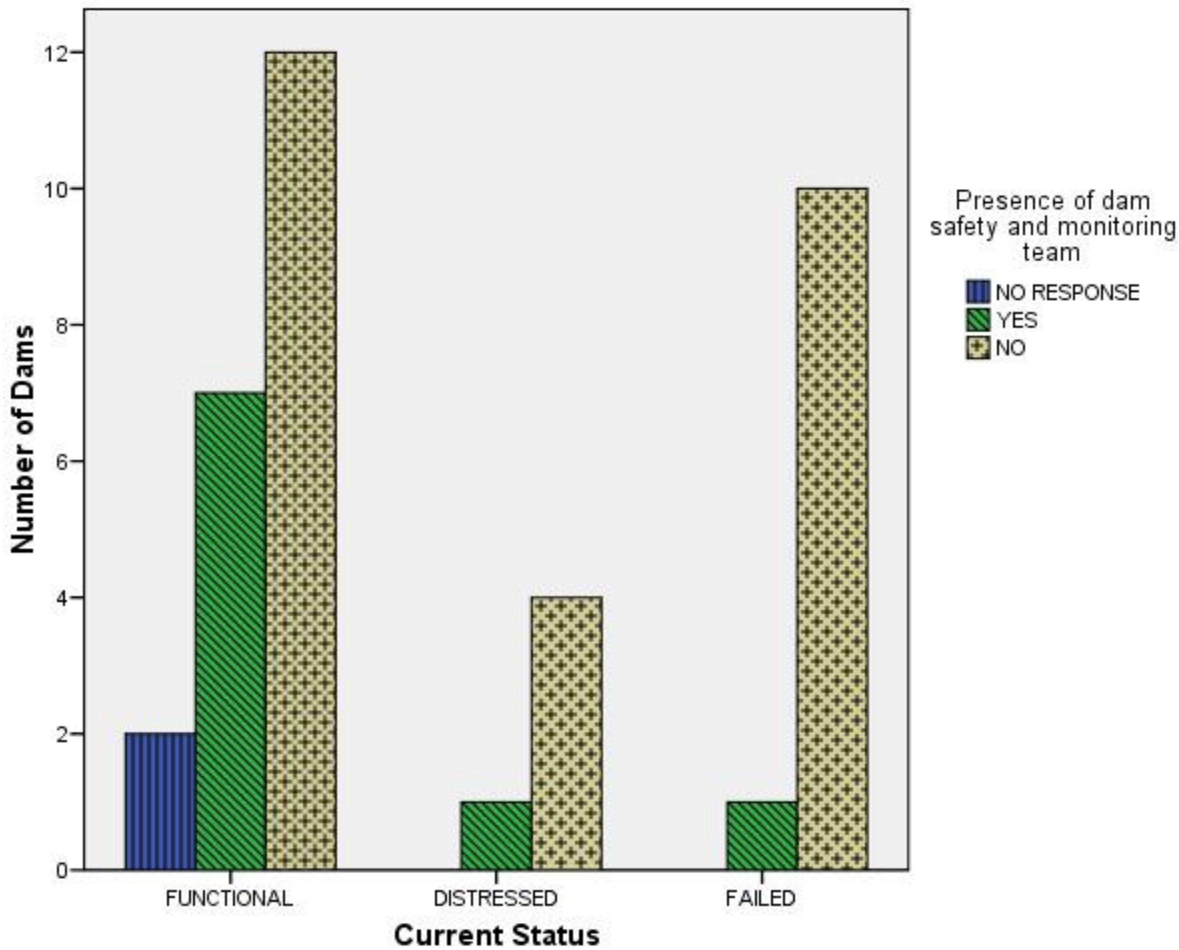


Fig 4.13; Effects of presence of dam safety monitoring teams on

Table 4.16; Summary of Site Visits and Inspections of Dams

S/N	Name	Current Status	Remarks
1.	Girei	Failed	• Weak clayey Soils around the spillway area manifested in ultimate Spillway failure. • Excessive wetness along the embankment at the toe of the dam. • The excavation for the foundation of the spillway does not reach stable grounds, hence the observed seepage water in the stilling basin. • Loose silty sands were observed as predominant soils in the reservoir area. Hence the low water retention capacity of the reservoir. • The reservoir is completely dry during the second site visit just after two months from the first visit when the reservoir was almost full and water was about to spill. • There is termite infestation on the crest along the embankment.
2.	Dam1	Functional	• There is no definite spillway, when the reservoir is filled up the water spill backwards over the lowest part of the left abutment. • There is general lack of maintenance of the dam. • Dead woods are observed in the reservoir. • The dam is still functional.
3.	Dam2	Failed	• There is general lack of maintenance of the dam. • An eye witness account has it that, the dam was in serious threat of failure for a long time; but nothing was done about it. • The dam finally gave way when part of the embankment was washed away as a result of serious erosion and ultimate overtopping. • There is no definite spillway in place. • A Gully cut the dam axis at the right abutment below original ground level. • Loose unstable soils were observed at the dam site.
4.	Dam3	Functional	• There is a general lack of maintenance of the Embankment and the reservoir. • Animals are left to trample on the embankment. • There is no Slope protection of any kind in place. • The spillway is made of unlined compacted heavy soil with outcrops of rocks in it. • There is evidence of erosion on the spillway channel. • Siltation of the reservoir is in progress.
5.	Dam4	Distressed	• There is general lack of maintenance of the dam. • The reservoir is progressively silting up. • The dam is still functional with some water in the reservoir.

S/N	Name	Current Status	Remarks
6.	Dam5	Failed	• Eye witness account has it that the dam failed from the initial impoundment. • A deep gully was observed to start from the reservoir and continues downstream cutting the embankment beyond original ground level. • There is no definite spillway in place for the dam's reservoir. • Deep wide cracks were observed on the underlining soils around the surrounding area where the dam was constructed. • The reservoir seems to be too close to the surrounding hills where high runoff velocities are very much expected. • The dam was never utilized and hence maintenance was not affected for any reason.
7.	Guyaku New	Under Construction	• The dam seems to be well designed. • The reservoir has already started impounding water. • Construction difficulty is observed as water from the reservoir has destroyed part of the intake and release facility. • The embankment near the intake and release facility has to be breached in order to allow excess water to pass downstream. • Rain erosion is observed on the embankment and slopes.
8.	Jibiro New	Under Construction	• Still under construction but has started impounding water in the reservoir. • Slope protection on the upstream but none on the downstream side. • A spillway in place. • Homogeneous embankment with different materials along the length of the embankment. • Animals graze around the reservoir catchment and at times trample on the embankment. • For now there is no siltation reducing measure in place.
9.	Paka	Functional	• The dam is still functional; containing water in the reservoir throughout the dry season. • The embankment is poorly maintained. • The dam is built round to intercept runoff from the surrounding hills and mountains. • The reservoir is used for both animal and human consumption directly without any silt trap. • No definite spillway in place. • No slope protection at both upstream and downstream sides of the dam.

S/N	Name	Current Status	Remarks
10.	Nzuzu	Failed	• A small dam constructed for flood protection of downstream Garkida Township and animals watering. • There is general lack of maintenance of the dam. • Animals also stampede the embankment. • There is no slope protection of any kind for the dam. • A stone pitched spillway gave way after a heavy storm; the flood breached the embankment below original ground level, retracing the original stream downstream. • The impounded water in the reservoir escaped through the eroded spillway, flooding some parts of Garkida town.
11.	Dam1	Distressed	• Generally poorly maintained. • Excessive erosion on the embankment near the right abutment from crest level. • Embankment height was increased sometime ago to compensate for embankment settlement and reservoir siltation. • The reservoir is heavily silted. • Still functional and has water in the reservoir.
12.	Dam2	Failed	• The reservoir silted up. • Erosion has destroyed the embankment due to lack of spillway and maintenance. • Generally poorly maintained. • No slope protection in place. • The dam has completely failed.
13.	Nasarawo Dam3	Failed	• Siltation of the reservoir. • Overtopping of the embankment. • Embankment broken and erosion cuts in inform of a big gully. • Manifestation of cracks on the embankment. • Due to difficult terrain the dam could not be accessed for physical examination and photographing. • No access road to the dam to facilitate maintenance.
14.	Shakawa	Functional	• Siltation of the reservoir is in progress. • Embankment heavily infested with termites • Due to shortages of water recently, pipes were laid to recharge the reservoir from Mayobelwa River. • A broad crested wear is in place as the spillway. • A stilling basin is constructed downstream the spillway.
15.	Sebore	Functional	• Siltation of the reservoir is progressing. • The reservoir storage capacity of $1.4m^3$ was not reached last

S/N	Name	Current Status	Remarks
			year due to water shortage in the rainy season. • Still functional the reservoir is having water. • Slope protection in form of rock riprap and stone pitched concrete is in place. • A spillway is in place.
16.	Musa Nyako	Functional	• No definite spillway in place. • No slope protection of any kind in place both at upstream and downstream sides. • The dam is poorly maintained. • The dam is still functional and still contains water in the reservoir.
17.	Ali Walga	Functional	• Siltation of the reservoir • Slope protection in the form of hand placed rock riprap with grasses as binders. • Embankment cracks. • Termite infestation on the embankment • Contains water throughout the dry season. • Vegetal overgrowth in the reservoir. • A spillway is provided for the reservoir of the dam.
18.	Salba	Functional	• Well constructed with slope protection in place. • A definite spillway in place. • A reservoir that is functional.
19.	Sallau Gidao	Functional	• Still under construction. • Started impounding water in the reservoir. • Irrigation is performed by pumping water from the reservoir through siphon tubes over the embankment. • There is slope protection with rock ripraps on the upstream while there is none on the downstream side. • The spillway is still under construction. • Cracks are noticed on embankment crest.
20.	Dam1	Distressed	• The dam is still functional; the reservoir contains water the year round. • Charging is through runoff from the surrounding hills and mountains. • Poorly maintained. • No slope protection of any kind in place. • Siltation of the reservoir is in progress as only small amount of water is left before rainy season.

S/N	Name	Current Status	Remarks
			• There is no definite spillway in place. • Animals graze around and on top of embankment.
21.	Dam2	Failed	• There is no slope protection. • As at the time of the site visit the seasonal stream does not flow into the reservoir; it is diverted over time to flow behind the dam due to lack of maintenance. • The reservoir is completely empty. • Originally runoff from the surrounding hills and mountains charges the reservoir. • There is virtually no access road to the dam site to facilitate maintenance. • The dam has virtually failed hydraulically. • No appropriate spillway at site.
22.	Dam3	Distressed	• The dam is heavily silted, poorly maintained, and generally unattended to. • The reservoir capacity is generally reduced with siltation. • Termite moulds infest the embankment. • Lack of access road to the dam to facilitate maintenance.
23.	Dam4	Distressed	• Still functional and contains water in the reservoir all year round. • Generally poorly maintained. • Erosion is taking place on the embankment at different points. • Animals trampling on the embankment also aggravate the erosion process. • Runoff from the surrounding hills and mountains recharge the reservoir. • There is no definite spillway at site. • Animals graze around and on top of embankment.
24.	Dam5	Functional	• The dam is hurriedly and poorly constructed. • The reservoir contains some water in it. • Due to lack of access road and difficult terrain, the dam could not be accessed for physical inspection.
25.	Kiri	Functional	• Well constructed • Well maintained • The Crest is well surfaced with asphalt and rock chippings • Reservoir water is highly underutilized • Gabions on the crest to prevent erosion • Instrumentations are working

S/N	Name	Current Status	Remarks
26.	Dadinkowa	Functional	• The dam is stable and functioning • Reservoir is functioning very well, with a lot of water being impounded • Small vegetal growth seen on the embankment • Recent maintenance work sealed up the cracks and removed the termite moulds on the embankment • The dam is highly underutilized
27.	Cham	Failed	• Seepage and piping at downstream dam toe • Crest crack • Land slide at the right abutment • The in complete construction of the spillway • Overtopping and near overtopping • Embankment Settlement
28.	Bambam	Failed	• There is no good dam site (no good abutments) • The foundation seems to be of black cotton soil which is expansive clay. • The embankment was not well compacted. • The contractor was hurriedly paid off even when the Job was not completed. • The design crest height was not achieved when the reservoir overtopped the embankment and the dam failed hydraulically and mechanically.
29.	Kaltingo	Under Construction	• The consultant drawings and estimates are grossly inadequate, for the dam to be successful. • It seems the consultant did not visit the site before submitting their report.. • Fairly good materials were on site, soil tests were found to be unnecessary and the site Engineers initiative is paramount. • The dam citing is politically motivated. • Presently lack of funds is hampering progress of work at site.
30.	Pindiga I	Functional	• The reservoir contains water for the whole year round. • Vegetative cover is used to protect the embankment. • Runoff water collects to charge the reservoir. • Spillway is functional but is been threatened by erosion at the downstream channel. • The embankment seem to be well compacted.
31.	Pindiga II(Madagaska)	Functional	• The reservoir contains water for the whole year round. • No slope protections on both upstream and downstream slopes,

S/N	Name	Current Status	Remarks
			but the community have started planting a type of shrub on the embankment slopes. • Runoff water collects to charge the reservoir. • Spillway does not start spilling as the reservoir did not fill up since first impoundment. • The embankment seem to be well compacted. • The borrow pit is at the dam site.
32.	Bojude	Functional	1. The reservoir contains water throughout the year. 2. The spillway has never spilled. 3. There is no slope protection at both the upstream and downstream slopes. 4. Erosion is threatening both the upstream and downstream slopes of the embankment. 5. The reservoir receives water from runoff of the surrounding area which is a bit flat. 6. The soil material excavated from the reservoir is used for the construction of the embankment. 7. There is poor compaction.
33.	Jombo Dam Dukku	Functional	• Embankment not protected for erosion and drawdown at downstream and upstream slopes. • The soil from the excavated reservoir was used for the embankment construction. • The reservoir and the embankment were protected by using local wooden fence. • There is some form of community effort in maintaining the dam. • The reservoir contains water throughout the year.
34.	Dukku Dam (Kogin Dole)	Functional	• The dam is stable and contains water in the reservoir. • There is serious erosion on the embankment, as no slope protection of any kind is in place. • Trees are left to grow on the embankment. • The reservoir does not contain water the year round due to over withdrawal as a result of high water demand. • Animals and people drink directly from the reservoir. • There is some form of community effort in maintaining the dam.
35.	Balanga	Functional	• The dam is sited in a rocky terrain • The reservoir contains water and the spillway (broad crested orgy weir) is spilling water. • Trees and shrubs are growing on the embankment.

S/N	Name	Current Status	Remarks
			• Termite infestation is noticed on the embankment. • Most of the dam body contains rocks with relatively small earth-fill embankment. • There is slope protection of ripraps on both the upstream and downstream shells of the dam. • Can't access material of embankment or burrow pit for sample examination.
36.	Waya	Repeated Failure but Rehabilitated	• Cracks on the crest • Slope failures at downstream shell • Failure of the flood wall • Spillway incapacitated • Seepage/piping through the embankment • Complete breach of the embankment
37.	Gubi	Functional	• The reservoir is functioning very well and water is retained all year round. • The embankment has arrangement for drainage of water. • Slope protection in form of rock riprap, barns, concrete masonry while some portions have no slope protection of any kind in place. • There are some small shrubs on the embankment. • Erosion has started setting. • There is termite and ant infestation on the embankment. • The spillway is a broad crested weir and is spilling as at the time of visit.
38.	Miri	Functional	• The embankment is having no slope protection at both the upstream and downstream slopes • The reservoir stores water throughout the year • The site is good but the job was poorly done • The spillway is out of level, not properly sited. • The job was hurriedly done
39.	Marraba Ganye Toro Dam	Functional	• The embankment is having slope protection of grasses on both the upstream, downstream slopes and the crest. • The reservoir does not store water throughout the year • The site is good but the job was poorly done • The spillway is spilling water as at the time of site visit.
40.	Tarangadi	Under Construction	• The dam is under construction. • Upgrading of a borrow pit to include an embankment and a spillway.

S/N	Name	Current Status	Remarks
			• More of a rehabilitation and improvement works. • The consultants / contractors documents did not tally with what obtains on ground at the site. There is complete lack of harmony between what is on their documents and the physical dam site; even the contours could not be traced. • The design drawings, estimates and BOQ are not applicable to the conditions on the site.
41.	Kufan Abba Rima	Under Construction	• The new dam is under construction. • Work is progressing with a lot of difficulties due to inconsistencies between the contract document and the site conditions. • The consultants / contractors documents did not tally with what obtains on ground at the site. There is complete lack of harmony between what is on their documents and the physical dam site; even the contours could not be traced. • The design drawings, estimates and BOQ are not applicable to the conditions on the site.
42.	Dull Dam	Failed	• The embankment is stable but is been threatened by erosion. • There are no upstream and downstream slope protections of any kind. • The embankment is made of two materials (Clayey and lateritic) separated by the spillway. • The spillway is stone pitched with concrete, without a good foundation and is poorly bound to the embankment. • There is no good abutment at both ends of the embankment. • The reservoir held the first impounded water until when the spillway failed. • The spillway was poorly designed and constructed. • As at the time if visit the stream is passing freely across the embankment through the spillway with no water in the reservoir.

4.5 Soil Properties of the Dams

The influence of soil properties on failure, distress and functionality of earth dams in the study area are discussed using the soil test results obtained from the laboratory experiments. Appendix III Tables B1 to B8 summarize the soil test results. Also Appendix IV Tables C1 to C9 gives the criteria for interpretation of the results from the fundamentals. This section tries to show how the engineering properties of soil influence failure, distress or functionality of earth dams in the study area.

4.5.1 California Bearing Ratio (CBR)

The CBR, which is a measure of bearing capacity of materials under dynamic or vibratory loading, gives an indication of how materials can bear imposed loads without failure. The range of values of CBR in percentage starts from 100% for crushed stones to less than 5% for organic clay or organic silt (Table C9 in Appendix IV).

In Table 4.17, the CBR values of the materials of construction of the failed dams range from 11 to 46%. These materials can exhibit a wide range of behaviors under loading as will be characterized by very strong materials like GW (well graded gravel) to relatively weak materials like ML (silts). The CBR values of materials of construction of the distressed dams were found to range from 14 to 28%. These materials can also exhibit a wide range of behaviors under loading as will be characterized by a relatively strong material like GC (clayed gravel) to a relatively weak material like ML (Silt). The CBR values of the materials of construction of the functional dams range from 14 to 36%. Again, these materials can exhibit a wide range of behaviors under loading as will be shown by relatively strong materials like GC to a relatively weak material like ML. Although the minimum CBR for the failed dams is the lowest, the range is not significantly different form distressed and functional dams soils. This shows that CBR values alone cannot predict failure, distressnes or functionality of earth dams in the study area, but have influenced particular failures and distresses as can be seen in the later part of this work.

Table 4.17; Some Soil Properties and CBR Values

S/N	Name of Dam	Soil Sample location	Moisture Content (%)	Bulk Density Mg/m^3	Dry Density Mg/m^3	CBR (%)	Status
1.	Girei	SPLW	6.4	2.20	2.07	46	Failed
		EM					
		RSV	-	-	-	-	
2.	Guyaku GR Dam 2	RSV	-	-	-	-	Failed
		EM					
3.	Guyaku GR Dam 5	IN GULLY	-	-	-	-	Failed
		EM					
4.	Nzuzu Dam	SPLW	8.6	2.00	1.84	24	Failed
5.	NGGR Dam 1(Dalehi)	EM	14.8	1.94	1.69	20	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	-	-	-	-	Failed
7.	Ali Walga Dam	EM	11.3	2.18	1.96	36	Functional
8.	SBGR Dam 3	RSV	-	-	-	-	Distressed
9.	SBGR Dam 4	EM	5.6	2.02	1.91	28	Distressed
		RSV	-	-	-	-	
10.	Dadinkowa Dam	EM	10.8	2.06	1.86	21	Functional
11.	Bambam Dam	EM	18.5	1.76	1.49	11	Failed
12.	Pindiga Dam I	EM/RSV	9.7	2.07	1.89	14	Functional
13.	Pindiga Dam II	EM/RSV	13.4	1.85	1.63	31	Functional
14.	Bojude	EM/RSV	8.9	1.87	1.72	24	Functional
15.	Jombo Dam Dukku	EM/RSV	15.5	1.70	1.47	25	Functional
16.	Dukku Dam(Kogin Dole)	EM	6.9	2.09	1.96	25	Functional
17.	Cham Dam	EM/RS	14.7	2.16	1.88	17	Failed
18.	Waya Dam	EM(SHELL)	10.8	2.07	1.87	15	Failed (rptdly)
		EM(CORE)	8.1	2.13	1.97	20	
19.	Gubi Dam	EM(SHELL)	9.7	2.08	1.90	36	Functional
		EM(CORE)	12.1	2.15	1.92	34	
20.	Miri Dam	EM/RSV	11.8	2.15	1.92	14	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	11.5	2.14	1.92	21	Functional
22.	Dull Dam	EM(Left)	15.9	2.00	1.73	19	Failed
		EM(Rigth)	10.8	2.23	2.01	23	

Soils were not sampled at some dams because of non cooperation of owners and logistic reasons

KEY;

EM = Embankment

RSV = Reservoir

SPLW = Spillway

4.5.2 Coefficient of Permeability, K

This is a measure of the ease with which water can flow through soil media. The permeability of soils has a decisive effect on the stability of foundations and seepage loss through embankments of reservoirs. Since earth dams are made up of soil materials, the embankments are permeable. The embankment are constructed in such a way that they should be least permeable at the core and safely draining towards the shells so as to allow for minimum seepage quantities and safe draining of seepage water. Table 4.18 and Table C3 (Appendix) and C5 (Appendix) shows the results and the interpretations criteria respectively.

In Table 4.18 the permeability of the materials of construction of the failed dams range from $1.21x10^{-8}$ to $1.21x10^{-6}$m/s which indicate a wide range of permeability phenomena from a practically impermeable material to a poor draining material. The range of permeability values for the distressed dams range from $1.76x10^{-8}$ to $5.65x10^{-8}$m/s indicating that the materials are practically impermeable; and are thus excellent for use as construction materials for earth dams in this regard. The range of permeability values for the materials of construction of functional dams was found to be $8.76x10^{-8}$ to $1.82x10^{-7}$m/s also signifying good materials that are impervious to some extent. About a third of the failed dams have their embankment soil materials with coefficient of permeability between $1.21x10^{-6}$ m/s to $2.10x10^{-7}$ m/s, suggesting the susceptibility of such dams to seepage failure. On a study of gully erosion in the north-eastern Nigeria, Obiefuna et al, (1999) obtained similar results.

Table 4.18; Some Soil Properties and Permeability Values

S/N	Name of Dam	Soil Sample Location	Moisture Content (%)	Bulk Density (ρ)Mg/m^3	Dry Density (ρ_d)Mg/m^3	Void Ratio	Permeability (K) m/s	Status
1.	Girei	SPLW	8.0	2.17	2.01	0.294	$1.21x10^{-6}$	Failed
		EM	18.5	1.88	1.59	0.535	$2.33x10^{-8}$	
		RSV	1.31	1.97	1.95	0.323	$2.73x10^{-6}$	
2.	Guyaku GR Dam 2	RSV						Failed
		EM	6.8	2.09	1.96	0.342	$4.9x10^{-8}$	
3.	Guyaku GR Dam 5	IN GULLY						Failed
		EM						
4.	Nzuzu Dam	SPLW	13.7	2.00	2.29	0.135	$2.79x10^{-8}$	Failed
5.	NGGR Dam 1(Dalehi)	EM	7.6	2.16	2.01	0.199	$5.65x10^{-8}$	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	15.4	1.83	1.59	0.635	$9.33x^{-8}$	Failed
7.	Ali Walga Dam	EM						Functional
8.	SBGR Dam 3	RSV	11.8	2.03	1.82	0.357	$2.31x10^{-8}$	Distressed
9.	SBGR Dam 4	EM	10.8	1.80	1.63	0.552	$1.76x10^{-8}$	Distressed
		RSV	11.5	2.17	1.95	0.349	$1.79x10^{-8}$	
10.	Dadinkowa Dam	EM	21.5	2.03	1.67	0.551	$8.78x10^{-8}$	Functional
11.	Bambam Dam	EM/RSV	12.5	1.99	1.77	0.429	$2.61x10^{-8}$	Failed
12.	Pindiga Dam I	EM/RSV	15.7	2.03	1.76	0.460	$7.71x10^{-8}$	Functional
13.	Pindiga Dam II	EM/RSV	20.1	2.63	1.67	0.614	$2.02x10^{-8}$	Functional
14.	Bojude	EM/RSV	15.6	2.03	1.76	0.534	$2.46x10^{-8}$	Functional
15.	Jombo Dam Dukku	EM/RSV	6.6	2.08	1.95	0.282	$4.43x10^{-8}$	Functional
16.	Dukku Dam(Kogin Dole)	EM	11.1	2.13	1.92	0.354	$1.82x10^{-7}$	Functional
17.	Cham Dam	EM/RS	1.76	1.98	1.95	0.364	$3.42x10^{-8}$	Failed
18.	Waya Dam	EM(SHELL)	14.5	2.04	1.77	0.478	$4.29x10^{-6}$	Failed (rptdly)
		EM(CORE)	13.9	2.01	1.77	0.469	$2.01x10^{-7}$	
19.	Gubi Dam	EM(SHELL)	14.5	2.05	1.79	0.469	$3.00x10^{-8}$	Functional
		EM(CORE)	7.8	2.03	1.88	0.399	$1.23x10^{-8}$	
20.	Miri Dam	EM/RSV	5.36	2.10	1.99	0.322	$3.91x10^{-8}$	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	9.3	2.03	1.86	0.452	$1.64X10^{-8}$	Functional
22.	Dull Dam	EM(Left)	8.84	2.11	1.94	0.289	$1.4x10^{-8}$	Failed
		EM(Rigth)	11.48	2.10	1.88	0.400	$1.21x10^{-8}$	

KEY; EM = Embankment RSV = Reservoir SPLW = Spillway

4.5.3 Shear Strength; C and ϕ

One of the most important properties of soil is its shear strength or ability to resist sliding along internal surfaces within a mass. The stability of a cut, the slope of an earth dam, the foundation of structures, natural slopes of hillsides and other structures built on soil depend upon the shearing resistance offered by the soil along the probable surface of slippage (Murthy, 2008). It is represented as composed of;

1. Internal Friction, or resistance due to inter locking of particles and friction between individual particles at their contact points.

2. Cohesion or the resistance due to interparticle forces which tends to hold the particles together in a soil mass. This can be represented by Coulomb's equation.

$$\tau = c + \delta \tan \phi$$

Where;

τ = Shear strength (kN/m^2)

δ = total normal stress on the failure plane (kN/m^2)

c = Cohesion (kN/m^2)

ϕ = Angle of internal friction (Singh, 2001).

The range of values for C and ϕ for the dams in the study area are as shown in Table 4.19 and the interpretation criteria is in Appendix IV Table C7 and C8.

Table 4.19; Triaxial Compression Test Results (Quick Undrained)

S/N	Name of Dam	Soil Sample location	Cohession (C) kN/m^2	Angle of Imternal Friction $(\phi)^o$	Status
1.	Girei	SPLW			Failed
		EM	74	5	
		RSV			
2.	Guyaku GR Dam 2	RSV			Failed
		EM	215	3	
3.	Guyaku GR Dam 5	IN GULLY			Failed
		EM	60	20	
4.	Nzuzu Dam	SPLW	40	13	Failed
5.	NGGR Dam 1(Dalehi)	EM	70	5	Distressed
6.	NGGR Dam 2(Dalehi)	RSV			Failed
7.	Ali Walga Dam	EM			Functional
8.	SBGR Dam 3	RSV			Distressed
9.	SBGR Dam 4	EM			Distressed
		RSV			
10.	Dadinkowa Dam	EM	40	50	Functional
11.	Bambam Dam	EM/RSV	100	5	Failed
12.	Pindiga Dam I	EM/RSV	85	13	Functional
13.	Pindiga Dam II	EM/RSV	100	8	Functional
14.	Bojude	EM/RSV	62	7	Functional
15.	Jumbo Dam Dukku	EM/RSV			Functional
16.	Dukku Dam(Kogin Dole)	EM	70	14	Functional
17.	Cham Dam	EM/RS	95	23	Failed
18.	Waya Dam	EM(SHELL)	35	18	Failed (rptdly)
		EM(CORE)	123	10	
19.	Gubi Dam	EM(SHELL)	61	18	Functional
		EM(CORE)	40	24	
20.	Miri Dam	EM/RSV	60	17	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	45	10	Functional
22.	Dull Dam	EM(Left)	70	20	Failed
		EM(Rigth)	100	13	

KEY;

EM = Embankment

RSV = Reservoir

SPLW = Spillway

The soil materials for the failed dams have (35 - 215) and (3 – 18) as C and ϕ respectively, showing an excellent soil as far as the shear strength properties are concerned. The ranges of C and ϕ for the distressed dams was found to be (70 – 60) and (5 – 17) which is also having a good shear strength property as to be used for embankment in dam construction. The functional dams have their range of C and ϕ values in the region of (100 – 40) and (8 – 50) respectively, this also indicate a material with good shearing strength characteristics. Ironically the shear strength of the failed dams seems to be better than the distressed and functional dams. Thus shear strength parameters alone cannot determine failure, distress or functionality of dams in the study but have influenced particular incidences of failures and distresses.

4.5.4 Atterberg Limits and Plasticity Index (PI)

This is a measure of consistency of the soils. Plasticity index indicates the degree of plasticity (remolding and shaping) of a soil. The difference between liquid and plastic limits is the plasticity of the soil. A cohessionless soil has zero plasticity index. Such soils are termed as non-plastic. Fat clays are highly plastic and possess a high plasticity index. The results and interpretation criteria are as detailed in Table 4.20 and Appendix IV Table C2 respectively.

In Table 4.20 the range of PI values for failed dams were found to be from 0 to 20 indicating a range of behavior for the construction materials from non-plastic to highly plastic. The range of PI values for distressed dams was 0 to 11, showing a range of behaviors for the embankment materials from non-plastic to soils of medium plasticity. PI values for the functional dams were found to be similar to the failed dams (0 - 21) indicating non-plastic to highly plastic materials. Sixty five percent of failed and distressed dams have their Plasticity indexes between (0-7) meaning failures and distresses affect embankments whose soil materials are of low plasticity.

Table 4.20; Atterberg Limits (Cone Test)

S/N	Name of Dam	Soil Sample location	Liquid Limit (LL)%	Plastic Limit (PL)%	Plasticity Index (PI)	Status
1.	Girei	SPLW	13	-	-	Failed
		EM	19	-	-	
		RSV	18	-	-	
2.	Guyaku GR Dam 2	RSV	-	-	-	Failed
		EM	22	15	7	
3.	Guyaku GR Dam 5	IN GULLY	25	23	2	Failed
		EM	30	23	7	
4.	Nzuzu Dam	SPLW	24	18	6	Failed
5.	NGGR Dam 1(Dalehi)	EM	24	14	10	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	16	-	0	Failed
7.	Ali Walga Dam	EM	19	-	0	Functional
8.	SBGR Dam 3	RSV	30	19	11	Distressed
9.	SBGR Dam 4	EM	26	-	0	Distressed
		RSV	9	-	0	
10.	Dadinkowa Dam	EM	27	16	11	Functional
11.	Bambam Dam	EM	47	27	20	Failed
12.	Pindiga Dam I	EM/RSV	46	25	21	Functional
13.	Pindiga Dam II	EM/RSV	24	21	3	Functional
14.	Bojude	EM/RSV	25	15	10	Functional
15.	Jombo Dam Dukku	EM/RSV	27	18	9	Functional
16.	Dukku Dam(Kogin Dole)	EM	15	-	0	Functional
17.	Cham Dam	EM/RS	43	23	20	Failed
18.	Waya Dam	EM(SHELL)	24	17	7	Failed (rptdly)
		EM(CORE)	21	-	-	
19.	Gubi Dam	EM(SHELL)	23	-	0	Functional
		EM(CORE)	18	-	0	
20.	Miri Dam	EM/RSV	22	15	7	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	23	17	6	Functional
22.	Dull Dam	EM(Left)	27	19	8	Failed
		EM(Rigth)	29	19	10	

KEY;

EM = Embankment

RSV = Reservoir

SPLW = Spillway

4.5.5 Compaction (OMC and Maximum Dry Density, MDD)

This is the process of packing soil particles closely together by mechanical manipulation, thus increasing the dry density of the soil. The MDD which can be obtained by compaction depend upon the type of soil. Well graded coarse-grained soils attain a much higher density than fine-grained soil. Heavy clays attain relatively the lowest densities. Because of the greater surface area of fine particles, fine-grained soils required more water for their lubrication and thus have higher optimum moisture content (OMC) (Singh, 2001).

In Table 4.21, the range of values for the OMC (%) and MDD`S (Mg/$m^3$) of the failed dams were found to be (7.6 - 13.2) and (1.64 - 1.84) exhibiting a wide range of behavior for the soil materials under compaction from coarse-grained non cohesive material to fine-grained cohesive material. The values of OMC and MDD`S for the distressed dams range from (8.7 – 10.7) and (1.75 – 2.01) which also depicts materials from coarse-grained non-cohesive to fine-grained cohesive soils. The OMC`s and MDD`S of the functional dams were also found to follow a similar trend with values ranging from (6.5 – 11.7) and 1.65 - 2.00) signifying embankment materials from coarse-grained non-cohesive soil to fine-grained cohesive material. There is an obvious overlap in the values of MDD and OMC from the results of the standard proctor compaction tests for all the dams in the study area. Most functional dams (80%) have high MDD of 1.84 mg/m^3 and above. This shows that the denser the embankment soil materials the more stable will be the embankment.

Table 4.21; Compaction Test Results

S/N	Name of Dam	Soil Sample location	Optimum Moisture Content (OMC)%	Maximum Dry Density (MDD) Mg/m^3	Status
1.	Girei	SPLW			Failed
		EM	10.5	1.80	
		RSV	-	-	
2.	Guyaku GR Dam 2	RSV	-	-	Failed
		EM	10.4	1.82	
3.	Guyaku GR Dam 5	IN GULLY	-	-	Failed
		EM	11.2	1.80	
4.	Nzuzu Dam	SPLW	7.6	1.84	Failed
5.	NGGR Dam 1(Dalehi)	EM	10.7	1.75	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	-	-	Failed
7.	Ali Walga Dam	EM	6.5	2.00	Functional
8.	SBGR Dam 3	RSV	-	-	Distressed
9.	SBGR Dam 4	EM	8.7	2.01	Distressed
		RSV	-	-	
10.	Dadinkowa Dam	EM	11.2	1.84	Functional
11.	Bambam Dam	EM	12.2	1.56	Failed
12.	Pindiga Dam I	EM/RSV	8.4	1.87	Functional
13.	Pindiga Dam II	EM/RSV	11.7	1.65	Functional
14.	Bojude	EM/RSV	9.7	1.84	Functional
15.	Jombo Dam Dukku	EM/RSV	10.8	1.70	Functional
16.	Dukku Dam(Kogin Dole)	EM	7.2	1.91	Functional
17.	Cham Dam	EM/RS	13.2	1.64	Failed
18.	Waya Dam	EM(SHELL)	10.8	1.96	Failed (rptdly)
		EM(CORE)	10.2	1.83	
19.	Gubi Dam	EM(SHELL)	6.7	1.89	Functional
		EM(CORE)	10.2	2.00	
20.	Miri Dam	EM/RSV	9.4	1.88	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	9.5	1.94	Functional
22.	Dull Dam	EM(Left)	8.8	1.84	Failed
		EM(Rigth)	8.9	1.95	

KEY;

EM = Embankment

RSV = Reservoir

SPLW = Spillway

4.5.6 Sieve Analysis; (USCS Soil grouping)

The classes of soil found in the construction materials of the failed dams include poorly graded sand (SP), well graded sand (SW) uniformly graded sands of low plasticity (SP-SC) and non-plastic well graded silty sands of low plasticity (SW-SM). This shows a wide range of soil materials that can exhibit a wide range of behavior when used as construction materials for earth dams. The distressed dams were found to have been constructed with poorly graded sands (SP), well graded sands (SW) and well graded silty sands of low plasticity (SW-SM). This also shows that the distressed dams construction materials vary widely from poorly graded sands to well graded silty sands of low plasticity. The soil materials for the functional dams include poorly graded sands (SP), well graded sands (SW) and uniformly graded silty sands of low plasticity (SP-SM). Embankment soil materials with Coefficient of uniformity of less than 5 accounted for about 79% of the failures and distresses. This implies that there is lack of finer particles in the soil that can help in cementation.

4.5.7 Consolidation Settlement

Consolidation is synonymous to compression and represents the phenomenon of the gradual reduction in volume of a soil mass, partly or fully saturated under a sustained pressure. Partially or fully saturated coarse-grained soils consolidate less under sustained pressure than partly or fully saturated fine-grained soils. The final test result on consolidation is given as settlement in mm, which is an indication of how embankment materials can settle under sustained loading over time.

The average settlement of failed dam soil materials was found to be 2.29 mm while that of a functional dam was found to be 1.18 mm, meaning that more settlement is recorded for the failed dam than functional dam. Hence, the weaker the soil the greater the chances of failure, this also agrees with Adejumo *et al*. (2012) on their work on major weak soils.

Overall, the results show that; Wide range of soil groups were used for construction of earth dams in north-eastern Nigeria. This range from poorly graded sands (SP) to silty sands (SM)/Clayey sands (SC), indicating that the soils are good to fair enough to be used as construction materials for earth dams. Dams whose embankment soil materials have Specific gravity (Gs) of 2.63 and below exhibited greater (92%) failures and distresses than those with higher Gs. This means that the cleaner the soil grains the less will be the cementing effects between them. Embankment soil materials with Coefficient of uniformity of less than 5 accounted for about 79% of the failures and distresses. This Means that, uniform graded soil lack the finer particles and intermediate particles that help in binding the soil together. Sixty five percent of failed and distressed dams have their Plasticity index (PI) values between (0-7) meaning failures and distresses affects embankments with low plasticity. Most functional dams (80%) have high MDD of 1.84 mg/m^3 and above. This shows that the denser the embankment soil material the more stable will be the embankment. Good percentages of the failed dams (33%) have their embankment soil materials with coefficient of permeability between 1.21x10^{-6} m/s to 2.10x10^{-7} m/s, suggesting the susceptibility of such dams to seepage failure.

Properties of soil can influence failure, distressnes or functionality of earth dams in conjunction with other engineering factors (feasibility studies, design and construction), geological factors and hydrometreological conditions as can be seen in the subsequent section of this thesis, where each individual dam will be discussed based on these factors. Table 4.22 gives the summary of the range of soil properties against status of dams in the study area.

Table 4.22; Soil Properties and Status of Dams

S/N	Status	Parameter Range of Values
1.		California Bearing Ratio CBR (%)
	Failed	11- 46
	Distressed	14 – 28
	Functional	14 – 36
2.		Permeability (m/s)
	Failed	$1.21x10^{-8}$ – $1.21x10^{-6}$
	Distressed	$1.76x10^{-8}$ – $5.65x10^{-8}$
	Functional	$8.78x10^{-8}$ – $1.82x10^{-7}$
3.		Shear strength (C in KN/m^3 and ф in Degrees(^{o}))
	Failed	215 & 3 – 35 & 18
	Distressed	70 & 5 – 60 & 17
	Functional	100 & 8 – 40 & 50
4.		Atterberg Limits (PI values)
	Failed	0 - 20
	Distressed	0 - 11
	Functional	0 - 21
5.		Compaction test (OMC in % and MDD in Mg/m^3)
	Failed	7.6 & 1.84 – 13.2 & 1.64
	Distressed	8.7 & 2.01 – 10.7 & 1.75
	Functional	6.5 & 2.00 – 11.7 & 1.65
6.		Sieve analysis (USCS groups)
	Failed	SP, SW, SP-SC and SW-SM
	Distressed	SP, SW and SW-SM
	Functional	SP, SW, SP-SM
7.		Consolidation (mm)
	Functional	1.180
		-
	Failed	2.29

The overall results of the soil analysis were analyzed with respect to failure, distressnes and functionality using ANOVA. This tool tries to show to what extent the soil properties differ for a particular status. The result shows that there are no significant differences in the soil properties with respect to status of the dams in the study area as presented in Appendix V. In categorical terms, it means soil properties alone cannot determine failure, distressnes or functionality of earth dams in the study area. In a similar study by Osim (2006) on the distribution of engineering properties of soils used in highway construction in Nigeria; it was concluded that the soils used as sub-base materials in the North-east were of good quality except for a few locations where the specifications were not met. Obiefuna *et al* (2010) found out that the soils at New Demsa, Farei, Numan town and Imbru in Northeastern Nigeria are suitable for use as sub-grade/filling materials, while the soils in a nearby area at Dowaya is unsuitable for use as sub-grade/filling and sub-base matetials for road construction.

4.6 Failed Dams

4.6.1 Bambam Dam

Bambam dam is located on the Bima sandstone which Offadile (1992) described as stable and fairly impervious to form a sound foundation. The environment is hot with maximum temperatures of 42^0C. The evaporation of 333.7mm/month(July) is slightly more than the rainfall of 327.1 mm/month(July) thus leading to a high loss of water from the reservoir (Oke et al, 2011).

The soil material is a poorly graded sand (SP) with medium plasticity (PI of 20%) indicating an excellent embankment construction material (Singh, 2001). The CBR of 11% indicates that the material can be affected by compression when poorly compacted (Murthy, 2008). The CBR can be improved by addition of Cement Kiln Dust according to Iorliam *et al*, (2012). The soil material has good shear strength with cohesion of 100KN/m^2 and angle of internal friction of 5^0 giving the material additional advantage as an embankment construction material.

The design height of the was not achieved when the contractor left the site and was paid off. Compaction was achieved using a dozer which usually does not give good results.

Bambam dam failed hydraulically when the reservoir water overtopped the otherwise short and settled embankment forming a complete breach of the dam with the stream water passing through freely (Figure 4.14A-C).

Fig 4.14; Babam dam failure

A; Embankment completely breached allowing the stream to pass freely

B; A section through the embankment failure

C; Spillway improperly located to carry excess flood

4.6.2 Dull Dam

Dull dam is situated on the basement complex that is relatively stable and fairly impermeable thus giving the dam a good foundation (Offadile, 1992). The rainfall in the area is around 339.1mm/month (June) with relatively lower evaporation. The dam is recharged from a stream and runoff waters generated upstream.

The embankment consisted of two different materials at the right and left side of the spillway. Both embankment materials fall within the USCS group of SW (well graded sand) of medium plasticity ($8 < PI > 10\%$), CBR (19 to 23%) and k values of $1.21x10^{-8}$m/s – $1.40x10^{-8}$m/s. The shear strength parameter C ranges from 70KN/m^2 to 100KN/m^2 and angle of internal friction of 13 to 20^o. This gives an excellent construction material with good plasticity, relatively non settleable under compression, highly impermeable and an excellent shear strength (Singh, 2001, Agarwal, 2001, Murthy, 2008 and Brink et al, 2008).

Dull dam failed as a result of a poorly constructed spillway which gave way during the first filling of the reservoir. The stone pitched spillway was fixed without a good foundation. These explains why the spillway failed and the embankment remained intact, allowing all the reservoir water to drain completely through the broken spillway. The appearance of ants infestation on the crest shows a sign of danger from seepage and probably piping (Figure 4.15A-D).

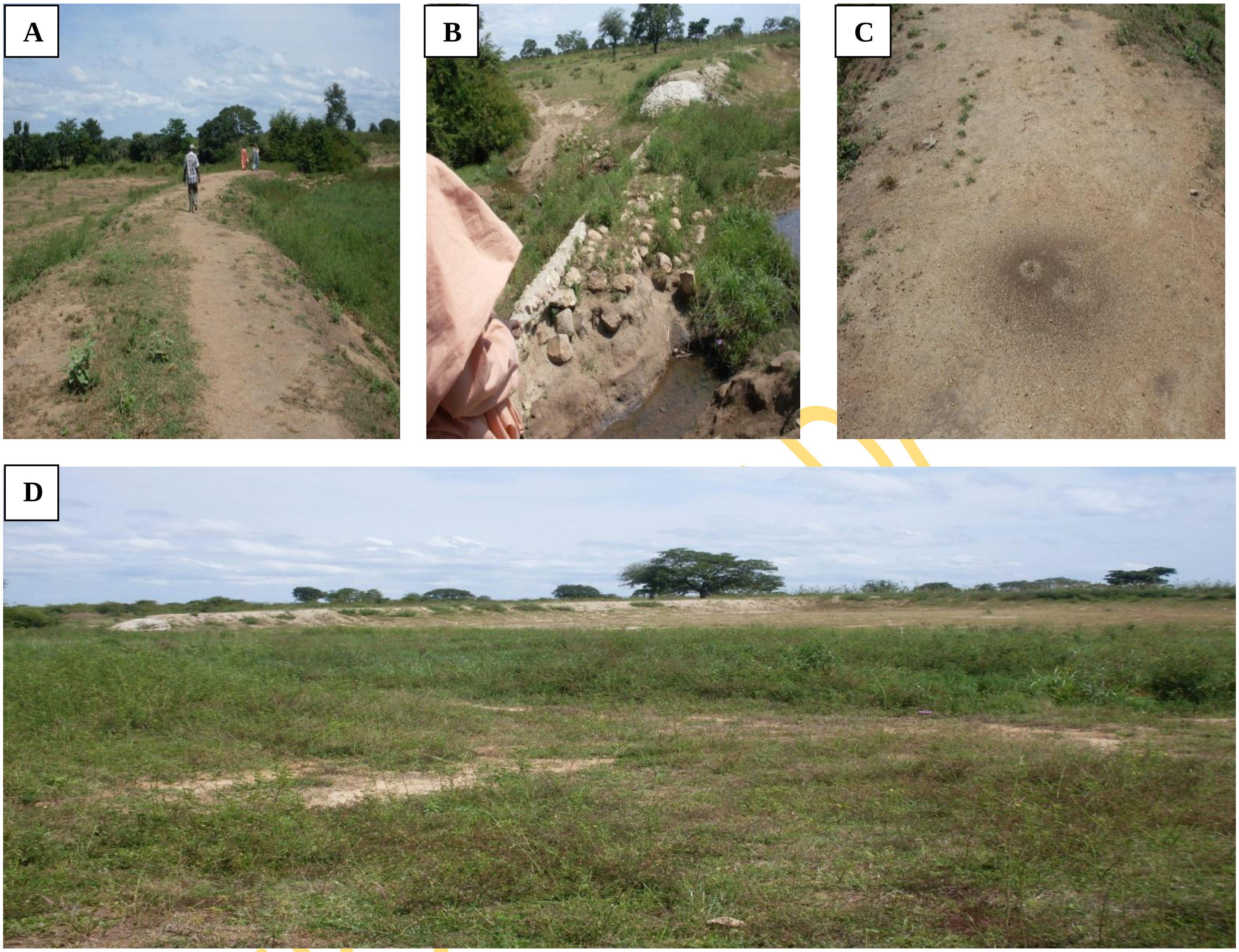


Fig 4.15; Dull Dam Failure

A; **A poorly compacted embankment**

B; **Lack of foundation and poorly stone pitch spillway led to its failure**

C; **Ants infestation on the embankment**

D; **Embankment without any riprap or turfing sod**

4.6.3 Guyaku Dam 5

The dam is situated on geology of basement complex formation which is fairly stable and draining (Offodile, 1992). The maximum temperature is 43^{o}C. A total monthly rainfall of 437.8mm (August) and relatively lower evaporation of 354.64mm gives a scenario that is tasking to the reservoir notwithstanding the relative humidity of 77%.

The soil is well graded sand with little or no plasticity (PI of 2). OMC of 11.2% and MDD of 1.8Mg/m^3. This signifies a soil that is stable when compacted. The permeability of 4.9x10^{-8} m/s indicate a fairly impermeable soil with good shear strength properties of 60kN/m^3 and angle of internal friction of 20^{o}. The overall soil analysis shows that the soil is good enough as an embankment material.

The dam failed as a result of poor construction. Eye witness account reveal that the dam failed from the initial impoundment. A deep gully was observed to start from the reservoir and continues downstream cutting the embankment beyond original ground level. Lack of definite spillway facilitated the sudden failure. The reservoir seems to be too close to the surrounding hills where high runoff velocities are very much expected.

Deep wide cracks were observed on the underlining soils around the surrounding area where the dam was constructed. The dam was never utilized and hence maintenance was not affected for any reason (Figure 4.16A-C).

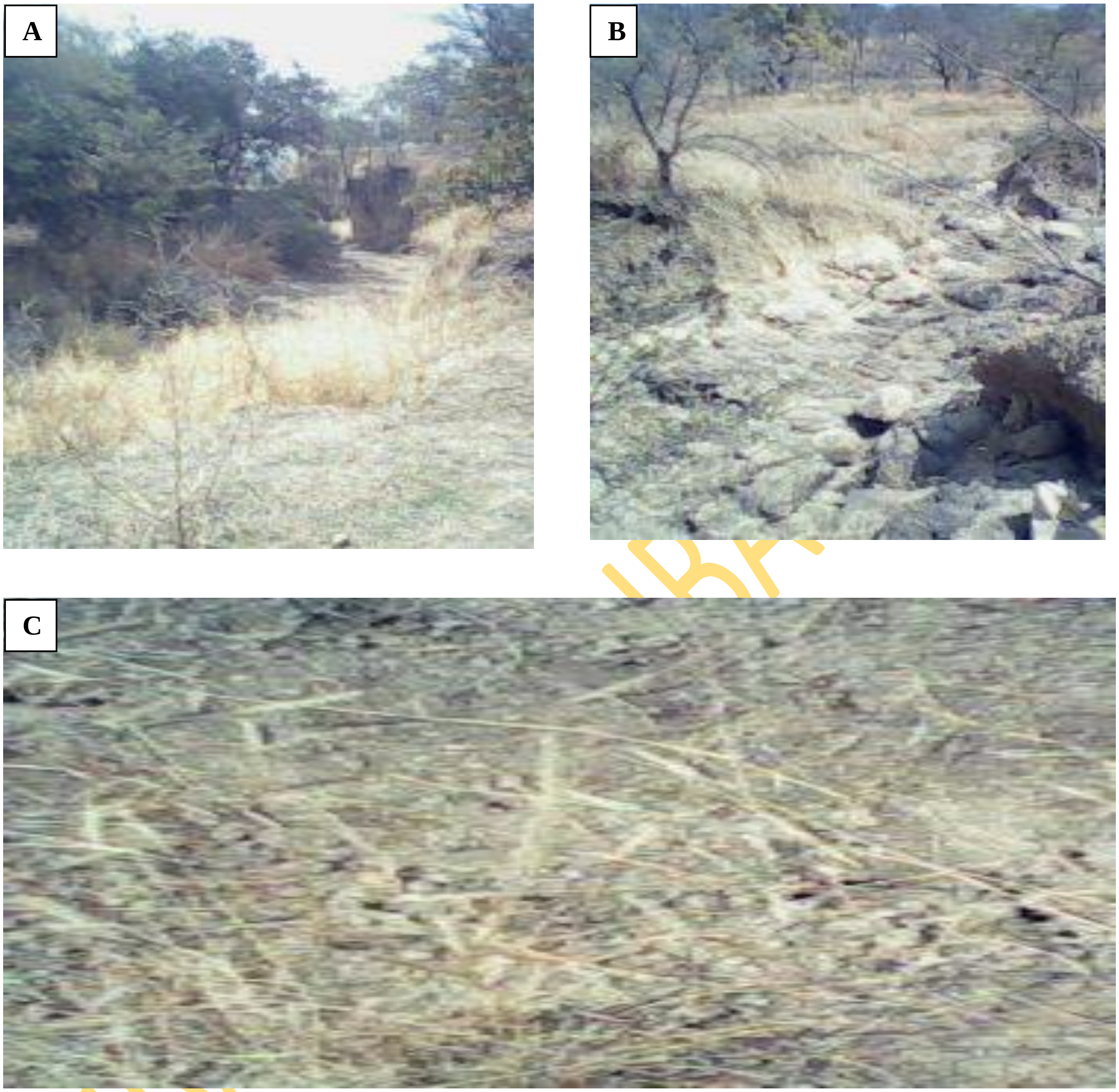


Fig 4.16; Guyaku grasing reserve dam 5 failure

A;	Gully cut across the embankment

B;	Embankment erosion

C;	Insitu soil conditions at dam site

4.6.4 Nasarawo Gongoshi Grazing Reserve Dam 2

The dam is also situated on a geology of basement complex formation which is stable and draining. The maximum temperature is 43^{o}C. The total maximum monthly rainfall is 437.8mm. The relative humidity is 77% with a slightly lower total monthly evaporation of 354.64mm.

The reservoir was observed to be silted up. Erosion has destroyed the embankment due to lack of spillway and maintenance. The dam is generally poorly maintained. There is no slope protection in place. The dam has completely failed (Figure 4.17A-E).

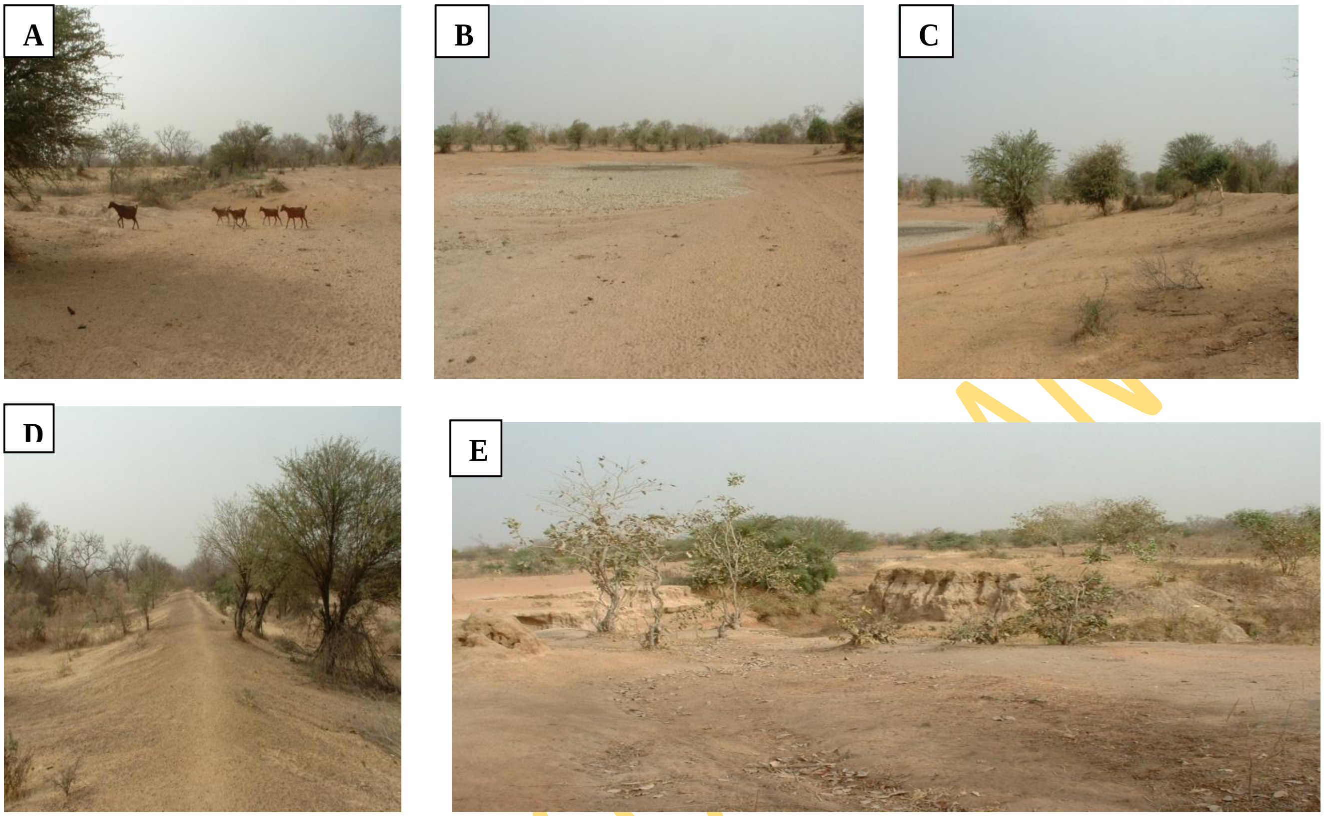


Fig 4.17; Nasarawo gangoshi grazing reserve dam2 failure

A; Land use and environmental factors encouraged erosion at dam site

B; Siltation of reservoir

C; Reduction of reservoir volume and embankment height

D; Trees on the embankment

E; Gully formed at the abutment of the embankment

4.6.5 Sarau Belel Grazing Reserve Dam 2

The dam is situated on geology of basement complex formation that is stable and has good drainage characteristics.

A maximum temperature of 43^oC, total maximum monthly rainfall of 437.8mm, a Relative Humidity of 77% and an evaporation of 354.64mm present a difficult situation for the reservoir to conserve the water inside it due to the high temperatures and relatively high evaporation obtained in the surrounding dam catchment.

Originally, runoff from the surrounding hills and mountains recharges the reservoir through a stream. As at the time of the site visit the seasonal stream that recharges the reservoir does not flow into the reservoir. It is diverted over time to flow away from the dam due to lack of maintenance.

The reservoir is completely empty. The dam has virtually failed hydraulically. There is no slope protection nor appropriate spillway at site. There is virtually no access road to the dam site to facilitate maintenance (Figure 4.18A-C).

4.6.6 Cham Dam

Cham dam lays on the foundation of stable basaltic rocks of the Dadiya formation, which gave the dam a solid foundation, the construction of which did not require a cut off wall.

The catchment area has an evaporation of 327mm/month, a rainfall of 327.1mm/month and a maximum temperature of 42^0C. Thus gives a scenario of a hot environment with high evaporation. Although the reservoir receives inflow from River Cham, a lot of water is lost through evaporation.

The soil material of both the embankment and the reservoir is a well graded sand (SW) of medium plasticity (20%) and a CBR of 17%. The shear strength of the soil is described by a cohesion of 95kN/m^2 and angle of internal friction of 23^0. The permeability coefficient is 3.42x10^{-8}m/s. This indicate an excellent workable material for use as an embankment material which is practically impervious with good shear strength when compacted (Arora, 2001, Singh, 2001 and Murthy, 2008).

The spillway of the dam was not completed signifying danger to the dam during high inflows. Again, the design height of the dam was not achieved when the dam was hurriedly commissioned in 1992. The dam was constructed without following the design specifications. Two different designs were found for the dam crossection; one design drawing shows a zoned embankment with clearly delineated zones of central impervious core flanked with zones of pervious shells while the other one does not contain such zonings. Physical examination of the dam crossection along the breach shows a homogeneous embankment with a mismatch of clayey to gravel size particles with no clear zoning. These are the major reasons for failure. A similar phenomenon was observed on world population of dams by Foster (2000) and Foster et al, (2000). The dam failed hydraulically and structurally when it was overtopped by a flashflood which undermines the height of the shortened embankment with uncompleted spillway and a very small outlet which could not drain the reservoir fast enough. With the absence of zoning a complete breach of the embankment resulted in the cutting of the dam up to foundation level, allowing the river to pass across the dam freely (Figure 4.19A-F).

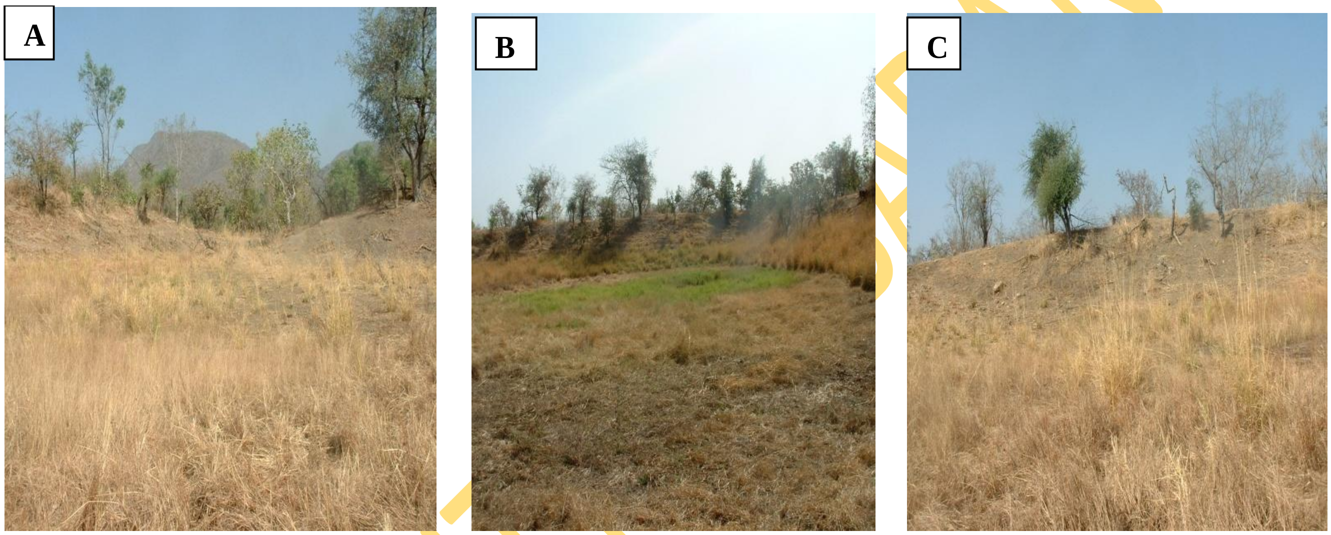


Fig 4.18; Sarau Belel Grazing Reserve Dam2 Failure

A; Chanel recharging the reservoir diverted overtime due to lack of maintenance

B; Empty reservoir indicating hydraulic failure

C; A poorly maintained embankment with trees growing on it

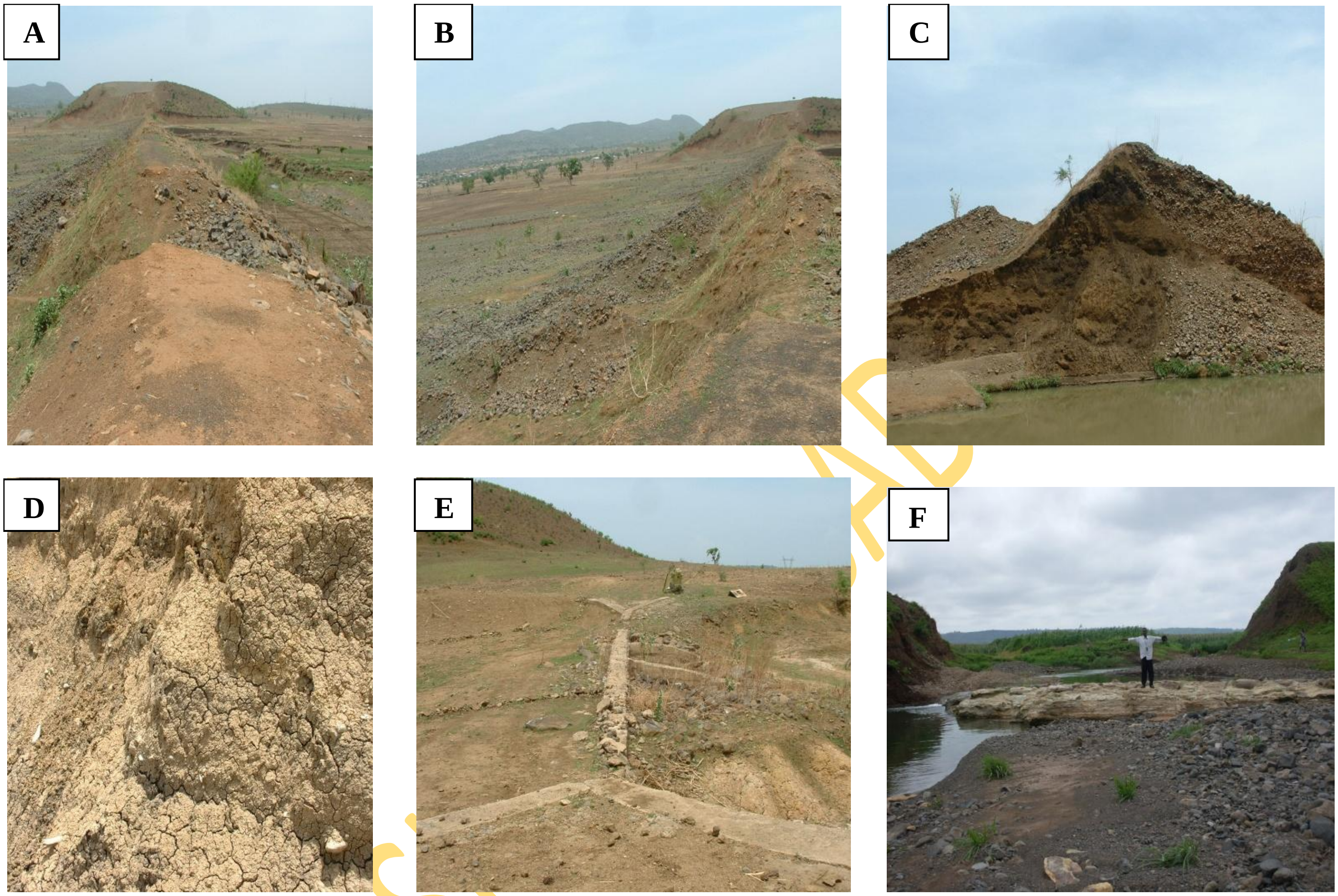


Fig 4.19; Cham Dam Failure

A; Settlement and slope failures at different parts of the embankment

B; Absence of toe drain or toe weir at downstream seepage path

C; A section through the embankment at failure point

D; A close look at the soil material

E; The uncompleted spillway (Broad crested weir)

F; Section where the breach cut the dam axis at it’s lowest

4.6.7 Guyaku Dam 2

The dam is located on a basement complex which is fairly stable and well draining. The maximum temperature of 43^{o}C, maximum monthly rainfall of 437.8mm, a relative humidity of 77% and a relatively lower evaporation of 354.64mm/month gives a favorable dam site with promising hydro-geological conditions.

The soil is well graded sand with no plasticity and hence depicts a soil that is highly stable. The compaction characteristics give an OMC of 10.4% and MDD of 1.82Mg/m^3 indicating a relatively stable soil under compaction. The permeability of 4.9x10^{-8}m/s gives a good sealing material for the embankment. The shear strength properties of Cohesion of 215KN/m^3 and an angle of internal friction of 3^{o}C implies a cohesive soil with little internal friction. The soil is good enough to be used as embankment material.

The dam failed as a result of general lack of maintenance. An eye witness account indicated that, the dam was in threat of failure for a long time but nothing was done. The dam finally gave way when part of the embankment was washed away as a result of serious erosion and ultimate overtopping. There was no definite spillway in place to cope with the reservoir water that led to the ultimate failure of the dam. This resulted in the formation of a gully that cut the dam axis at the right abutment below original ground level.

Loose unstable and fissured soils were observed at the dam site, showing the presence of expansive soils at site (Figure 4.20A-B).

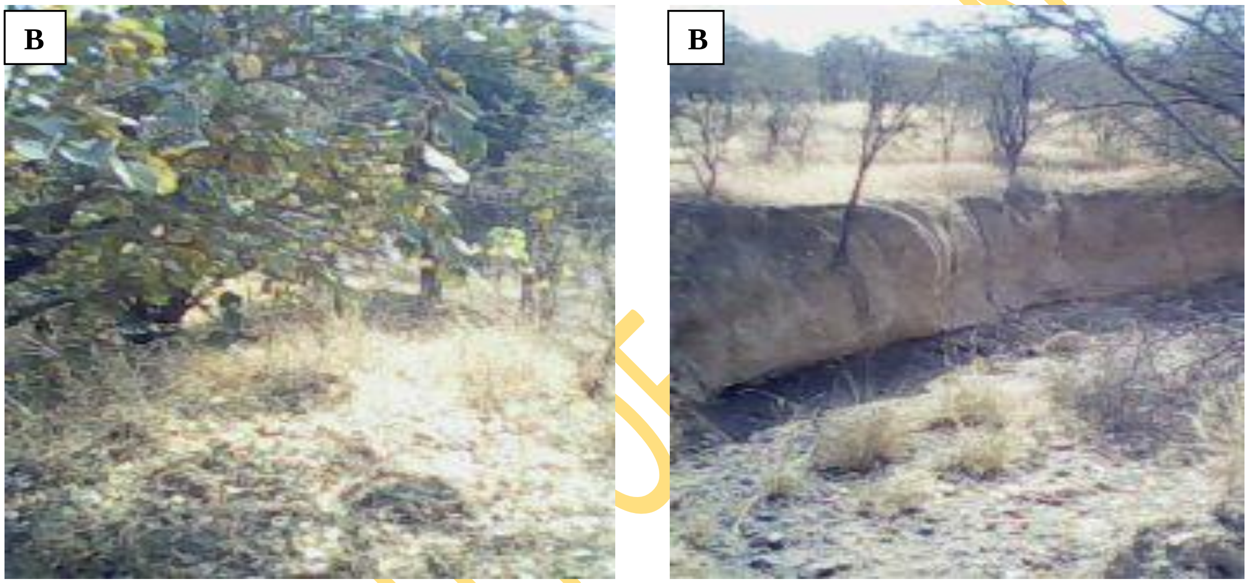


Fig 4.20; Guyaku Grazing reserve Dam2 failure

A; Trees and shrubs growing on embankment

B; A gully cut across the embankment at the right abutment

4.6.8 Girei Dam

Girei dam failed partly as a result of its location on alluvium formation which is highly draining. The reservoir catchment and the spillway area have high coefficient of permeability (1.21x 10^{-6}m/s).Seepage was also noticed around the spillway area in the stilling basin.

The maximum temperature around the catchment can be as high as 42^{o}C, signifying a hot environment with high evaporation loss in the reservoir. The evaporation of 339 mm/month is far higher than the average monthly rainfall of 244.6mm/month.

The embankment soil material is a poorly graded sand of medium plasticity (13%) with a CBR of 40% indicating a very good construction material that can be stable (Singh, 2001) along with a permeability of 2.233x10^{-8} (relatively impervious) giving it an additional advantage. The material also has good shear strength with cohesion of 74KN/m^{2} and angle of internal friction of 5^{0} (Murthy, 2008). This explains the unique failure of Girei dam where the embankment remains intact while the reservoir remains empty for greater part of the year. The reservoir dries up almost immediately after the rains. Girei dam failed essentially as a result of seepage, with spillway failure and high evaporation from the reservoir area (Figure 4.21A-F).

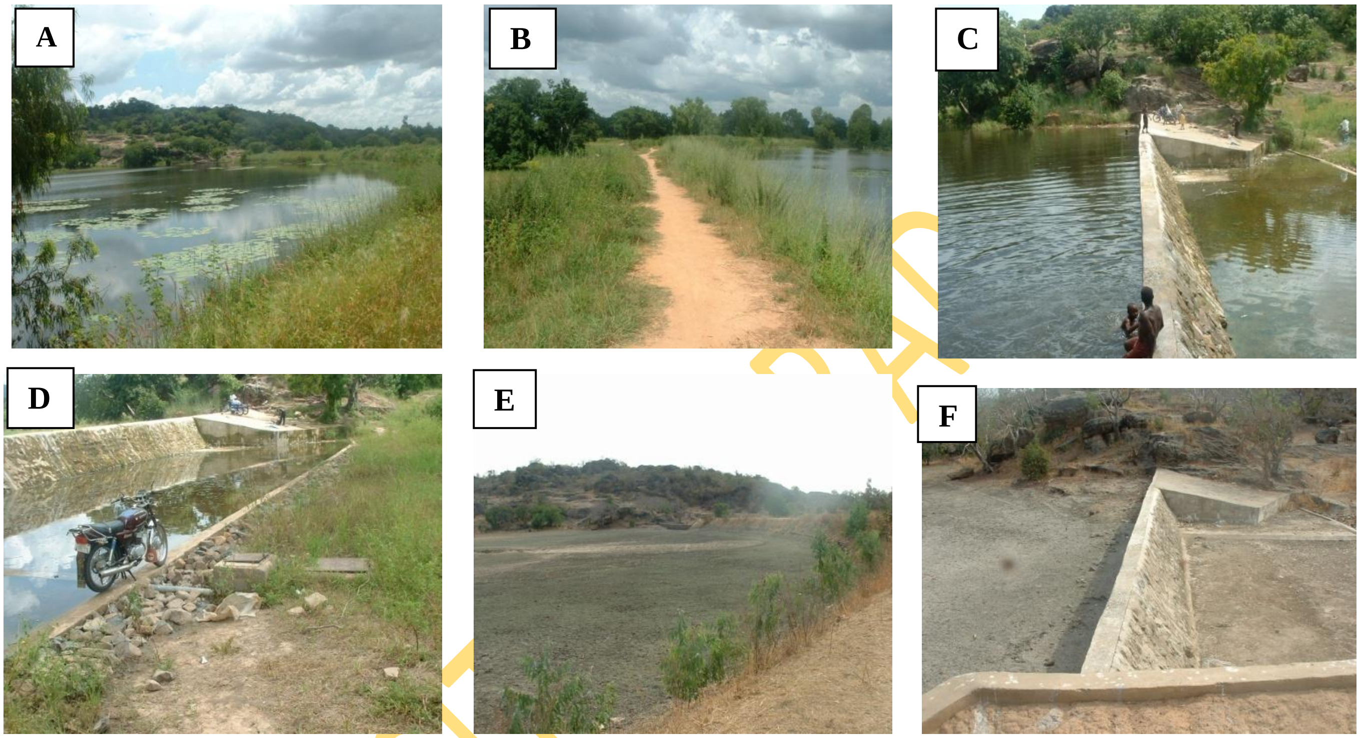


Fig 4.21; Girei Earth Dam

A;	Reservoir contains water during rainy season

B;	Stable embankment with grasses growing on top

C;	Spillway about to spill downstream

D;	Seepage water observed in the stilling basin

E;	Reservoir empty during dry season

F;	Dry spillway in the dry season

4.6.9 Nzuzu Dam

This is a small dam constructed for flood protection of downstream Garkida Township and for animals watering. The dam is located on a basement complex that is fairly stable and well drained. A maximum temperature of 43^{o}C, total monthly rainfall of 437.8mm and a relatively lower total monthly evaporation of 354.64mm indicate a situation of high evaporation loss in the reservoir notwithstanding the high relative humidity of 77%.

The soil is uniformly graded sand of low plasticity (PI of 6). The compaction characteristics of OMC of 7.6% and MDD of 1.84mg/m^3 shows a soil that is stable under compaction as is usually required for embankment construction. A permeability of 2.79x10^{-8}m/s shows a relatively impermeable membrane that is good for embankment construction. A CBR value of 24% indicated a fairly stable soil with little or no settlement under loading. The overall soil analysis indicate that the soil is a bit weak in supporting the stone pitched spillway. This observation agrees with Yunis *et al* (2010) in a geotechnical study of gully sites around the study area. The soils are poorly to well sorted in some places, possess moderate to high plasticity and are easily friable. The study also revealed that the soil is generally loose with low content of fine grain material such as silt and clays that provide cohesion, have moderate seepage fluxes and adverse pore pressure which make them easily erodible.

The dam failed as a result of poor construction of the spillway coupled with general lack of maintenance of the dam. The stone pitched spillway gave way after a heavy storm. The flood breached the embankment below original ground level, retracing the original stream downstream. Animals also stampede the embankment further weakening it and aggravating erosion problems. There is no slope protection of any kind for the dam. The impounded water in the reservoir escaped through the eroded spillway, flooding some parts of Garkida town (Figure 4.22A-C).

Fig 4.22; Nzuzu dam failure

A; Spillway failure allows water to pass freely through the embankment

B; Big trees growing on embankment

C; Reservoir emptied through the failed spillway

4.6.10 Waya Dam

The dam is located on a basement complex formation that is stable and with good drainage properties (Offodile, 1992). An average maximum temperature of 40^{o}C, total monthly rainfall of 478.8mm, a Relative Humidity of 88% and a very low monthly evaporation of 17.4mm gives excellent conditions for reservoir performance as far as the weather conditions are concerned. The weather of the surrounding dam catchment does not encourage any significant loss of water from the reservoir.

The soil of the core material fall within the USCS group of SW-SM (well graded silty sand) of low plasticity, with a PI of 7. The compaction parameters with OMC of 10.2% and MDD of 1.83Mg/m^3, indicated a soil that is stable under compaction and loading. A CBR of 20% also indicates a material that can undergo little or no settlement under loading. The shear strength characteristics of Cohesion of 123kN/m^2 and an angle of internal friction of 10^{o} also give an excellent construction material for earthen embankments. The permeability of 2.01x10^{-7}m/s indicated a material that is relatively permeable but can be used successfully as a core material of an earthen embankment. From the soil analysis of the core material, the soil is excellent enough to be used as an embankment material without any chance of failure.

The shell soil material fall within the USCS group of SP i.e non plastic poorly graded sand. This gives the embankment additional stability and drainage capabilities. The compaction parameters of OMC (10.8%) and MDD of 1.96Mg/m^3 depict a material that is stable under compaction. A CBR of 15% for the shell material shows that the material can settle a little under loading. This is a sign of danger in a way because the shell material is supposed to give the core additional stability and drainage not to give way under loading and pore water pressure. The permeability of 4.29x10^{-6}m/s provides the required drainage conditions for shell materials of earthen embankments. The shear strength properties of Cohesion (35kN/m^2) and an angle of internal friction of 18^o give a material that is stable and rough enough to be successfully used as a shell material of an earthen embankment. Overall the shell material is good enough to be used as such.

Cracks were noticed on the crest at several positions. Slope failures were also noticed at different points on the embankment. The spillway was undermined and the flood wall failed subsequently. Seepage was noticed on the downstream shell of the embankment which resulted to piping and subsequently led to complete breach of the embankment.

Reports have it that the design crest level was not achieved and coupled with too narrow spillway, the reservoir water overtopped the embankment. Both the spillway and the embankment suffered serious damages. The spillway was washed away by impounded water, the water tracks back and washed most of the downstream shell. Seepage resulted to piping and complete failure of the embankment as water from the reservoir escaped through the embankment. The official reason given is that the stockpiled materials after scraping was left on the embankment and later spread when the contractor left the site. Later the work was continued as direct labor under the UBRBDA. The loose materials (dust) did not compact to desired level due to difficulty with terrain as the roller cannot be maneuvered to achieve that. After the contractor left the site, vehicular, human and animal traffic created a loose dusty layer on the embankment. Loose materials (dust) in between the contractors work and that of UBRBDA staff (direct labor) was created. The loose material that was left on the embankment which could not compact very well gave way for seepage and resulted into piping which led to the failure of the Waya dam in Bauchi.

The embankment is now being rehabilitated but still there are signs of erosion on the downstream face of the embankment. The reservoir is functional and contains water throughout the year. The upstream face is protected with rock ripraps while the downstream face has a combination of riprap and grasses (kirikiri). As at the time of visit, the spillway is spilling water safely downstream. Termite infestation is observed on the embankment (Figure 4.23A-E).

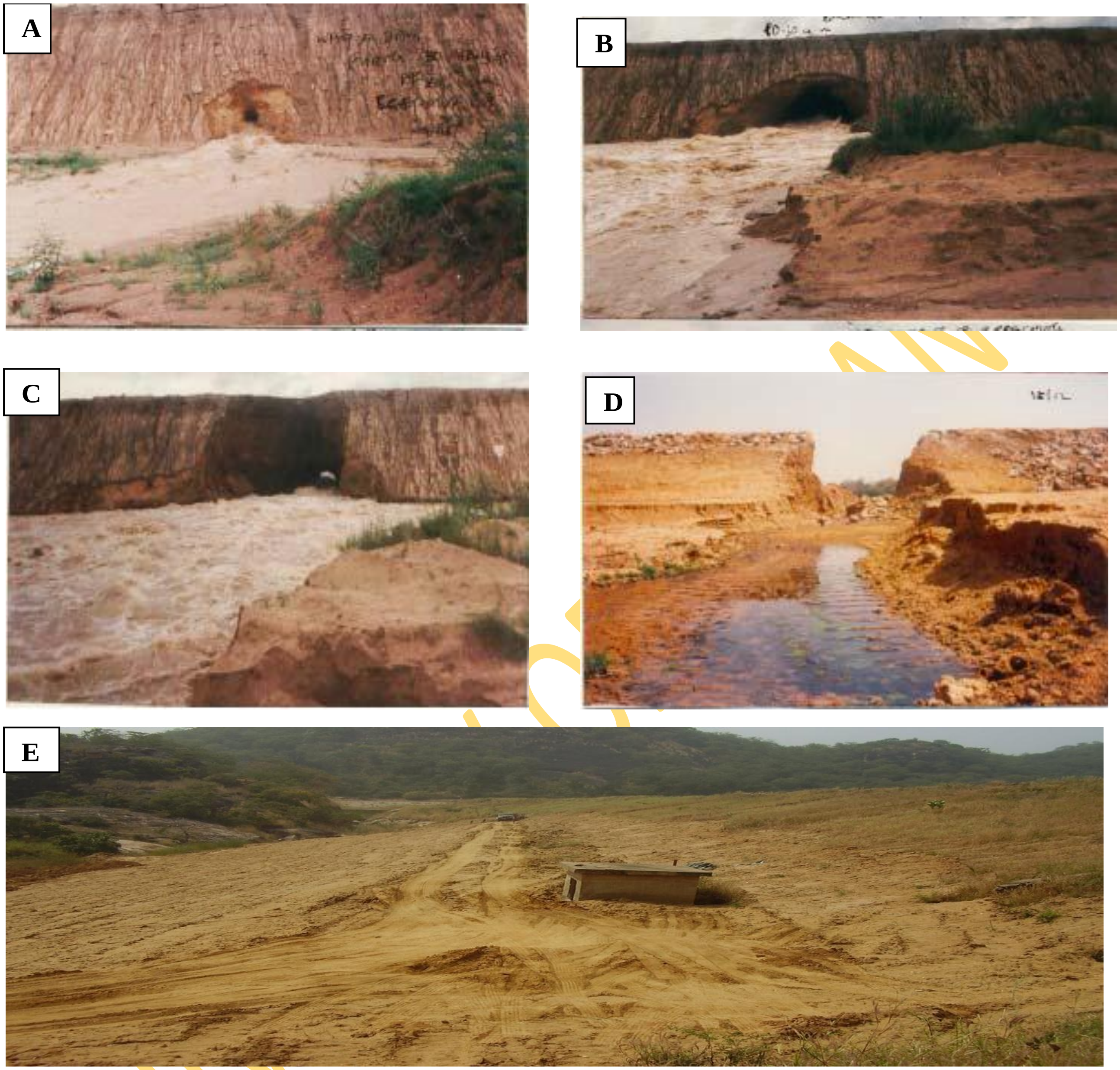


Fig 4.23; Waya Dam Failure (Seepage and piping failure)
(source: Bada, 2008)

A; Piping 45mins after seepage was observed

B; Piping 2hrs 30mins after seepage was observed

C; 9hrs 30mins after piping was observed

D; Breached section of dam

E; Rehabilitation work on progress on the downstream shell

4.6.11 Nasarawo Gongoshi Grazing Reserve Nasarawo Dam 3

The dam is situated on a basement complex formation which is relatively stable and fairly draining. The maximum temperature of 43^oC, total monthly rainfall of 437.8mm, total monthly evaporation of 354.64mm and Relative Humidity of 77% gives a scenario that can task the reservoir as far as the weather situation is concerned.

Due to difficult terrain, the dam could not be accessed for physical examination and picking of soil sample for analysis. Eye witness account confirm that serious cracks have manifested on the embankment. Official report indicate that the reservoir was heavily silted after which the embankment was overtopped as a result of the reduced carrying capacity of the reservoir. The embankment was broken as a result of overtopping and a big gully resulted that cut the embankment below foundation level. Generally the dam failed as a result of poor construction and bad maintenance practice. There is no access road to the dam to facilitate maintenance.

4.7 Distressed Dams

4.7.1 Miri Dam

The dam is situated on the basement complex that is fairly stable and impervious. The weather is hot with maximum temperature of 42^0C an evaporation of 299.48mm/month with a rainfall of 244.6mm/month showing that more water is lost than is received directly from the rainfall. The reservoir is recharged through a stream and runoff from the surrounding hills. Thus explains why the reservoir contains water throughout the year.

The soil material of the embankment is a well graded sand (SW) with medium plasticity (PI = 7), a CBR of 14%, permeability, K of 3.91x10^{-8}m/s and shear strength values of cohesion of 60KN/m^2 and $\phi = 17^0$. This shows a good material for construction and is impermeable with little compressibility under loading. The soil is of good shear strength (Brink *et al*, 1982; Alam, 2001; Arora, 2001; Murthy, 2008).

The embankment seems to be poorly compacted and poorly maintained with no slope protection of any kind in place. The spillway is not located properly. Persistent loading of the reservoir may result to seepage and piping of the embankment as a result of the poor compaction of embankment. Since the spillway is out of alignment, water from the reservoir may overtop the dam and lead to complete failure. Siltation of the reservoir is also evident which again reduces the reservoir carrying capacity and exposes the dam to dangers of overtopping and complete breach (Figure 4.24A-C).

Fig 4.24; Miri Distressed Dam

A; A poorly constructed and poorly maintained embankment

B; A big life reservoir supported by this small embankment

C; Spillway designed and constructed without achieving the desired result

4.7.2 Sarau Belel Grazing Reserve Dam 3

The dam is located on a basement complex formation. With a maximum average temperature of 43^{o}C, monthly total rainfall of 437.8 mm, total monthly evaporation of 354.64mm and a Relative Humidity of 77% give a scenario of a heavily tasked reservoir. There is high tendency for the water to be lost due to high temperatures and high evaporation in the surrounding catchment.

Logistics reasons could not allow the possibility of picking soil samples for analysis of the embankment soil.

The reservoir is heavily silted, the embankment is poorly maintained, and generally unattended to. The reservoir capacity is generally reduced with siltation. Termite moulds have infested the embankment. There is no access road to the dam to facilitate maintenance. If nothing is done, the dam will definitely fail due to siltation and subsequent overtopping when the reservoir could not carry its design capacity. The resultant effect will be failure hydraulically by overtopping of the embankment (Figure 4.25A-C).

4.7.3 NGGR Dam 1 (Dalehi)

The dam is situated on the basement complex. (Offadile, 1990) that is fairly stable and impervious. The weather is hot with maximum temperatures of 42^{0}C and evaporation of 399.48mm/month which is more than the rainfall of 244.6mm/month giving a serious task to the reservoir (Oke *et al*, 2011) though the reservoir is being recharged from runoff of the catchment.

The soil material for the embankment is a poorly graded sand (SP) of medium plasticity (PI=10%), CBR of 20% and permeability coefficient of K= 5.65x10^{-8}m/s with shear strength parameters of 70KN/m^{2} as cohesion and 5^{0} as angle of internal friction. This indicates an excellent construction material that is fairly stable and impermeable. The shear strength properties are good enough (Foster et al, 2000; Foster, 2000 and Agarwala, 2009).

The dam is poorly maintained and excessively eroded. Trees grow on the embankment while the reservoir is heavily silted. In the event that the reservoir receives the design capacity, the embankment may be easily overtopped and coupled with a highly eroded embankment, a dam breach may occur (Fig. 4.26A-D).

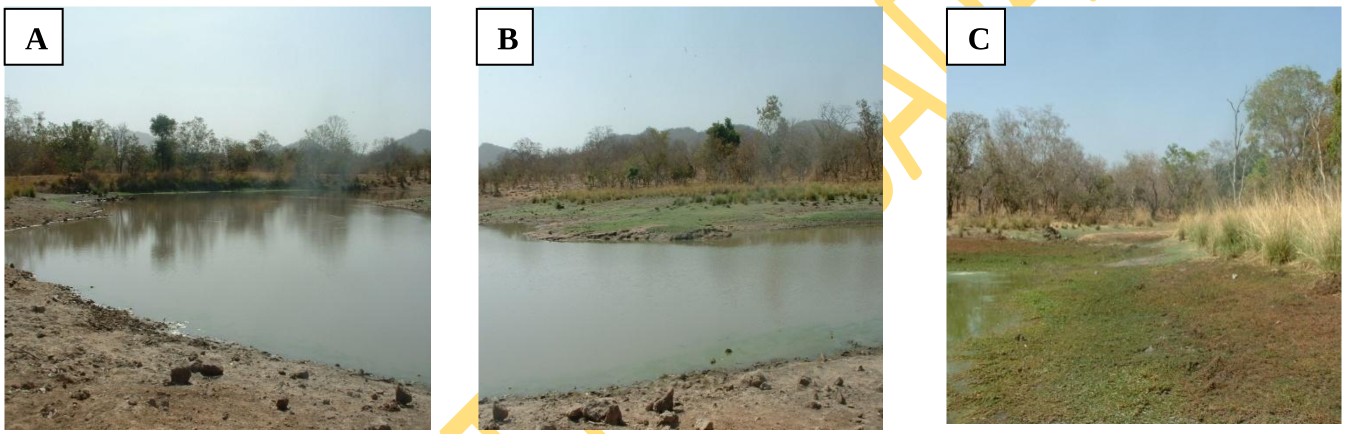


Fig 4.25; Sarau Belel Grazing Reserve Distressed Dam3

A; Progressive siltation of reservoir

B; Poorly maintained embankment and a reduced reservoir capacity

C; Recharge channel poorly maintained

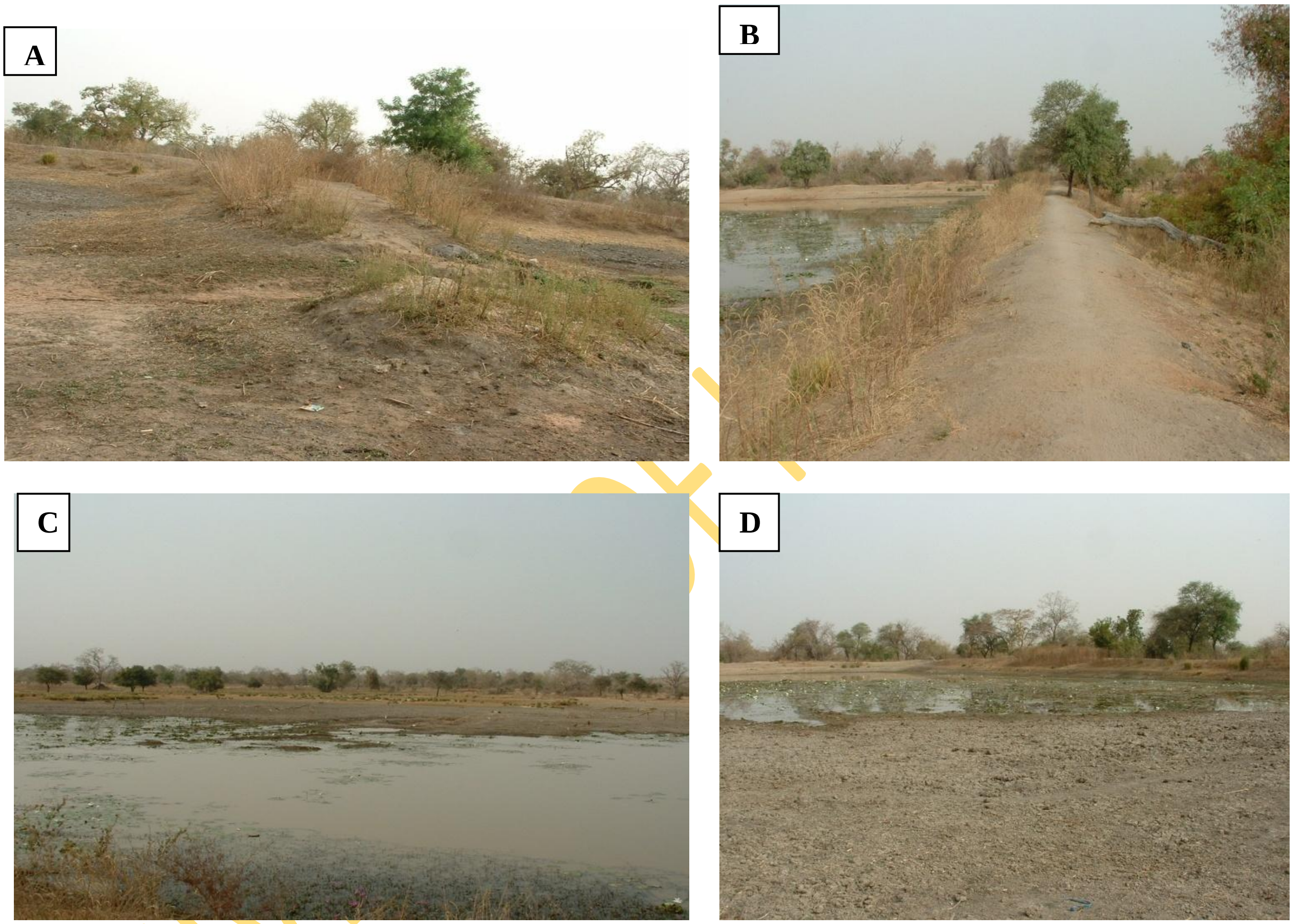


Fig 4.26; Nasarawo Gongoshi Grazing Reserve Distressed Dam1

A; Poorly maintained embankment with excessive erosion and no definite spillway

B; Trees growing on embankment, people and animals move on the bare embankment

C; Progressive siltation of the reservoir

D; Land use enhances siltation resulting in reduced storage capacity of the reservoir

4.7.4 SBGR Dam 4

The maximum temperature of the catchment can be up to 42^0C with an evaporation of 299.48mm/month which is more than the rainfall of 244.6mm/month. Although the reservoir is recharged by runoff from the surrounding hills, a lot of water is lost through evaporation.

The embankment soil is a non plastic well graded sand (SW) with CBR values of 28% and permeability, k of 1.7×10^{-8} m/s indicating a stable and impermeable embankment but slightly erodible (Foster, 2000; Foster et al, 2000 Singh, 2001and Murthy, 2008).

The embankment is poorly maintained, with trees and shrubs cover on it. The reservoir is highly silted and embankment is highly eroded at different sections. Animals graze and trample on the embankment further aggravating the erosion problem. The dam may be undermined when the reservoir capacity is directed towards the highly eroded embankment and silted reservoir. Sustained reservoir loading may lead to complete collapse of the dam (Figure 4.27A-C).

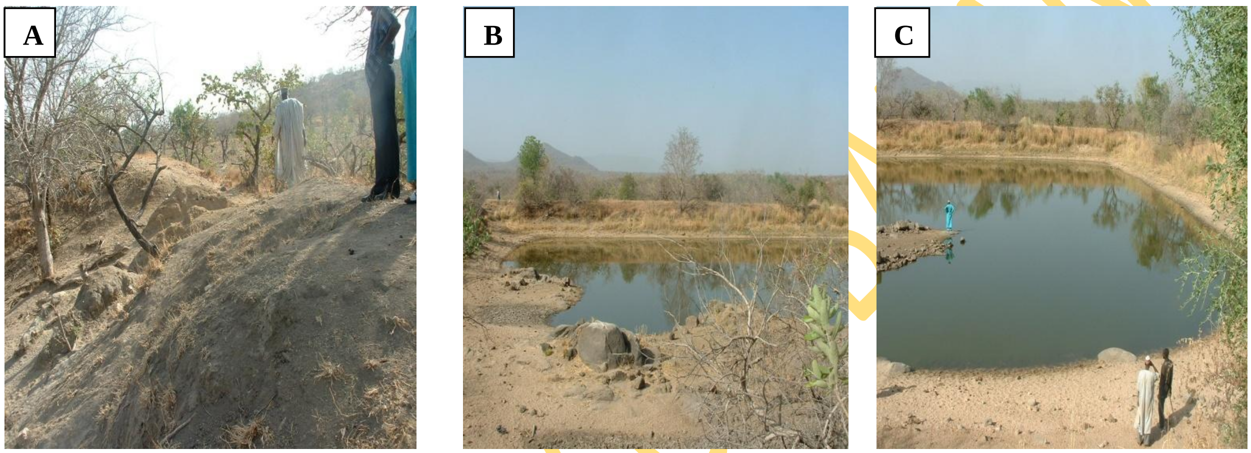


Fig 4.27; Sarau Belel Grazing Reserve Distressed Dam4

A; **A poorly maintained embankment with trees on top, excessive erosion & human and animal traffic aggravating the situation**

B; **Siltation of reservoir and absence of a definite spillway**

C; **Reduced reservoir capacity as a result of siltation**

4.7.5 Sarau Belel Grazing Reserve Dam 1

Having a maximum average monthly temperature of 43°C, a monthly total rainfall of 437.8mm, total monthly evaporation of 354.64mm and a Relative Humidity of 77% the operating conditions of the reservoir is going to be difficult as far as water conservation in the reservoir is concerned. There is high tendency for the reservoir to loose water due to high temperatures and an equally high evaporation.

Due to logistic reasons, the soil sample could not be picked for analysis.

The dam is still functional and has some water in the reservoir. There is excessive erosion on the embankment near the right abutment from crest level. The reservoir is heavily silted. The embankment is generally poorly maintained. If nothing is done, the dam will fail hydraulically due to progressive siltation and reduction of the height of the embankment as a result of erosion and settlement. Embankment height was increased sometime ago to compensate for embankment settlement and reservoir siltation.

4.7.6 SBGR Dam 4

The dam is situated on the basement complex. The maximum temperature of the catchment can be up to 42^0C and an evaporation of 299.48mm/month more than the rainfall of 244.6mm/month. Although the reservoir is being recharged by runoff from the surrounding hills, a lot of water is lost through evaporation.

The embankment soil is a non plastic well graded sand (SW) with CBR values of 28% and permeability K of 1.7×10^{-8} m/s indicating a stable and impermeable embankment but slightly erodible (Foster, 2000; Foster et al, 2000; Singh, 2001 and Murthy, 2008).

The embankment is poorly maintained, with trees and shrubs cover on it. The reservoir is highly silted and embankment is highly eroded at different sections. Animals graze and trample on the embankment further aggravating the erosion problem. The dam may be undermined when the reservoir capacity is directed towards the highly eroded embankment and silted reservoir. Sustained reservoir loading may lead to complete collapse of the dam.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

A sample of 42 earth dams were selected and investigated in North-Eastern Nigeria to find out the reasons for failures and distresses of such dams. The study concludes as follows:

1. Generally there is lack of information and data on failures and distresses of earth dams in the study area.
2. Of the dams under study, eleven (27%) were found to have failed, five (12%) were distressed, five (12%) were under construction and twenty one (49%) were functional.
3. Most of the dams are small with few medium and fewer large dams. Most of the failures and distresses affected the small dams. Small dams were designed and constructed haphazardly.
4. Most of the dams in the study area are of homogeneous type with very few zoned embankment types. Most of the failed and distressed dams are of homogeneous embankments types (90%). Only 10% of zoned ones failed, hence, the zoned embankments are more stable than the homogeneous types.
5. The geology of the study area comprises 11 formations. The basement complex formation covers most of Adamawa and Bauchi and small parches in Gombe. The geology fo Gombe consist of Bima sandstone, Pindiga formation, Kerri-Kerri formation and Gombe sandstones. Most of the formations are relatively impervious, hard and stable and good for supporting dams and their reservoirs, except the Kerri-Kerri and Alluvium formation which are loose and coarse, with high porosity which can allow reservoir water to escape easily and weaken the foundations.

6. The climate of the study area is characterized by low rainfall with seasonal variability frorm year to year. The temperatures are relatively hot and also vary seasonally from year to year. Evaporation rate which are high and relatively higher than rainfall, also vary seasonally from year to year. The relative humidity is low and also vary seasonally from year to year. About 80% of the failures and distresses occurred during peak rainy season. The dry season with high temperatures and low relative humidity is characterized by high evaporation which accounted for 20% of failures.
7. The failure modes exhibited include hydraulic (50%), seepage (5%), structural (1%), piping (8%). Hydraulic, seepage and structural and a combination of two or more of the above, interacting in a complex manner accounted for 36% of the failures.
8. The main causes of failure include; inadequate maintenance (71%), lapses in design (9%) and poor construction (15%) among others. Mode of construction also affected the failures and distresses of earth dams in the study area with direct labor accounting for greater failures and distresses.
9. The failures and distresses were not caused by the engineering properties of the soil alone, but in conjunction with geological factors, hydrometreology, design, construction and maintenance issues.
10. Lack of and poor maintenance, embankment erosion, reservoir siltation and inadequate spillways are the major reasons that led to distresses of earth dams in the study area.

5.2 Recommendations

(A) Before constructing earth dams, adequate feasibility studies should be carried out on the project area. These should include hydrometeorology, geology and soil among others.

(B) Design should be based on the results of the feasibility study carried out.

(C) Projects should not be commissioned before they are fully completed.

(D) Experts from all the relevant areas must be involved in the planning and development of the project.

(E) Engineering procedure of project conception, implementation operation and maintenance should be strictly adhered to.

(F) There should be a well designed and constructed spillway.

(G) Construction should be strictly based on the design specifications and standards.

(H) The downstream slope should be protected against rainfall erosion by heavy gravel or rock riprap. Sod may also be provided to guard against erosion if the rainfall is sufficient to grow and maintain grasses.

(I) If highly permeable material is to be used at all in constructing the dam, it will be used at the outer parts of the dam to aid drainage as a fill. In particular attention must be given to the use of impervious materials in the core.

(J) The embankment height should be such that water cannot over-top it.

(K) The seepage line should be well within the downstream face the dam. This is to prevent sloughing and possible failure.

(L) Water passing through or under the dam should not be strong enough to remove materials of the dam or the foundation

(M) There should be no opportunity for free flow of water from upstream to downstream face.

(N) Well equipped and adequate dam safety monitoring team should be on site all the time.

(O) Log books should be provided to enhance accurate record taking as well as record keeping

(P) The site monitoring team should be well trained and be sent to refresher courses from time to time.

(Q) All the instrumentation facilities should be well maintained to avoid malfunctioning.

(R) Embankment should be maintained, trees and shrubs be removed, reservoirs should be desilted and spillways should be adequately designed and placed. Appropriate land use activities should be encouraged upstream. The embankment heights should be increased to compensate for erosion and siltation of the embankment and reservoir respectively.

(S) Slope protection in form of turfing (grasses) surd or rock rip rap should be applied on both the upstream and downstream slopes of embankments. Ant's and termite's infestation should be treated and removed from the embankments. Animal fence should be placed around the whole length of the embankment to avoid trampling. Access roads to the dams should be provided to facilitate maintenance.

(T) Adequate compaction should be done to all the specific layers at all stages of the embankment construction. Adequate moisture should be applied between old and new surfaces for proper binding and uniformity of compaction. The surfaces of old and new layers should be properly cleaned before the application of appropriate moisture and subsequent compaction to desired level.

5.2.1 Scope for Further Research

The following areas would need to be investigated to complement the present study:

(I) The investigation of failures and distresses should be extended to cover the entire country. This would offer adequate information on failures, distress and performance of earth dams. This would also reveal more elaborate reasons for failures and distress.

(II) Some soil tests should be done in-situ so as to get precise information on the embankment, reservoir and spillway.

(III) Boreholes should be dug at the vicinity of the foundations to specifically characterize the geologic formations at the specific locations of dam sites. This would elaborate the investigation of the foundation materials and geology in relation to status.

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APPENDICES

APPENDIX I

COVER LETTER AND QUESTIONAIRE DISTRIBUTED

DEPARTMENT OF AGRICULTURAL AND ENVIRONMENTAL ENGINEERIG

UNIVERSITY OF IBADAN

Dear Respondent,

I Ahmadu Umaru Babayi (Matric. No. 109913) a Doctoral Research Student in the University of Ibadan, Department of Agricultural and Environmental Engineering, do hereby request that you please assist me with the information needed in this Questionnaire to enable me conduct a thorough study. The information so given will be used for research purposes only and will be treated with outmost confidentiality.

Thank you.

Section A; General Information

1. Name of Dam……………………………………………………………………………
2. Owner…………………………………………………………………………………..
3. Year of Construction………………………………………………………………...
4. Mode of Construction; (a) Direct labor (b) Contract
5. Embankment type; (a)Homogeneous (b) Zoned (c) Rock-fill
6. Condition of Dam; (a) Functional (b) Damaged (c) Failed (d) Under construction
7. If the answer to Question 6. is (c) what was the year of failure……………………………
8. Was there any loss of life or property? (a) Yes (b) No. If the answer is (a) what was the;

   (i) Estimated number of lives lost…..

   (ii) Estimated value of Properties lost in Naira…………………...
9. What was the mode of failure? (a) Hydraulic (b) Seepage (c) Structural (d) a & b above

   (e) a & c above (f) b & c above (g) All of the above (h) Others, please specify…………

   ………………………………………………………………………………………
   ………………………………………………………………………………………
10. The causes of failure are associated with; (a) Feasibility studies (b) Design (c) Construction (d) Maintenance (e) a & b above (f) a & c above (g) a & d above (h) b & c above (i) b & d above (j) c & d above (k) All of the above (i) Others, please specify……………………………………………………………..

    ………………………………………………………………………………………

    ………………………………………………………………………………………
11. What were the most likely causes of the failure? ....

    ………………………………………………………………………………………

    ………………………………………………………………………………………
12. Please mention what could have been done to avoid the failure…………………………

    ………………………………………………………………………………………

    ………………………………………………………………………………………

    ………………………………………………………………………………………

Section B; Design and Construction

13. Height of dam…………………………………………………………………………………
14. Length of dam…………………………………………………………………………………
15. Crest width…………………………………………………………………………...…..
16. Reservoir capacity……………………………………………………………………..
17. Design life of reservoir……………………………………………………………………..
18. Predominant embankment soil material……………………………………………………
19. Upstream slope……………………………Downstream slope……………………………
20. Number of zones and the soil materials in the zones……………………………………

    ……………………………………………………………………………………………………
21. Construction method…………………………………………………………………………..
22. Periods of construction; (a) Rainy season only (b) Dry season only (c) Both a & b above.
23. Compaction density per layer ……………………………………………………………
24. Thickness of layers for compaction……………………………………………………
25. Type of equipment used for compaction………………………………………………
26. Number of passes to achieve desired level of compaction per layer………………………

Section C; Operation and Maintenance

27. How well is the reservoir water utilized? (a) Well utilized (b) Under utilized
28. The spillway is ; (a) Functional (b) Damaged
29. The spillway is ; (c) Usually fully open (d) Usually Closed
30. How often is the embankment cleared of shrubs, trees, termites, ants, rats etc?
    (a) Regularly (b) Not regularly (c) Not at all
31. Is there any maintenance schedule for the reservoir? (a) Yes (b) No
32. Are there any safety instrumentations in place? (a) Yes (b) No
33. Types of safety instrumentations in place………………………………………………
34. Condition of safety instrumentations; (a) Functional (b) Damaged (c) Some functional & some damaged
35. Is there a dam safety and monitoring team in place; (a) Yes (b) No.
36. If yes how equipped? (a) Well equipped (b) Poorly equipped
37. Does the team go for training to update skills? (a) Yes regularly (b) Yes but not regularly (c) No at all.

APPENDIX II

WEATHER OF THE STUDY AREA

Table A1: Monthly Mean Maximum Temperature (^{O}C); Adamawa State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1982	35	37	39	40	38	35	32	31	31	34	35	35
1983	30	37	38	42	40	33	31	32	32	34	37	36
1984	34	37	40	39	34	34	32	32	32	34	36	34
1985	37	36	40	39	37	34	31	31	31	35	37	35
1986	35	39	40	41	36	34	31	31	32	33	35	33
1987	36	38	39	42	41	35	34	31	32	33	37	35
1988	35	38	40	40	37	33	32	31	30	34	36	33
1989	31	34	39	40	39	33	31	31	32	34	37	34
1990	36	36	38	41	35	34	30	30	30	33	34	32
1991	31	37	39	37	32	32	30	30	31	32	35	34
1992	31	35	39	43	37	34	32	31	31	31	35	35
1993	33	37	39	40	36	35	32	30	32	34	37	35
1994	35	37	32	39	37	34	33	31	31	33	35	33
1995	34	39	41	41	37	34	34	31	33	33	34	35
1996	37	39	40	40	35	33	32	30	30	30	28	29
1997	30	29	35	33	31	30	29	31	35	34	37	36
1998	37	37	35	35	35	34	37	32	31	31	33	31
1999	31	34	37	36	30	32	29	29	28	29	32	30
2000	31	30	34	41	37	28	-	27	27	30	32	-
2001	35	37	39	34	33	-	33	31	32	34	36	36
2002	31	39	43	42	41	34	32	33	32	33	38	35
2003	38	41	43	43	39	35	34	33	33	35	38	38
2004	39	40	43	39	39	36	36	34	35	37	35	32
2005	31	38	41	41	35	33	30	29	31	33	36	35
2006	36	38	39	39	35	32	31	29	30	33	35	33
2007	31	37	38	38	35	32	31	29	30	32	33	32
2008	32	32	35	34	33	32	31	30	29	33	31	35
2009	36	38	40	38	35	33	32	31	30	32	34	35
2010	31	33	40	36	36	33	31	30	29	32	32	30

Source: UBRBDA, 2010

Table A2: Monthly Total Rainfall (mm); Adamawa State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total (MM)	No. Of rainy days
1982	3.9			39	63.8	116	260.7	211	234.8	32.1			961.2	66
1983				29	127	156	207.1	219	126.8	19.7			884.1	53
1984			25.9	87	138	76.2	242.6	183	168.3	54.5			970.9	53
1985			54	40	159	132	200.6	201	173.6	10.8			970.5	65
1986				19	155	107	312.2	118	78.7	98.6			900.6	65
1987			5	0.3	34.6	105	102.5	200	127.3	44.1			678.7	57
1988				21	138	168	202	187	312.5	55.3	12		1084	74
1989				49	174	88.3	132.5	438	81.5	19.2			982.2	62
1990				43	90.1	94.7	225.3	200	123.6	32.5			823.8	58
1991				54	217	100	164.3	215	86.9	24.9	15		861.8	69
1992			27.9	49	191	87.8	105.1	173	227.8	5.7			969.3	64
1993			14.9	60	143	111	218.8	176	186.2	73.7	2.1		983.4	71
1994				79	96.8	194	102.8	267	106	79.3			924.5	61
1995			3.7	38	102	181	1991	240	133.8	193			1081	69
1996			1.7	41	183	108	160	200	263.1	52.2			1010	74
1997				89	68.5	21.2	194.9	133	187.2	103			977.6	68
1998				51	61	97.7	264.9	137	355.2	55.7			1023	74
1999				8.5	141	137	138.2	245	264.4	193	TR		1113	73
2000				3.2	149	219	164.6	202	183.2	26.5			947.6	73
2001			TR	44	93.7	247	208.8	102	193	28.8			915.8	61
2002				13	44.1	119	93.6	83.3	249.2	53.5	0.3		656.7	68
2003				10	56.5	103	143.3	199	183.6	88.7			784.7	77
2004				12	117	118	114.6	225	150.6	62.7			800	60
2005				30	84.1	103	186.3	235	130.4	29.7			799.1	62
2006				29	63.9	120	135.8	173	227.8	15.7			764.9	71
2007				62	51.1	97.6	250.6	269	122.7	49.9			903.4	68
2008				20	115	115	152.9	194	174.8	37.1			808.9	64
2009				15	129	200	193.4	247	238.1	41			1063	72
2010	-	-		32	76.9	211	213.2	199	199	131	-	-	1064	72

Source; UBRBDA, 2010

Table A3: Monthly Total Evaporation (mm); Adamawa State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1982													
1983		247.4	234	310.2		158.6	136.7	126.8	123.4	166.7	185.5	241.7	2037.1
1984	229.2	256.4	284.6	265	166.5	138	139.6	150.5	147.7	157.7	209.3	228.3	2372.6
1985	240	243.3	293	233.4	204.3	152.9	126.6	123.6	126.4	168.8	201.6	194	2308
1986	210.5	229.1	276.2	273.7	219.8	172.7	125.2	130.5	128.4	162.1	169.2	181.5	2278.3
1987	214.2	229	269	333.5	298.8	178.3	163.3	124.1	130.3	167.1	182.8	197.6	2492.1
1988	216.5	267.6	304.6	262.5	217.7	146.5	129.4	115.2	101.7	162.9	189.6	186.9	2300.5
1989	220.1	242.2	318.7	289.6	194.5	149.5	119.2	109.1	127.5	173.8	198.6	192.9	2332.7
1990	214.7	230.6	312.8	270.2	205.9	142.7	110.9	131.5	135.6	164.7	179.1	209.3	2310.1
1991	235.7	257.7	300.7									152.4	944.43
1992	250.5	259.1	283.1	220.1	293.5	150.2	139.4	141.9	134.7	166.2	181	185.8	2407.2
1993	210.2	246.5	289.7	259.1	190.4	150	149.9	195.9	154.2	153.6	168	209.5	2377.1
1994	213.3	252.9	324.6	242.4	212.9	166.9	158.8	108.8	134.1	136.1	188.2	207	2345.9
1995	211.2	226	285	277.8	218.4	144.5	155.1	145.7	159.1	151	167.3	207	2347.9
1996	212	234.6	295.8	254.1	187.5	149	139.2	112.7	122.3	144.2	182.8	198.8	2241
1997	220.9	212	291.6	218.5	172.8	136.9	135.4	119.3	144	147.7	172	185.3	2157.4
1998	206.7	255.1	299.3	266.8	224.3	172	143.4	116.7	177.8	143.7	182.5	186.4	2314.7
1999	216.8	236.1	301.1	252.4	210.4	172.4	128	112.3	116.8	139.3	169.3	189.3	2244.6
2000	22.9	257.7	286.7										2483.9
2001													
2002			328.3	281.8	263.2	153.7	123.8	97.07	91.2	125.9	183.4	204.7	1853
2003	216.1	261	329.2	288.1	271.4	233.8	113.7	83.01	102.8	124.5	172.4	190.5	2285.6
2004	211.1	354.6	300.1	275.7	202.2	110	127.2	116	101.6	149.5	193.1	204.3	2347.4
2005	214.3	248.1	318.2	305.5	232.5	185.9	112.2	110.1	126.5	162.3	199.8	213.4	2428.6
2006	233	258.8	317.9	335.6	181.9	133	135.5	107.5	135.4	177.9	218.5	218.4	2453.4
2007	209.4	242.1	299.5	279.6	187	180.2	255.8	100.8	120.2	152.3	192.8	211	2430.6
2008	210.7	247.5	339.4	280.2	211.3	158.1	151.1	108.1	113.7	158.6	205.9	205.1	2389.6
2009	156.6	167.9	220.3	181.9	146.4	122.1	106.5	92.37	100.3	108.8	125.4	147.3	1675.9
2010	169.2	170.1	224.6	227	159.5	107.4	86.14	79.42	97.01	107.8	126.4	146.5	1692.7

Source; UBRBDA, 2010

Table A4: Monthly mean Relative Humidity (%); Adamawa State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1982	35	24	32	44	59	69	76	80	78	70	43	41
1983	38	44	44	42	49	68	76	77	78	68	38	30
1984	37	39	34	48	67	68	74	76	73	67	41	25
1985	26	16	40	47	62	70	80	79	77	61	39	32
1986	27	21	41	51	62	72	82	80	78	73	51	35
1987	31	24	38	28	46	68	74	81	81	67	39	31
1988	31	27	25	48	61	75	76	82	82	69	44	34
1989	18	17	16	27	49	61	66	37	69	49	26	25
1990	19	15	14	29	49	48	66	68	65	52	37	38
1991	20	18	21	28	48	57	66	70	NR			
1992	NR							66	NR			
1993	29	21	76	88	79	74	72	68	72	57	29	26
1994	27	NR	NR	35	47	55	NR			48	18	
1995	33	26	33	30	48	59	58	64	63	62	30	26
1996	45	41	50	64	69	81	79	86	87	79	48	47
1997	43	35	43	67	76	90	69	76	75	73	52	53
1998	39	23	22	44	60	68	78	80	81	72	38	35
1999	34	29	39	47	58	63	69	76	77	76	39	36
2000	38	35	24	NR	61	NR						
2001	NR											
2002	NR	41	53	68	56	66	75	77	79	70	42	30
2003	34	21	19	43	48	66	79	84	82	75	49	32
2004	27	20	19	47	64	75	74	79	79	69	47	27
2005	25	NR	27	40	59	66	79	80	77	68	39	30
2006	24		33	37	65	70	76	79	80	71	38	28
2007	27	21	28	47	68	70	74	82	79	72	51	26
2008	25	21	29	45	56	68	73	81	77	66	36	36
2009	27	22	26	47	62	71	75	79	80	74	46	26
2010	24	26	29	36	63	72	76	79	80	80	52	29

Source; UBRBDA, 2010. NR = No Record

Table A5: Monthly mean maximum temperature (^{o}c); Gombe state

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sept.	Oct.	Nov.	Dec
1983	28	34	35	41	40	35	31	32	33	35	36	35
1984	32	35	40	39	35	35	34					
1985												
1986					33	30	28	27	28	30	29	25
1987	26	28	34	38	39	35	34	33	32	34	34	33
1988	31	35	38	39	38	35	32	30	31	34	36	32
1989	29	32	38	40	36	35	33	32	33	35	36	33
1990	34	34	38	43	39	36	33	33	34	37	38	37
1991	34	39	40	40	35	36	32	31	33	34	33	32
1992	31	35	36	37	35	33	30	29	32	32	30	32
1993	29	33	38	40	40	35	31	29	30	33	36	35
1994	33	33	35	35	35	32	31	32	33	35	35	33
1995	34	35	39	39	40	36	34	31	33	34	36	35
1996	36	38	37	40	36	34	33	32	31	34	34	34
1997	36	34	37	37	36	34	32	33	33	35	36	35
1998	33	35	33	40	41	35	33	31	32	36	37	35
1999	36		42	42	38	35	32	30	31	32	36	35
2000	36	32	38	43	40	36	33	31	34	36	38	34
2001	35	35		41	39	35	34	32	33	35	38	37
2002	32	34	39	41	38	35	34	32	31	33	36	34
2003	35	38	39	42	40	33	32	31	32	35	37	35
2004	34	37	39	42	37	34	33	32	33	36	37	35
2005	32	36	42	42	38	36	30	31	33	34	36	36
2006	35	38	39	41	37	34	33	32	34	35	36	34
2007	31	35	37	38	38	33	32	31	32			
2008	33	35	42	41	38	34	31	29	33	35		37
2009	37	35		39	38	36	33	31	32	32	35	38
2010	37	37	38	42	38	34	31	31	33	35	36	

Source; UBRBDA, 2010

Table A6: Monthly Total Rainfall (mm); Gombe State

Year	Jan	Feb	Mar.	Apr.	May	Jun.	Jul	Aug.	Sept.	Oct.	Nov.	Dec.	Total	No rainy days
1983					37.5	164.5	138.6	127.0	227.8	0.0			695.4	50
1984				101.7	182.6	54.7	167.7	194.3	146.9	18.0			865.9	63
1985			12.3	0.0	117.1	86.1	214.7	173.4	66.3	13.5			683.4	57
1986			4.3	21.4	115.6	147.0	164.8	139.4	100.9	8.3	1.0		702.7	59
1987			TR	2.7	19.9	86.0	72.1	125.1	141.3	59.2			506.3	44
1988				27.0	55.9	77.6	165.4	276.3	303.9	50.0			956.1	63
1989				2.3	108.0	117.9	117.4	277.1	174.1	36.2			833.0	67
1990				15.5	98.9	112.3	61.9	120.0	160.4	30.4			599.4	56
1991			12.1	40.3	158.9	116.2	199.2	194.3	90.2	44.7			855.9	38
1992			1.6	14.6	86.0	50.3	264.5	181.2	75.9	63.7	12.5		750.3	58
1993				31.1	108.3	64.1	93.1	182.5	150.1	29.9			659.1	NR
1994				41.5	73.7	157.6	168.9	285.5	132.3	28.0			887.5	57
1995			1.0	26.2	39.5	57.8	236.6	205.3	207.8	16.5			790.7	57
1996				4.10	136.7	264	116.7	251.3	82.2	58.0			913.0	
1997				72.3	229.7	115.9	224.8	82.2	162.2	41.5			928.6	
1998				0.2	60.2	180.9	172.3	173.00	192.80	49.40			828.8	
1999				44.40	106.90	86.70	142.90	264.30	128.10	84.60			857.9	
2000				36.50	88.40	206.10	262.40	264.80	246.40	14.00			1118.6	
2001				33.30	111.2	122.9	120.5	167.50	229.50	14.00			798.9	
2002				27.50	11.2	82.8	191.2	287.20	164.50	28.60			793.0	
2003				21.00	36.00	124.20	327.10	191.00	254.80	44.70			998.8	
2004				11.60	110.20	149.80	226.20	297.30	96.40	9.30			900.8	
2005				11.30	143.30	71.20	281.40	260.00	94.70	76.60			938.5	
2006				28.40	121.50	140.60	270.30	134.50	220.60	48.90			964.8	
2007					50.90	133.50	290.70	178.70	249.90				903.7	
2008				88.50	106.00	137.70	190.70	279.30	150.50	121.60			1074.3	
2009				55.80	55.40	299.00	133.70	214.60	142.90	153.60			1055.0	
2010			TR	41.70	88.80	214.20	230.20	106.50	107.90	108.00			897.3	

Source; UBRBDA, 2010

Table A7: Monthly Total Evaporation (mm); Gombe State

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1982			390.00	404.00	308.70	304.00	175.00	118.10	130.50	114.80	158.00	217.30
1983	231.3	290.40	409.70	381.50	3740.00	266.10	175.60	164.60	149.60	199.10	207.80	232.40
1984	240.6	269.90	341.10	275.60			162.10					
1985												
1986				338.70	305.20	265.30	187.20	137.10	146.70	179.70	204.10	231.00
1987	230.2	250.60	307.10	333.40	369.90	263.10	239.20	200.60	168.70	162.20	178.60	185.40
1988	229.7	275.60	339.00	310.00	305.90	210.50	190.14	121.20	134.76	188.88	209.14	217.42
1989	260.52	263.30	315.66	343.28	259.56	158.32	170.38	151.44	158.08	162.89	170.20	163.19
1990	194.78	252.31	316.00	228.00	283.50	175.36	189.05	183.54	148.32	109.06	153.88	210.23
1991	233.28	218.12	262.46	228.28	186.43		154.64	166.91	163.34	148.49	115.94	161.32
1992	168.94	201.94	202.26	270.31	200.46	211.55	238.39	211.15	203.76	197.73	241.72	173.34
1993	213.78	242.71	270.10	191.62					224.54	233.50	240.30	291.74
1994	272.57	274.50	200.76	297.81			264.60	395.54	176.46	61.98	329.64	254.10
1995	193.56	178.97	279.60	311.51	334.92	191.80	194.06	152.92	174.56			
1996						230.37	178.14	134.35	159.71			163.36
1997	224.080	228.46	274.27	336.63	311.53							
1998					342.00	334.60	333.74	266.65	232.06	229.00	249.00	
1999						241.98	289.78	271.80	1995.60	173.59		
2000						181.65	202.27	210.81	205.75			
2002	254.26	267.62	396.10	276.83	321.67	255.04	195.87	176.24	157.62	160.43	214.75	219.01
2003	236.47	295.40	354.10	362.20	334.47	201.87	172.87	147.62	146.07	155.52	207.16	210.35
2004	240	281.59	353.80	316.95	276.44	200.34	166.94	158.42	142.58	201.90	194.06	225.32
2005	231.02	282.94	351.03	333.76	308.67	184.34	108.84	185.00	187.51	150.90	214.54	213.62
2006	216.49	269.38	314.50	316.72	211.89	203.61	193.31	137.00	174.80	134.18	245.10	217.02
2007	295.2	277.67	333.05		1799.81	289.87	214.81	176.31	168.61			
2008	263.89	282.54	327.32	271.21	207.60	220.49	184.18	202.85	268.09	158.10		198.29
2009	151.64	238.55		286.78	182.08	192.30	186.09	149.07	158.10	180.10		
2010	177.35	265.01	263.07	252.29	279.91	214.27	279.42	115.58	129.05	143.21	245.55	

Source; UBRBDA, 2010

Table A8; Monthly Relative Humidity (%); Gombe State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1982			44	55	72	62	70	74	72	72	32	22
1983	22	15	15	20	43	62	72	71	73	47	26	32
1984	27	16	25	42	67	57	65					
1985												
1986				45	53	63	72	75	75	57	34	19
1987	20	19	19	26	40	57	62	70	73	59	32	29
1988	25	22	22	43	53	61	70	81	80	60	37	33
1989	31	28	28	35	58	66	71	76	74	61	33	30
1990	24	18	18	26	48	65	71	70	58	66	36	45
1991	29	29	29	50	68	67	66	76	74	70	36	26
1992	31	27	27	39	42	50	56	71	69	68	55	45
1993	28	30	30	23	44	50	64	69	72	55	40	39
1994	29	22	22	39	50	48	65	67	69	70	42	29
1995	23	27	27	41	58	56	66	74	70	65	40	31
1996	27	26	26	36	58	65	69	68	74	63	35	29
1997	24	20	20	42	61	61	67	65	67	62	50	29
1998	31	24	21	33	45	58	62	76	77	67	41	34
1999	32		29	34	44	59	58	77	76	74	53	37
2000	35	21	25	42	51	70	73	77	74	62	37	35
2002	37	37	41	55	64	61	70	76	76	64	44	33
2003	37	23	27	42	58	69	72	76	77	73	54	27
2004	24	32	37	50	62	72	74	79	80	62	46	26
2005	27	31	32	49	59	70	77	77	76	72	45	53
2006	52	51	45	55	73	72	72	75	76	68	37	37

Source; UBRBDA, 2010

Table A9; Monthly Average Maximum Temperature (^{O}C); Bauchi State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1980	32.3	38.1	36.6	37.8	34.6	31.5	29.2	29.1	30.6	32.8	33.1	29.8
1981	29	33.3	36.4	37.8	34.2	32	28.8	29.6	30.4	33.2	30.8	31.8
1982	30.7	32.5	35.5	37.2	34.4	32.3	29.9	29.2	29.8	32.4	31.1	31.7
1983	24.9	34.1	34.5	38.1	37.2	31.8	30	29.8	30.6	33.1	33.7	32.4
1984	30	33	37.6	37.3	34.2	33.9	30.3	30.7	30.8	32.8	33.6	29.9
1985	32.9	31.1	36.5	36.3	36.5	32.6	29.3	30.3	30.7	34	34.3	29.8
1986	30.8	35.3	37	38.8	36.6	32.9	30.2	29.8	30.5	32.9	33	28.9
1987	32.3	34.6	36.6	37.7	38.4	33.5	32.2	31	32	33.5	33.9	31.6
1988	28.4	33.6	37.4	38.1	36.8	32.6	29.9	28.9	29.9	33	33.3	29.8
1989	27.1	30.1	36.1	38.3	34.7	33.5	31	30.1	30.1	31.6	32.6	30
1990	32.3	31.4	34.8	39.3	36.4	34.7	30.3	30.4	31.7	33.3	34.6	33.7
1991	30.5	36.1	36.7	36.7	32.2	32.5	30.2	28.7	32	33.6	33.7	29.8
1992	27.9	31.9	36.5	37.3	35.3	32.1	29.8	28.3	29.8	32.9	31.4	31.3
1993	28.3	33.5	36.4	38.5	36.6	32.8	30.3	29.2	31.3	33.3	34.7	30.9
1994	30.8	32.9	38.2	38.8	36.2	34	31.1	30.1	30.6	31	32.5	31.3
1995	32	32.4	38.3	38.9	36.5	34.1	31.3	30.2	30.9	31.1	32.6	31.4
1996	33.5	34.7	38.5	38.9	36.5	32.7	31.5	30.5	30.7	32.6	32.2	32.8
1997	33	34.2	38	38.4	36	33.2	32.8	32.2	32.3	33.7	34.7	32.8
1998	31.1	34.7	35.5	40	35	34.9	33	28	33.2	33.1	33	33.2
1999	33.1	34.6	37.2	38.2	35.9	34	31.5	29	30.4	31	33.2	30
2000	32.4	30	36	29	38	29	31	29.3	31	33	34	30.8
2001	30.8	32	37	37	36	31.9	29.9	29.3	29.9	32.5	34	30.9
2002	28	33.6	37.2	38	38	34.9	32	30	30.1	31.7	33	31
2003	32	35.4	36.1	37.1	35.4	33.3	31.1	29.5	30.8	32.7	33.3	31.4
2004	30.3	33.5	36	38.3	34.9	32.7	30.5	29.7	30.7	32.7	33	33
2005		35.8	36.7	37.6	35.7		29	29	31	33	34	33
2006	34	36	37.1	38.4	34.9	32.1	31	29.4	30	32.9	32.5	31.1
2007	31.0	33.1	36.8	37.2	35.8	32.3	30.1	28.4	30.5	32.5	34.3	32.1
2008	28.5	30.9	38.0	37.2	35.8	32.7	30.1	28.8	30.6	33.6	34.2	32.8
2009	33.6	35.8	37.8	37.7	35.5	33.1	31.3	30.0	31.0	31.8	32.2	32.7
2010	33.2	36.6	37.4	39.9	36.7	32.5	30.2	29.9	30.2	32.2	34.4	31.8

Source; NIMET Abuja, 2012

Table A10; Monthly Total Rainfall (mm); Bauchi State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1980	0	0	0	0.5	111.6	121.3	314.1	317.9	82.5	40.3	0	0
1981	0	0	0	28.2	120.2	246.3	396.3	268.9	172.5	18.4	0	0
1982	0	0	0	43.8	38.4	164.9	233.2	239.1	146	31.6	0	0
1983	0	0	13	0	95	123.5	227.1	184.2	130.5	0	0	0
1984	0	0	6.8	19.3	148.7	77.8	239	227.8	164	10.3	0	0
1985	0	0	33.2	0	122.3	108.4	152.2	162.2	145.6	1.7	0	0
1986	0	0	5.6	8	40	129.5	342.8	171	228.9	20.3	0	0
1987	0	0	0	0	56.3	219.3	151.1	240.4	31.2	46.3	0	0
1988	0	0.5	1.8	66.6	90.9	143.3	173.3	276.1	159	9.2	0	0
1989	0	0	0	26.6	100	77.6	182.3	324.7	140.8	57.4	0	0
1990	0	0	0	5.9	108.7	100.5	284.7	262.3	87.8	29.7	0	0
1991	0	0	28.5	85.9	149.1	103.3	283.1	244.7	35.9	19.1	0	0
1992	0	0	2.4	49.4	51.2	177.5	328	357.6	233.5	28.1	2.7	0
1993	0	0	0	14.9	81.3	236.1	231.8	337.7	179.5	60.6	0	0
1994	0	0	0	106.9	58.4	56.4	225.3	280.4	295.7	44.2	0	0
1995	0	0	0	27.2	104.8	197	123.1	277.1	212.8	19.4	0	0
1996	0	0	0	18.1	95	151.4	237.2	341.4	265.5	32	0	0
1997	0	0	0	30.2	38.7	178.9	240	213.6	178.7	16.6	0	0
1998	0	0	0	11.1	138.3	153.8	303.1	324.9	183.6	7.8	0	0
1999	0	0	0	7.9	41.2	118.8	440	344	262.4	186.3	0	0
2000	0	0	0	10.3	80.5	219	251.8	308.4	168	20.9	0	0
2001	0	0	0	37.1	155.7	234.4	324.8	354	200.2	1.2	0	0
2002	0	0	0	76.9	26.6	112.2	155.4	238.9	192.9	16.2	0	0
2003	0	0	0	31	73	295	124.4	262.5	173.9	29.7	0	0
2004	0	0	1.9	33.8	87.8	277.7	267.4	138.3	54.9	4.1	0	0
2005	0	0	0	24.8	91.8	225	157.9	344.2	166.5	24.3	0	0
2006	0	0.7	0	10	157.5	195.9	241.8	229.5	146.5	36	0	0
2007	0	1.3	2.6	37.6	17.3	122.3	276.2	520.9	162.3	7	0	0
2008	0	0	0	0	86.6	351.6	209.5	388.8	57.1	39.7	0	0
2009	0	0	0	82.6	106.2	184.8	211.6	403.8	288.9	173.4	0	0
2010	0	0	0	36.8	74.8	200.4	379.8	219	489.3	146.9	0	0

Source; NIMET Abuja, 2012

Table A11; Monthly Average Evaporation (mm); Bauchi State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1980	11.4	12.9	13.8	14	7.2	4.3	2.6	2.9	3.2	4.9	9.2	9.9
1981	10.5	13.6	15.1	13.4	6.4	4.5	2.5	2.6	2.8	5.4	9.5	10.1
1982	10.1	15.7	14.9	10.3	6.6	4.1	2.6	2.1	2.7	4.6	8.3	9.7
1983	10.2	14.3	15.7	15.1	9.6	4.1	3.1	2.3	3.1	7.6	9.6	9.8
1984	10.4	13.1	14	11.7	6	6	3.3	3.1	3.1	5.7	10.2	9.1
1985	11.4	13.3	12.2	12.9	7.8	5.5	2.8	2.7	3.3	8.6	9.7	9.1
1986	9.1	11.6	11.8	11.3	7.7	4.7	2.4	2.5	2.5	5.5	8.1	8.4
1987	9.2	11.4	11.6	13.8	11.6	5.4	3.4	3	3.5	6.2	9.6	8.7
1988	9.5	11.4	13.6	9.6	7	4.2	2.3	2.1	2.5	6	7.9	7.8
1989	8	9.5	12.1	10.5	6.4	5.3	3.3	2	2.7	4.7	7.8	7.3
1990	8.7	9.6	12.1	10.2	6.3	5.1	2.5	2.8	3.4	5.7	8.7	8.5
1991	9	11.2	12	8.3	3.8	3.3	2.5	2	3.7	5.9	9.4	8.9
1992	9.2	11.8	12	9.8	6.5	3.7	2.6	2	2.5	4.8	7.5	8.2
1993	8.1	10.6	12.7	11.3	8.3	4.4	3	2.6	3.4	5.1	5	8.3
1994	9.5	11.1	13.9	8.7	7.5	5.1	3	2	2.3	4	8	8.4
1995	8.5	10.4	12.3	10.6	6.9	5.1	3.4	1.9	3	5.7	9.2	8.9
1996	10.4	12.9	15.6	15.3	8.8	5.6	2.5	2.4	2.6	3.4	6.8	7.2
1997	8.4	9.2	11.2	10.5	9.3	5		3	3.3	4.9	9	8.9
1998		11.4	13.4	9.3	6.2	4	2.4	2	3	5.9	9.1	
1999	8.5	10.9	13.8	9.6	9.4	6.9	4.1	3	3	4.8	8.6	9.4
2000	9.6	11.9	13.4	10.9	10.7	4	3.3	2.7	2.9	5.5	16.2	9
2001	9.8	11.7	12.9	10.5	6.2	3.9	3.1	0.9	3.8	7.2	10.4	10.5
2002	10.3	12.9	14.6	13.3	8.1	4.8	3.1	2.6	3.3	6.1	17.4	9.9
2003	10.5	12.3	11.6	10.1	5.7	5.2	4.1	2.1	3.5	7.3	10.4	9.8
2004	14.6	11	12.7	10.1	6.3	4.9	3.2	1.8	3.1	5.8	8.2	8.1
2005	8.1	9.7	12.3	10.3	7.9	4.5	2.9	2	3.1	5.9	8.4	8.3
2006	10.5	13.6	15.1	14.4	9.2	5.7	3	2.5	3	7.3	10.1	9.8
2007	10.8	13.1	14.4	10	6.7	4.8	3.1	2.5	3.2	5.9	9.4	9.4
2008	11.6	14	15.9	13.2	9.9	6.4	3.5	2.2	2.8	6.5	9.7	10.3
2009	10	13.1	15	11.5	7.7	5.4	2.4	2.2	2.6	4.8	8.9	9.1
2010	9.3	12.8	15	11.1	7.1	5.2	2.6	2.3	2.7	5.5	8.8	9.8

Source; NIMET Abuja, 2012

Table A12; Monthly Average Relative Humidity (%); Bauchi State

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1981	27.5	20.5	27.7	39.0	62.3	71.0	79.4	78.2	74.7	57.7	34.0	25.7
1982	27.6	23.8	23.4	46.9	61.3	69.4	77.2	78.7	76.1	64.4	31.9	28.2
1983	28.8	23.5	24.0	36.1	57.1	73.8	78.4	80.3	74.9	43.4	34.0	42.2
1984	42.8	28.0	38.4	47.0	64.6	64.1	76.3	75.8	75.7	62.0	40.5	33.7
1985	31.7	18.4	33.7	29.5	58.2	67.0	77.2	80.4	75.2	48.7	35.9	35.2
1986	28.3	22.5	28.3	46.3	55.9	66.1	76.5	76.6	74.2	56.7	36.2	29.5
1987	26.5	23.8	27.6	20.6	44.1	63.7	72.8	76.1	71.0	53.9	32.6	27.3
1988	26.4	22.1	22.4	41.7	54.4	68.7	76.3	78.3	78.0	48.4	35.0	36.5
1989	28.9	22.1	21.0	36.9	59.0	62.8	70.9	79.2	73.0	56.4	34.4	34.8
1990	32.1	31.8	19.1	41.3	57.8	63.5	77.4	75.2	71.4	55.1	35.4	34.9
1991	25.5	25.2	29.3	50.1	71.8	72.2	76.6	80.8	70.8	57.1	29.2	30.1
1992	25.5	20.4	29.2	39.7	59.2	71.0	77.7	81.1	76.9	58.9	40.6	37.2
1993	32.2	25.2	27.9	36.1	54.4	68.5	76.5	80.1	71.4	58.4	37.2	30.8
1994	30.3	23.3	20.6	66.5	75.3	80.7	78.3	66.4	35.7	32.2	31.2	27.9
1995	28.7	31.2	24.4	40.2	56.9	65.0	73.7	79.4	76.7	61.9	39.0	40.3
1996	41.9	30.3	31.5	38.1	59.4	71.4	75.6	81.3	76.1	59.4	36.4	33.0
1997	35.9	31.6	57.4	58.8	70.0	73.7	76.5	71.2	65.9	54.3	36.3	30.7
1998	35.7	33.6	24.5	42.0	69.1	72.3	76.0	80.6	78.2	73.8	50.4	48.8
1999	46.4	47.7	50.9	53.4	62.2	79.6	81.3	84.7	82.1	76.1	53.9	46.8
2000	49.2	46.8	44.3	58.2	69.1	82.8	85.7	88.0	86.6	76.1	69.5	63.3
2001	70.3	51.9	18.7	42.6	61.1	69.5	78.7	80.4	77.5	51.1	29.3	30.2
2002	27.7	23.1	22.5	43.2	48.9	59.5	71.5	75.1	74.1	59.0	31.5	29.2
2003	27.5	24.8	18.8	43.8	41.2	72.1	75.1	77.3	75.1	61.4	35.6	27.6
2004	25.7	24.3	22.9	47.4	62.2	70.3	78.5	80.6	74.5	62.9	43.1	33.1
2005	26.7	31.7	35.1	46.5	64.2	73.8	78.4	81.0	74.9	58.8	36.8	32.1
2006	27.9	28.6	24.6	30.1	59.0	69.6	74.4	79.8	77.2	65.3	38.1	33.7
2007	30.4	25.4	26.5	45.8	54.0	61.4	70.8	86.7	69.7	76.6	50.4	39.9
2008	28.1	22.1	21.6	35.6	51.8	62.0	69.5	77.3	70.1	54.3	32.1	29.7
2009	21.4	17.9	18.0	33.7	49.2	61.5	72.3	77.3	73.4	52.8	30.8	27.1
2010	28.5	15.6	19.2	44.8	57.8	65.8	79.6	81.9	76.9	61.6	33.1	30.1

Source; NIMET Abuja, 2012

APPENDIX III

SOIL TEST RESULTS

Table B1: Specific Gravity Results (Gs)

S/N	Name of Dam	Soil Sample location	Specific Gravity(Gs)	Status
1.	Girei	SPLW	2.60	Failed
		EM	2.44	
		RSV	2.58	
2.	Guyaku GR Dam 2	RSV	2.63	Failed
		EM	2.63	
3.	Guyaku GR Dam 5	IN GULLY	2.44	Failed
		EM	2.60	
4.	Nzuzu Dam	SPLW	2.60	Failed
5.	NGGR Dam 1(Dalehi)	EM	2.41	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	2.60	Failed
7.	Ali Walga Dam	EM	2.63	Functional
8.	SBGR Dam 3	RSV	2.47	Distressed
9.	SBGR Dam 4	EM	2.63	Distressed
		RSV	2.53	
10.	Dadinkowa Dam	EM	2.59	Functional
11.	Bambam Dam	EMRSV	2.53	Failed
12.	Pindiga Dam I	EM/RSV	2.67	Functional
13.	Pindiga Dam II	EM/RSV	2.63	Functional
14.	Bojude	EM/RSV	2.70	Functional
15.	Jumbo Dam Dukku	EM/RSV	2.50	Functional
16.	Dukku Dam(Kogin Dole)	EM	2.60	Functional
17.	Cham Dam	EM/RS	2.66	Failed
18.	Waya Dam	EM(SHELL)	2.63	Failed (rptdly)
		EM(CORE)	2.60	
19.	Gubi Dam	EM(SHELL)	2.63	Functional
		EM(CORE)	2.63	
20.	Miri Dam	EM/RSV	2.63	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	2.70	Functional
22.	Dull Dam	EM(Left)	2.50	Failed
		EM(Rigth)	2.63	

KEY:
EM = Embankment
RSV = Reservoir
SPLW = Spillway

Table B2: Sieve Analysis (BS1377: 1990 Part 2:9.3)

S/N	Name of Dam	Soil Sample Location	Cu	Cc	USCS	Description	Status
1.	Girei	SPLW	3	1	SP	Non plastic poorly graded sand	Failed
		EM	3	1	SP	Non plastic uniform sand	
		RSV	2	1	SP	Uniformly graded sands of low plasticity	
2.	Guyaku GR Dam 2	RSV	4	1	SW	Non plastic well graded sand	Failed
		EM	3	1	SP	Poorly graded sand of low plasticity	
3.	Guyaku GR Dam 5	IN GULLY	5	2	SW	Well graded sands of low plasticity	Failed
		EM	3	1	SW	Well graded sand of low plasticity	
4.	Nzuzu Dam	SPLW	2	1	SP	Uniformly graded sand of low plasticity	Failed
5.	NGGR Dam 1(Dalehi)	EM	3	0.8	SP	Uniformly graded sand of low plasticity	Distressed
6.	NGGR Dam 2(Dalehi)	RSV	3	1	SP	Non plastic poorly graded sand	Failed
7.	Ali Walga Dam	EM	4	1	SP	Non plastic poorly graded sand	Functional
8.	SBGR Dam 3	RSV	2.1	1.4	SP	Poorly graded sand of low plasticity	Distressed
9.	SBGR Dam 4	EM	5.1	1	SW	Non plastic well graded sand	Distressed
		RSV	3.1	1	SP	Non plastic poorly graded sand	
10.	Dadinkowa Dam	EM	2	1	SP	Uniformly graded sand of low plasticity	Functional
11.	Bambam Dam	EM/RSV	3.1	0.78	SP	Uniformly graded sand of medium plasticity	Failed
12.	Pindiga Dam I	EM/RSV	4.6	2.1	SW	Well graded sands of low plasticity	Functional
13.	Pindiga Dam II	EM/RSV	15	0.6	SW	Well graded sand of low plasticity	Functional
14.	Bojude	EM/RSV	5	1	SW	Well graded sand of low plasticity	Functional
15.	Jombo Dam Dukku	EM/RSV	4	1	SW	Well graded sand of low plasticity	Functional
16.	Dukku Dam(Kogin Dole)	EM	2.5	1.2	SP	Non plastic uniform sand	Functional
17.	Cham Dam	EM/RS	5	1	SW	Well graded gravelly Sand of medium plasticity	Failed
18.	Waya Dam	EM(SHELL)	4	2	SP	Non plastic poorly graded sand	Failed (rptdly)
		EM(CORE)	4	1	SW-	Non plastic well graded	

					SM	silty sand	
19.	Gubi Dam	EM(SHELL)	5	1	SW	Non plastic well graded sand	Functional
		EM(CORE)	2.1	1.2	SP	Non plastic poorly graded sand	
20.	Miri Dam	EM/RSV	7.2	0.78	SW-SM	Well graded silty sand of low plasticity	Distressed
21.	Marraraba Ganye Toro Dam	EM/RSV	4	1	SP-SM	Uniformly graded sands of low plasticity	Functional
22.	Dull Dam	EM/RSV(Left)	3	1	SP-SC	Uniformly graded sand of low plasticity	Failed
		EM/RSV(Rigth)	4	1	SW	Well graded sand of medium plasticity	

KEY;

EM = Embankment

RSV = Reservoir

SPLW = Spillway

Cu = Coefficient of Uniformity

Cc = Coefficient of Curvature

USCS = Unified Soil Classification System

Table B8; Consolidation Test

Dadinkowa Dam – Status; Functional

LL – 43%, PL – 23%

Before Test	**After Test**
Initial Moisture Content = 24.1%	Final Moisture Content = 23.0%
Density = 1.87 Mg/m^3	Final Density = 1.97 Mg/m^3
Dry Density = 1.51 Mg/m^3	Final Dry Density = 1.60Mg/m^3
Initial Void Ratio = 0.76	Final Void Ratio = 0.662
Initial Saturation = 84.1%	Final Saturation = 92.4%

Void Ratio Change Factor = 0.0881

Degree of Saturation = 84.1%

Overall Settlement = 1.180mm

Cham Multipurpose Dam – Status; Failed

LL – 24%, PL – 15%

Before Test	**After Test**
Initial Moisture Content = 19.6%	Final Moisture Content = 15.5%
Density = 1.93 Mg/m^3	Final Density = 2.11Mg/m^3
Dry Density = 1.61 Mg/m^3	Final Dry Density = 1.83Mg/m^3
Initial Void Ratio = 0.609	Final Void Ratio = 0.415
Initial Saturation = 83.4%	Final Saturation = 96.7%

Void Ratio Change Factor = 0.08045

Degree of Saturation = 83.4%

Overall Settlement = 2.2646mm

APPENDIX IV

ENGINEERING PARAMETERS OF SOIL PROPERTIES

Table C1; Uniformity Coefficient (Cu)

Cu	Type of Soil
< 5	Uniform size particles
5 – 15	Medium graded soil
> 15	Well graded

Source; (Murthy, 2008)

Table C2; Plasticity Index (PI)

Plasticity Index (PI)	Plasticity
0	Non – Plastic
< 7	Low plastic
7 – 17	Medium plastic
> 17	Highly plastic

Source ; (Murthy, 2008)

Table C3; Coefficient of Permeability (k)

K (cm/sec)	Soil Type	Drainage Condition
10^{1} to 10^{2}	Clean gravels	Good
10^{1}	Clean sands	Good
10^{-1} to 10^{-4}	Clean sand & gravels mixture	Good
10^{-5}	Very fine sand	Poor
10^{-6}	Silt	Poor
10^{-7} to 10^{-9}	Clay soil	Practically Impermeable

Source; (Murthy, 2008)

$$Relative\ Compaction\ \% = \frac{Field\ \ Compaction}{Proctor\ \ Maximum\ \ dry\ density}\ \%$$ .... (Singh, 2001)

Table C4; Specific gravity of Soils (Gs)

Soil type	Specific gravity
Clean sands	2.67
Silty stained sands	2.67 – 2.70
Inorganic clays	2.70 – 2.80
Soil high in mica, iron	2.75 – 2.85
Organic soils	Quite variable; as low as 2.2

Source; (Singh, 2001)

Table C5; Coefficient of permeability (k) of various soils

Type of Soil	Coefficient of permeability (cm/sec)
Gravel	$10^{2} – 1.0$
Sand	$1.0 – 10^{-3}$
Silt	$10^{-3} – 10^{-4}$
Clay	Less than 10^{-4}

Source; (Singh, 2001)

Table C6; Comparative Engineering Properties of Soil Groups

Group Symbol	Permeability when compacted	Shear strength when compacted and saturated	Compressibility when compacted and saturated	Workability as construction materials
GW	Pervious	Excellent	Negligible	Excellent
GP	Very pervious	Good	Negligible	Good
GM	Semi-pervious	Good	Negligible	Good
GC	Impervious	Good to fair	Very low	Good
SW	Pervious	Excellent	Negligible	Excellent
SP	Pervious	Good	Very low	Fair
SM	Semi-pervious to pervious	Good	Low	Fair
SC	Impervious	Good to fair	Low	Fair
ML	Semi-pervious to pervious	Fair	Medium	Fair
CL	Impervious	Fair	Medium	Good to fair
OL	Semi-pervious	Poor	Medium	Fair
MH	Semi-pervious to Impervious	Fair to poor	High	Poor
CH	Impervious	Poor	High	Poor
OH	Impervious	Poor	High	Poor
PT	-	-	-	-

Source; (Singh, 2001)

Table C7; Approximate limits of ϕ' in cohessionless soil

Type of soil	ϕ'	
	Dense	Loose
Uniform sands	30	40
Well graded sands	32	45
Sandy gravels	35	50

Source; (Singh, 2001)

Table C8; Typical values for ϕ and ϕ_u for angular soils

Type of soil	Φ	ϕ_u
Sand; rounded grains		
Loose	28 to 30	
Medium	30 to 35	26 to 30
Dense	35 to 38	
Sand; angular grains		
Loose	30 to 35	
Medium	35 to 40	30 to 35
Dense	40 to 45	
Sandy gravel	34 to 48	33 to 36

Source; (Murthy, 2008)

Table C9; Typical CBR values and soil groups

Group symbol	Group name	CBR(%)
GW	Well graded gravel	40 - 80
GP	Poorly graded gravel	30 – 60
GM	Silty gravel	40 – 60
GC	Clayey gravel	20 – 40
SW	Well graded sand	20 – 40
SP	Poorly graded sand	10 – 40
SM	Silty sand	15 – 40
SM-SC	Silty sand and Clayey sands	-
SC	Clayey sand	5- 20
ML	Silt	15 or less
ML-CL	Silt - Lean clay	-
CL	Lean clay	15 or less
OL	Organic clay, Organic silt	5 or less
MH	Elastic silt	10 or less
CH	Fat clay	15 or less
OH	Organic clay, Organic silt	5 or less

Source; (Singh, 2001)

APPENDIX V

RESULT SHEET OF ANOVA (TEST OF DIFFERENCES) ON SOIL PROPERTIES

		N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
						Lower Bound	Upper Bound		
Liquid Limit (LL)%	Functional	10	24.7	8.447	2.671	18.66	30.74	15	46
	Distressed	5	22.2	7.95	3.555	12.33	32.07	9	30
	Failed	14	25.57	9.549	2.552	20.06	31.08	13	47
	Total	29	24.69	8.706	1.617	21.38	28	9	47
Plastic Limit (PL)%	Functional	6	18.67	3.724	1.52	14.76	22.57	15	25
	Distressed	3	16	2.646	1.528	9.43	22.57	14	19
	Failed	9	20.44	3.779	1.26	17.54	23.35	15	27
	Total	18	19.11	3.787	0.893	17.23	20.99	14	27
Plasticity Index (PI)	Functional	10	7.7	9.37	2.963	1	14.4	0	26
	Distressed	5	5.6	5.32	2.379	-1.01	12.21	0	11
	Failed	10	8.7	6.617	2.093	3.97	13.43	0	20
	Total	25	7.68	7.443	1.489	4.61	10.75	0	26
Moisture Content (%)	Functional	10	10.98	2.408	0.762	9.26	12.7	7	16
	Distressed	3	10.73	4.692	2.709	-0.92	22.39	6	15
	Failed	8	11.73	4.228	1.495	8.19	15.26	6	19
	Total	21	11.23	3.352	0.731	9.7	12.75	6	19
Bulk Density Mg/m3	Functional	10	2.02	0.158	0.05	1.91	2.13	2	2
	Distressed	3	2.04	0.106	0.061	1.77	2.3	2	2
	Failed	8	2.07	0.151	0.053	1.94	2.2	2	2
	Total	21	2.04	0.144	0.032	1.97	2.11	2	2
Dry Density Mg/m3	Functional	10	1.82	0.164	0.052	1.71	1.94	1	2
	Distressed	3	1.84	0.13	0.075	1.52	2.16	2	2
	Failed	8	1.86	0.183	0.065	1.7	2.01	1	2
	Total	21	1.84	0.16	0.035	1.77	1.91	1	2
CBR(%)	Functional	10	26.7	7.334	2.319	21.45	31.95	14	36
	Distressed	3	20.67	7.024	4.055	3.22	38.11	14	28
	Failed	8	21.88	10.616	3.753	13	30.75	11	46
	Total	21	24	8.701	1.899	20.04	27.96	11	46

		N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
						Lower Bound	Upper Bound		
Optimum Moisture Content (OMC)%	Functional	10	9.19	1.895	0.599	7.83	10.55	7	12
	Distressed	3	9.6	1.015	0.586	7.08	12.12	9	11
	Failed	10	10.38	1.652	0.523	9.2	11.56	8	13
	Total	23	9.76	1.734	0.362	9.01	10.51	7	13
Maximum Dry Density (MDD) Mg/m3	Functional	10	1.86	0.115	0.036	1.78	1.95	2	2
	Distressed	3	1.88	0.13	0.075	1.56	2.2	2	2
	Failed	10	1.8	0.123	0.039	1.72	1.89	2	2
	Total	23	1.84	0.119	0.025	1.79	1.89	2	2
Specific Gravity(Gs)	**Functional**	**10**	**2.63[a]**	**0.058**	**0.018**	**2.59**	**2.67**	**3**	**3**
	Distressed	**5**	**2.53[c]**	**0.097**	**0.044**	**2.41**	**2.65**	**2**	**3**
	Failed	**15**	**2.58[b]**	**0.069**	**0.018**	**2.54**	**2.62**	**2**	**3**
	Total	**30**	**2.59**	**0.076**	**0.014**	**2.56**	**2.62**	**2**	**3**
Moisture Content (%)	Functional	9	13.58	5.259	1.753	9.54	17.62	7	22
	Distressed	5	9.41	2.815	1.259	5.92	12.91	5	12
	Failed	12	10.56	5.345	1.543	7.16	13.95	1	19
	Total	26	11.38	5.052	0.991	9.34	13.42	1	22
Bulk Density (?)Mg/m3	Functional	9	2.12	0.196	0.065	1.96	2.27	2	3
	Distressed	5	2.05	0.152	0.068	1.86	2.24	2	2
	Failed	12	2.01	0.097	0.028	1.95	2.08	2	2
	Total	26	2.06	0.149	0.029	2	2.12	2	3
Dry Density (?d)Mg/m3	Functional	9	1.81	0.102	0.034	1.73	1.89	2	2
	Distressed	5	1.88	0.158	0.071	1.68	2.08	2	2
	Failed	12	1.87	0.193	0.056	1.75	1.99	2	2
	Total	26	1.85	0.157	0.031	1.79	1.91	2	2

Note: a>b>c

		N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean Lower Bound	95% Confidence Interval for Mean Upper Bound	Minimum	Maximum
Void Ratio	Functional	9	0.46	0.103	0.034	0.38	0.54	0	1
	Distressed	5	0.36	0.127	0.057	0.2	0.51	0	1
	Failed	12	0.39	0.131	0.038	0.31	0.47	0	1
	Total	26	0.41	0.123	0.024	0.36	0.46	0	1
Permeability (K) m/s	Functional	9	5.50E-08	5.46E-08	1.82E-08	1.30E-08	9.70E-08	1.23E-08	1.82E-07
	Distressed	5	3.08E-08	1.68E-08	7.51E-09	9.98E-09	5.17E-08	1.76E-08	5.65E-08
	Failed	12	7.26E-07	1.38E-06	3.99E-07	-1.53E-07	1.60E-06	1.21E-08	4.29E-06
	Total	26	3.60E-07	9.81E-07	1.92E-07	-3.63E-08	7.56E-07	1.21E-08	4.29E-06
Cu	Functional	10	4.82	3.749	1.186	2.14	7.5	2	15
	Distressed	5	4.1	2.051	0.917	1.55	6.65	2	7
	Failed	15	3.41	0.907	0.234	2.9	3.91	2	5
	Total	30	3.99	2.399	0.438	3.1	4.89	2	15
Cc	Functional	10	1.11	0.384	0.122	0.84	1.38	1	2
	Distressed	5	1	0.249	0.111	0.69	1.31	1	1
	Failed	15	1.12	0.362	0.094	0.92	1.32	1	2
	Total	30	1.1	0.346	0.063	0.97	1.22	1	2
Cohesion (C) KN/m3	Functional	8	62.88	21.676	7.664	44.75	81	40	100
	Distressed	2	65	7.071	5	1.47	128.53	60	70
	Failed	10	91.2	51.646	16.332	54.25	128.15	35	215
	Total	20	77.25	40.551	9.068	58.27	96.23	35	215
Angle of Internal Friction $(\Phi)^o$	Functional	8	18	14.071	4.975	6.24	29.76	7	50
	Distressed	2	11	8.485	6	-65.24	87.24	5	17
	Failed	10	13	7.149	2.261	7.89	18.11	3	23
	Total	20	14.8	10.416	2.329	9.93	19.67	3	50