ASSESSING THE FLOW CHARACTERISTICS OF AMANYI STREAM FOR A SMALL SCALE HYDROPOWER DEVELOPMENT

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL
FEDERAL UNIVERSITY OF TECHNOLOGY OWERRI

IN PARTIAL FULLFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF MASTER OF ENGINEERING DEGREE (M.ENG) IN CIVIL ENGINEERING
(WATER RESOURCES)

JUNE, 2013

CERTIFICATION

We certify that this work " Assessing the flow characteristics of Amanyi Stream for a small scale hydropower development" was done by Chijioke Chidiebere David (REG.NO. 20104768538) and it has been read and approved as meeting the requirements of the post graduate school of Federal University of Technology, Owerri for the award of Master of Engineering degree (M.Eng) in Civil Engineering (Water Resources) .

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DEDICATION

This work is dedicated to God Almighty for His Faithfulness on my life and family despite my unfaithfulness. To him alone be all the Glory.

Also to Professor and Dr (Mrs) E.J Okereke.

ACKNOWLEDGEMENT

I appreciate the effort of my main supervisor Engr. Dr. B.C Okoro and co-supervisor Dr(Mrs) B.U Dike, my lecturers; Engr Prof .Ify Nwaogazie, Engr. Prof. J.C Agunwamba, etc for their role one way or the other in the course of this work, including my HOD, Engr.Dr. J.C Ezeh.

I am indeed indebted to my parents Pastor and Mrs RC Nwebelle and my siblings Adaugo, Nomso, Chijioke and Oluchi for their care and love towards me.

I specially appreciate and honour my Uncle and his wife, Professor and Dr (Mrs) E.J. Okereke for the role they have played in my life and in the course of this study. It is God that will reward you.

I am also grateful to my Bosses; Engr and Dr (Mrs) M.C . Nwachukwu, Engr and Mrs Kingsley Onwubike for their support.

Furthermore, I am grateful to my friends and acquaintances; Chidinma Ogbennaya ,Ogechi Precious, Nwadike Onyedikachi, Dr Adaobi Ejiogu, Dr Adanna Agoha, Dr (Mrs) Onuoha Chikemdi, Engr Ekweozor Obinna, Bro Tony Onyenucheya, Amarachi, Auntie Lydia, Juliet, Chinaza, Chimezie Onwubiko, Ijeoma, Sam, Friday Agbo, etc. May the good Lord bless you all.

I can't end this without acknowledging the contributions and positive criticisms of my colleagues in the course of this Programme, Hon Ben Ekwueme, Ifeanyi Nwosu, Solomon, Anthony Ekeleme, Engr Chike, Samuel, Onyekachi, Festus, and Simeon, etc

Most of all, I appreciate God Almighty for his mercies over my life. To Him alone be all the glory.

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ABSTRACT

The study assessed the flow characteristics of Amanyi stream for a mini hydro power development for Neke Uno, Isi-Uzo Local Government Area of Enugu State, Nigeria. The stream has a maximum flow discharge, Q_{max} of $8.5m^3/s$, average discharge, Q_{av} of $6.9m^3/s$, minimum dependable discharge, Q_{min} of $4.21m^3/s$ and a selected design discharge Q_d of $6.37m^3/s$**.** Topographic survey and plant layout showed a head of 3.63m (low head) which is possible for power generation. It was also determined that for the available head and selected design discharge, a power output of 154KW is achievable. This power can meet 83% of power requirement for Neke Uno Community near the hydropower plant. This power however, can be increased by some modification of the existing topography.

It is recommended that the hydropower development of the Amanyi stream be given serious consideration under public private agreement with some financial assistance from the United Nations Industrial Development Organization (UNIDO) to make it feasible for the community. This could be a pilot hydropower supply source for rural areas that have similar streams and topography.

CHAPTER ONE

INTRODUCTION

1.1 Background Information

Flowing water creates energy that can be harnessed and converted into electrical power called hydropower. Hydropower, therefore, can be defined as electrical power generated from flowing water.

Hydropower schemes are designed to utilize the flow in a stream or river to generate electrical power, which is transmitted, to the load centre where it is consumed.

It is essential for continuity of the kind of life we now live that there should be made available a continuous supply of Electricity for domestic and industrial use.

The demand of electricity is measured in kilowatts (KW) or megawatts (MW), one megawatt being 1000 kilowatts. The quantity of electricity produced, or used over a period is expressed in kilowatts-hours (KWh).

Electricity cannot be stored in any worthwhile quantity. It therefore, has to be produced and supplied exactly as at when it is needed by the users. The potential sources of electrical energy include, coal, oil and gas, tidal energy, nuclear power, solar energy, wind power and water power. They can broadly be classified as thermal and hydro-power sources. There are well known arguments in favor or against the use of any of the above potential sources of electricity energy.

Generally, small hydropower schemes depend on either water-storage reservoir upstream of a dam, where water flow can be controlled and a nearly constant water level assured or weir where water is harnessed from the direct flow of the river through an intake structure with control gates. Water flows through the conduit called penstock, controlled by valves or turbine gates that regulate the flow rate into the turbine, turns the turbines which in-turn rotates the electric power generator to generate the much needed electricity. This is the process of hydropower generation of electricity.

The main economic arguments against generation of electricity by hydropower is that the capital cost in relation to capacity and unit output is much higher than for conventional thermal generation, particularly in drainage basins where topography and rainfall are unfavorable. On the other hand it is argued that the high capital costs of hydro-electric schemes are off-set by their relatively long life and low running cost. Besides, water power is a renewable source.

1.2 Problem statement

There is epileptic power supply in the country. The collective national power supply has failed to meet the domestic as well as industrial demand. This has led to the use of petrol/ diesel generators, firewood etc as sources of power and this contributes greatly to climate change.

The issue of climate change and the progressive effect of global warming on the environment has been attributed to the continual emission of Green House Gases (GHG), the major component of which is carbon dioxide (CO_2) emitted in the combustion of organic compound, coal, firewood, natural gas etc. thermal generation of power utilizes fossil fuel- coal,(Heavy oil) in steam turbines and therefore emits enormous amounts of CO_2 into the atmosphere. The present energy mix as practiced in Nigeria contributes positively to global warming.

The current power generation in Nigeria is the hydroelectric/thermal power generation mix. Hydroelectric Power Generation is mainly from the large dams and constitutes 39% of the total grid connected power generation.

Thermal Power Generation utilizes steam and gas turbine technologies and constitute 62% of the total grid connected power generation (Essan, 2006).

There is therefore need for the development of independent power units within the country which is less hazardous and more friendly to the environment especially where the resources that favor its exploitation exist.

Small hydropower units are most favored because it utilizes water which is available in the form of streams and rivers for its operation. There is need for small hydropower plants given the abundance of streams and rivers in Nigeria.

Amanyi stream has adequate flow for this project. There is the possibility of locating small hydropower plants at different points along the course of the river to generate electricity.

1.3 Objectives of study

The objectives of the study included:

(i) Determination of the flow characteristic of the stream under study

(ii) Assessing the adequacy of the existing flow characteristics for possible power supply.

(iii) Obtaining a hydraulic design of hydropower scheme for Amanyi stream.

(iv) Estimating the cost of the development of the scheme to ascertain feasibility

1.4 Justification of Study

The present situation of fuel crisis, poor management and distribution of electric power generated, lack of adequate infrastructure, the vanderlization of Power Holding company of Nigeria (PHCN) transmission lines has resulted in inadequate and erratic power supply in the country. There is therefore the need to seek for and develop alternative energy sources to meet the energy needs of Nigerians.

Globally, the current trend in power generation is to develop power from an energy source which is environmentally friendly and would reduce to the bearest minimum the production of $C0_2$ which is a major component of Green House Gases, (GHG).

Hydropower is the only clean and ever renewable source of power available round the clock as contribution to the renewable energy requirement of the Energy Master Plan for Nigeria.

Furthermore, this will reduce technical, institutional as well as information barriers to the implementation of community based projects.

Small Hydropower Development for Amanyi stream would be a viable alternative for providing plentiful, cheap, renewable and environmentally clean energy In addition, it will promote community self sufficiency through rural electrification and industrialization by locating the power source nearby Neke-Uno community, rather than depend on external unreliable supplies from PHCH.

The development of Small Hydropower Schemes is also in line with Government policies like National Poverty Eradication Programme (NAPEP) as well as New Partnership for African Development (NEPAD), the African Union whose programme at local, state and regional levels have energy development and rural infrastructure as one of its priorities.

1.5 Scope of Study

In a study of this nature, hydrological and topographical conditions are the limiting factors that determine the type of Small Hydropower Scheme for any location. Therefore, the study was limited to the hydrology and topography of the selected site:

- The study is limited to Amanyi stream for which flow measurements were taken.
- Only ten year hydrological data (1988 to 1997) was used in this study due to unavailability of more data.
- The topographical map available for the site has a scale of 1:50000. For this scale of map, head variations could not be precisely determined for the river.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

The study of hydropower potentials of a stream or river is fundamental to the development of any Hydropower Scheme. It indicates the maximum or minimum power that can be generated from a stream or river.

2.2 Origin of Hydropower

Hydropower first started as early back as 100 B.C. when the Greeks and Romans used a waterwheel, which was vertical and placed along a stream or river, for grinding corn with its gears. Running water in the stream would turn the wheel, therefore operating the mill. Milling was the main task performed with hydropower back then, and soon it traveled through Asia and the rest of Europe by 4 A.D. (http//web.mit.edu,2012).

As the waterwheel spread to different parts of the world, people began to try ideas to improve on it. Change in wheel orientation was one important development. The horizontal wheel laid on its side and the wheel turned from left to right. The vertical wheel stood up, turning from top to bottom. In the beginning, "millers mounted the wheel so that the center was above the water surface and the running water would turn the bottom of the wheel. Later, they would dip the wheel below water level in an "overshot" orientation."

In the 18th century, John Smeaton tested both orientations and found that the overshot worked more efficiently. In the next century, engineers perfected the waterwheel and found two improvements: curved paddles worked better and that the breasted position (where the center of the wheel lies on the water surface) made the wheel more efficient .
These developments helped people apply the waterwheel to more tasks, such as a mill where gears, shafts, and conveyors would not only grind grain but also transport the grain up, down, and sideways within a mill. In the 19th century, the water turbine slowly replaced the waterwheel due to its higher efficiency. However, waterwheels still exist throughout the world to this day. Following the waterwheel came the water turbine, which used gravity to turn the wheel. James Francis perfected Samuel Howd's turbine by curving its blades, and today it is known as the Francis turbine. This was used for a long time in mills, but eventually steam engines took the turbines place as it's source of power. However, the Francis turbine would make a come back as a different source of energy- hydropower (Suwa, 2009).

2.3 Hydropower Potentials in Africa

Sub-Saharan Africa has a tropical climate and a significant number of perennial rivers, with a potential to generate over 1,750 Terawatts of electricity. However, only about 7 per cent of this potential has been developed. Exploration of key rivers such as the Zambezi, Congo, Nile, and

Niger could provide a solution to Africa's power problems. The Congo River in DRC has the potential to produce over 100,000MW, which is sufficient to meet the energy needs of the whole Southern Africa. The Zambezi River can produce 10, 000MW. Other examples include ; Ethiopia, with a hydro potential of 30,000MW, and Nigeria with over 20,000MW (Mckenzie, 2012) see Table 2.1.

Table 2.1 Hydropower Potentials of selected countries in Africa

Country	Hydropower potential (MW)	Current installed capacity (MW)	Percentage (%)	Electricity access rate (%)
Angola	18,000	527	3	15
Cameroun	20,000	720	4	47
Congo DR	100,000	2446	2	6
Ethiopia	30,000	796	3	15
Gabon	6,000	170	3	47
Madagascar	7,800	150	2	15
Mozambique	13,000	2199	17	6
Nigeria	20,000	301	7	40
South Africa	10,000	2000	20	70
Zambia	6,000	1796	29	19

(Mckenzie, 2012)

2.4 Hydropower development in Nigeria

Hydropower Development in Nigeria commenced with the establishment of the Nigerian Electricity Supply Company Limited (NESCO); which was floated at the London Stock Exchange in 1923 by a group of businessmen to provide electricity to the mechanized Tin Mining Companies in the Plateau areas. In 1929, NESCO commenced operation with the installation of a 1000 KVA (800KW) hydro-electric plant at Kura falls in Jos Plateau, with concession granted by the Federal Government to Supply the Plateau minefields with electric power, provide a bulk supply of electricity to Electricity Corporation of Nigeria(ECN) and provide direct electricity supplies to the rural areas of Plateau not served by ECN (Arugbemi , 1984).

Electricity Corporation of Nigeria existed prior to 1951, overseeing the diesel electricity generation and distribution systems in the country. In 1951, it authorized hydro-electric investigation of the River Niger upstream of Jebba (Arugbemi ,1984).

In 1962, the Niger Dam Authority (NDA) was established by an act of parliament to oversee hydropower development and in particular the construction of Kainji hydropower plant between 1962 and 1969.

In 1972, NDA and Electricity Corporation of Nigeria were merged to form Nigeria Electric power Authority (NEPA) which saw the building of Jebba and Shiroro hydropower stations in 1984 and 1990 respectively (Arugbemi, 1984).

1n 2002, a Memorandum of understanding was signed between Energy Commission of Nigeria, United Nations Industrial Development

Organization and International center for Small Hydropower (ICSHP),China for Small Hydropower (SHP) Development in Nigeria. This has led to the development of two pilot SHP schemes-150kw and 30kw capacity schemes on Waya dam in Bauchi and at Mgbowo in Enugu state respectively (Nwachukwu, 2009).

In 2005, United Nations Industrial Development Organization(UNIDO) Regional Center for SHP development in Africa was established in Abuja to coordinate all activities associated with the development of SHP in Africa which includes Capacity building, feasibility and detailed project reporting and documentation of SHP sites.

Furthermore, in 2005, UNIDO carried out a study and design of Micro-Hydropower Scheme for Okwamu and Keijiukwu Communities of Obudu Cattle Ranch in Cross River State of Nigeria. This is a 30KW capacity, run-off the river type micro-hydropower scheme located on the hilly and rolling terrain of the Obudu hills.

Also, in 2006, UNIDO carried out the study, design and construction of a 30kw capacity hydropower scheme for Mgbowo community in Awgu Local Government Area of Enugu State. This was done in partnership with the Anambra Imo River Basin and Rural Development Authority.

Nigeria has a total installed electricity generation capacity of over 6000MW and only about 39% is from hydropower and these are mainly large hydropower schemes (Makanju, 2003).

2.5 Need for Small Hydropower Development

Small Hydropower is defined internationally as any hydro installation rated at less than 10MW of installed capacity that makes use of a full renewable(at every stage of energy generation) indigenous and readily available natural source water(Essan, 2004).

Nigeria signed in as the 52nd member country of the Kyoto protocol agreement by which it became mandatory for Nigeria to adopt new policy in clean energy source for its energy development of power than from other environmentally unfriendly sources, i.e, thermal, etc. Hence, there is the need for hydropower development in rural areas of Nigeria to effect the policy. The 1990 energy master plan for Nigeria indicates that 15,000MW of power was planned for the country by the year 2003. But due to gradual decline in Nigerian economy, this was reviewed downward to 9,000MW of which 1200MW must be developed from renewable energy sources (Makanju, 2003).

About 70% of the population of Nigerians resides in the rural area where most of the rivers that can be harnessed for SHP development are located. These areas are known to be usually far away from the National Electricity grid and the prohibitive cost of grid extension has further deterred the provision of electricity to the rural communities.

The cheapest power is developed and utilized close to where it is needed. This makes Small Hydropower Development for rural electricity supply the best alternative source of electricity especially for communities that have favorable topography and water flow.

In most of the rural areas and suburbs of urban areas in the country, firewood is the primary source of energy for cooking. Gas, kerosene, and diesel are secondary sources of energy for cooking and lighting. These forms of energy are damaging to the environment in a number of ways including global warming. While wood is a commercial source of energy and is almost free, other sources of energy, like gas, diesel, kerosene, etc have to be transported over long distances to the rural and remote areas. Consequently, they are expensive and do not provide many viable options.

The cost of petroleum products; petrol, diesel, kerosene, gas, etc are ever increasing with associated increase in cost of operation and maintenance of alternative energy sources like thermal and diesel driven electricity generators. These also contribute to global warming. Also, fuel crisis, poor management and distribution of electric power generated, lack of adequate infrastructure to make power available to the teaming population, and the vanderlization of Power Holding Company of Nigeria(PHCN) transmission lines had further compelled the Federal Government of Nigeria to seek for and develop alternative energy sources to meet the energy needs of its population.

Small Hydropower, with clean, abundant and ever renewable source of power remains the most viable alternative suitable for further development in Nigeria.

The biggest advantage of small hydropower is that it is the only clean and renewable source of energy available round the clock. The earth's water cycle guarantees an endless supply of water from rain and snow.

The Amanyi stream is one which can be investigated for its hydropower potentials and possible exploitation.

There are speculations about the hydropower potentials of Amanyi. However, no scientific or systematic attempts have been made to assess the full potential available.

This study is the first time such an attempt is being made to fully assess the overall hydropower potential of Amanyi stream.

Therefore, this literature review will include hydropower development in Nigeria, the description of the types and classification of hydropower schemes, their components and existing small hydropower potentials in Nigeria.

2.6 Nigeria Hydropower Environment

Nigeria has a population of over 150million people. Over 70% of these people reside in the rural area and only about 18% of these rural dwellers have access to electricity presently (Aliyu, 2004).

Nigeria is naturally endowed with the abundance of water resources (rivers) criss crossing the length and breadth of the country as shown in Fig 2.1. The rivers are organized into five drainage systems, namely the Niger, Benue, Chad, Cross and Atlantic Systems. Apart from the Chad system of inland drainage, almost all the rest of Nigeria's river systems drain into the sea. The River Niger and its major tributary the Benue are very outstanding features of the Physical Geography of Nigeria. They have not only punctuated the land surface of the country in a remarkable way but also greatly influenced the lives of its people and form the major drainage route of all the rivers to the Atlantic Ocean.

In the North, the central high lands of Jos Plateau form a major hydrographical centre from which radial pattern of drainage develops with streams draining to Zanfara, Sokoto and Chad basin. Other rivers drain into the Niger and Benue River. These form the Niger -Benue Systems.

In the Southeast, the major rivers in this area, the Imo River and the Cross-River are oriented towards the sea while the Anambra River joins the Niger River near Onitsha. These form the Cross System. In the West, the major rivers of the area are the Ogun, Oshun, Shasha, Yewa and Oluwa which orients in the North South direction towards the Atlantic System.
The major stream which drains into the Chad, namely the Yobe River and its main tributaries, the Ngade, the Mbudic and the Gome form a centripetal pattern and constitute the Chad System (Nwachukwu, 2009).
These rivers in their naturally occurring state are the basis for the formation of the eleven river basins in Nigeria as shown in Fig 2.1.

Figure 2.1 Nigerian hydropower environment

2.7 Advantages and disadvantages of small hydropower scheme

2.7.1 Advantages

- Small Hydropower does not potentially cause the submergence of forest, reservoir enlargement, residential relocation or seismological threats.
- The generation of Small Hydropower produces no green house gas emissions, pollutants or any waste products, which might require special handling or disposables.
- It is a low cost alternative that attracts industries and jobs, stimulates the economy and is flexible and reliable.
- Small Hydropower scheme enhances economic development and living standard especially in remote and rural areas with limited or no electricity. In some cases, rural dwellers have been known to switch from firewood for cooking to electricity, thus limiting deforestation and also cutting down on carbon emission.
- On a large scale, rural communities have been able to attract new industries, mostly related to agriculture owing to their ability to draw power from isolated and remote based hydropower schemes.
- It is an attractive alternative to diesel systems in rural and remote areas where it can serve as a means of achieving rural electrification (Mckenzie, 2012).

2.7.2 Disadvantages

- It has high capital cost outlay in relation to capacity and unit output when compared with other conventional types of generation.
- It prevents navigation.
- The safety of aquatic life in the river is not usually guaranteed during operation of the turbine (Mckenzie, 2012).

2.8 Types of Small Hydropower Scheme

There are basically three common types of Small Hydropower Schemes namely:

a. Diversion or Run-of-River
b. Canal Fall based
c. Dam Toe based.

2.8.1 Diversion or run-of-river hydropower scheme

This type of scheme utilizes water diverted from a river or canal and flowing through the turbine to generate electricity.

The basic components of this type of scheme are:

i) Diversion/intake structure
ii) Head race canal
iii)Desilting basin
iv) Conveyance channel
v) Fore bay
vi)Penstock
vii)Power house

The river is diverted into the headrace canal by the diversion/intake structure, causing water to flow through the components, the desilting basin, and canal, fore bay, penstock and hydraulic turbine in the powerhouse where electricity is generated. This type of scheme is suitable generally in any terrain where there is appreciable water flow and the topography not favorable for dam construction.

2.8.2 Canal fall based

This type of scheme is similar to the Run-of-River type in having the same basic components and also utilizes water diverted from a river or canal which flow through the turbine to generate electricity.

The basic components of this type of scheme are:

i) Diversion/intake structure

ii) Head race canal

iii) Desilting basin

iv) Conveyance channel

V) Fore bay

VI) Penstock

Vii) Power house

The river is diverted into the headrace canal by the diversion/intake structure, causing water to flow through to the components, the desilting basin, canal, fore bay, penstock and hydraulic turbine in the powerhouse where electricity is generated. This type of scheme is the ultra low head type and is applicable on irrigation canal where there is constant water flow and a drop in head range between 1.0m – 1.5m in the canal (Nanayakkara, 2004).

2.8.3 Dam toe based hydropower scheme

The major components of this type of scheme are:

i. Dam
ii. Penstock
iii. Powerhouse

In this type, an impoundment facility is used to store water in a reservoir and to build up the water level to the desired head for power generation. The penstock connects the dam to the powerhouse, which is located at the toe of the dam at a lower elevation. Power is generated from water flowing through the penstock and the capacity depends on the gross head between the upstream water level in the reservoir and penstock intake to the turbine in the powerhouse.

This type of scheme is applicable where the stream flow, Q, is not appreciable or adequate to generate the power required and the desired head, h, has to be achieved by constructing a dam in order to generate higher power.

It is generally expensive due to high cost of civil works (dam, spillway, penstock, etc) involved. However, when incorporated as in a multi-purpose Dam its increased benefits accruing from the other uses of the water, e.g., water supply, irrigation, navigation, etc. makes it economically viable.

2.9 Classification of small hydropower scheme

Hydropower schemes are classified according to its installed capacity or according to the available gross head at any site, see Tables 2.2 and 2.3.

Table 2.2: Classification of Hydropower Scheme by installed capacity

Scale of Hydropower Scheme	**Capacity Range**
Big	> 10MW
Small	1.0 -10MW
Mini	100KW – 1MW
Micro	< 100KW

(Source; http/www.unesco-ihe.org/edu/hydropowerdevelopment, 2011)

Table 2.3: **Classification of Hydropower Scheme by Gross available head**

Type of Hydropower Scheme	Gross Head available
Ultra low-head	< 3 meters
Low-head	< 40 meters
Medium/High head	> 40 meters

(Source; http/www.unesco-ihe.org/hydropowerdevelopment, 2011)

2.10 Existing Small Hydropower Schemes in Nigeria

There exist small hydropower schemes in Nigeria. According to the 2007 renewable energy master plan for Nigeria, the types, capacity and location of existing small hydropower schemes in Nigeria are shown in Table 2.4.

Table 2.4 Existing small hydropower stations in Nigeria

s/n	Location	State	Installed capacity	Current status
1	Kwa falls	Plateau	6.0MW	Operational
2	Kwa falls	Plateau	19.0MW	Operational
3	Bakolori	Sokoto	3.0MW	Dam construction completed. electromechanical equipment, never installed
4	Tiga	Kano	6.0MW	Dam construction completed. electromechanical equipment, never installed
5	Ikere Gorge	Oyo	6.0MW	Dam construction completed. electromechanical equipment, never installed
6	Oyan Dam	Ogun	9.0MW	Dam construction completed. electromechanical equipment, never installed
7	Waya Dam	Bauchi	150KW	Completed 2006
8	Mgbowo Dam	Enugu	30KW	Completed 2006
9	Challawa Gorge Dam	Kano	7.0MW	Dam construction completed. electromechanical equipment, never installed

(Source; Nwachukwu, 2009)

2.11 Big hydropower scheme in Nigeria

In Nigeria and worldwide, the development of big hydropower scheme was favored before the 1973 world energy crisis. According to Arugbemi (1984), this was so because the economies of scale clearly favored its development as well as thermal plants. This led to development of various hydropower schemes across the country. The plants installed capacity and period of construction and installation of plants in these schemes are outlined below:

i. Kainji Hydroelectric Power Station, Kainji.(760MW) 1962- 1969
ii. Jebba Hydroelectric Power Station, Jebba. (540MW). 1984
iii. Shiroro Hydroelectric Power Station, Shiroro,(600MW). 1988-1989.

Other schemes that were developed as private sector initiatives include:

a) National Electricity Supply Company Bukuru ,Jos.(25MW)
b) Onyan Dam(9MW)
c) Dadin Kowa Dam(34MW)

(Arugbemi, 1984)

CHAPTER THREE

METHODOLOGY

3.1 Description of Study Area

Neke-Uno is an autonomous community in Isiuzo local Government area of Enugu State. It has four villages namely; Isi-enu, Umugwu, Obegabu and Umuegwu. It has a population of about 7500 persons (2006 population census). Neke-Uno is located within the natural drainage area of Amanyi stream covering Neke and Mbu communities in Isiuzo local Government area of Enugu State.

3.1.1 Amanyi Stream

Amanyi stream takes its source from Oba in Udenu Local Govt Area of Enugu State, flowing southwards through Mbu community, Neke community, Down to Ikem (all in Isiuzo Local Gov Area) where it discharges into Ebenyi River and flows down to Ebonyi state.

3.1.2 Physiography

There is little physiographic differentiation over the entire water shed which is uniformly undulating.

The geology of this area reveals the predominance of lateritic and gravelly soil type which is suitable foundation as well as construction materials. (Onu, 2009)

The selected site for this study is located at the box culvert on Amanyi stream along Neke-Obollo Road. Neke Uno community is the nearest load centre and it is within 1.5km radius of the site.

3.1.3 Hydrology

The Amanyi stream is perennial, however, water availability needs to be evaluated to determine the desirable quantity of water required and its distribution in order to maximize the power generation from the river. The total quantity of water available can be estimated from the stream flow data and its variation determined from the flow duration curve.

3.1.4 Rainfall

There are two major seasons in Nigeria, the rainy season from April to October and the dry season from November to March. The study area lies within the rain forest region where rainfall is experienced all through the rainy season. The average monthly rainfall has a minimum of 12.06mm occurring in December and a peak of 360mm occurring in September with an annual monthly average of 198mm.

3.2 Socio-economic activities

The people of Neke –Uno are predominantly farmers and traders and their farm produce includes; cassava, oil palm, maize, fruits and vegetables, etc. They are also involved in Cassava processing, palm kernel cracking, palm wine tapping, and petty trading.

Generally, Neke is a peace-loving community and also, welcomes developmental changes in terms of infrastructural development.

3.3 Site selection

In order to ensure that a hydropower scheme is operational throughout its envisaged life span, water which is the primary non -consumptive fuel must be available in sufficient quantity throughout its life span and its utilization should not adversely affect downstream users. Water should therefore be available in sufficient quantity all the time. The river or stream from which water is tapped for small hydropower development must therefore be perennial.

The suitable site for Small Hydropower Development must also show good head variation such that the combination of the head and flow produces the best hydropower potential and presents a lay-out that will influence minimum civil works cost.

The following criteria adopted by the Alternate Hydro Energy Centre (2004) (AHEC,) of the Indian Institute of Technology, IIT, Roorkee were considered in the selection of the site for this study.

i. The catchment should have an area of at least about $250km^2$. The objective is to ensure that there is a certain minimum discharge to generate small hydropower.
ii. The channel must be perennial in nature. The idea is to have round the year electricity generation.
iii. Main channel should be straight with no obstruction to flow; this is to enable a free flowing channel.
iv. Availability of head should be at least 3m as lower head and discharge may not yield significant power potential.
v. The location should be near centers of concentrated population (1.5 -2.0km) to reduce transmission losses and associated overall cost.

The study site was selected by applying the criteria adopted by the Alternate Hydro Energy Centre (AHEC) of the Indian Institute of Technology (IIT),Rookee, India.

3.4 Topographical Map

The study area was surveyed by carrying out leveling and traversing on upstream and downstream location of the existing culvert to obtain a topographical map of the study area.

Thereafter the location of the components of the scheme are identified and superimposed on the topographical map to produce a lay-out of the scheme. The major components of the scheme are the gated culvert, intake structure, penstock, power house, and tailrace canal. These are shown in Fig 3.1.

3.5 Data collection

The discharge data for this study were obtained from the hydrological year book of Anambra-Imo River Basin Development Authority. The data consist of daily river discharge records for the existing gauge station at Uno-Neke old bridge. The daily flow discharges data are characterized by missing data within some months in the hydrological year and in some cases missing years. This is due to discontinuity in data collection in the late 1998 and early 1999. The available record for a period of 10 years (1988 -1998) was collected for this study and is shown in appendix A.

Missing records in the discharge data were replaced by simple interpolation technique. The rainfall data was obtained from the Nigerian Meteorological Station main office in Lagos. Twenty years record is available and was collected for this study.

Lay out of scheme fig 3.1

3.6 Hydrograph

In assessing the hydropower potential of a river, it is important to know whether the river flow is sustained by direct runoff or by base flow or both. A river sustained by direct runoff is most likely to fluctuate adversely with rainfall availability than one sustained by base flow alone. The base flow is therefore the minimum design flow of any river.

The stream flow discharge of a river consists basically of base flow and direct runoff components. The base flow is the flow of a river when there is no influence of rainfall precipitation within the catchment contributing flow to the river while the direct runoff is that component of flow resulting from the influence of rainfall precipitation within the catchment.

Generally, the stream flow discharge when plotted against time presents the hydrograph. A typical hydrograph is shown below in Fig 3.2. The hydrograph consists of a rising limb, a crest segment and a recession limb. The rising limb BC has a well defined point of rise B followed by increasing discharge.

The crest segment CE contains the peak discharge within it and from E it is the recession limb with decreasing discharge. The recession limb is also known as the falling limb or depletion curve. Points C and E are inflection points on the rising and recession limbs respectively. The segment AB is called the approach segment which indicates the base flow in the river prior to the storm.

The shape of the rising limb depends mainly on the duration and the intensity distribution of rainfall and to some extent on the antecedent condition and the shape of the time area diagram of the basin.

The peak discharge included in the crest segment represents the highest concentration of the runoff from the basin. It occurs usually at a certain time

TYPICAL HYDROGRAPH FIG 3.2

after rainfall has ended and this time depends on the area distribution of rainfall. The point of inflection on the recession limb is assumed to mark the time at which surface inflow to the channel or overland flow ceases. Thereafter the recession curve represents the withdrawal of water from storage within the river system. Therefore, it is more or less independent of variations in rainfall and infiltration. On large basins, the shape of the recession curve may vary from storm to storm. The recession curve is also known as depletion curve since it represents the depletion from the channel storage and can be expressed as

$$Q_t = Q_0 \, Kr^{(t_1 - t_0)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.1$$

where,

Q_t =discharge times after time t,

Q_0 =discharge at the beginning of storm,

K_r = recession constant.

The usual values for K_r lie between 0.85 and 0.90 (Nwachukwu, 2009)

3.6.1 Hydrograph Analysis

The discharge data for Amanyi stream is presented in Appendix A and were analyzed for mean monthly flows and recorded in Table 3.1.

The average monthly discharge of Table 3.1 was plotted on the same axis to reveal the various features of the hydrographs for each year. This is presented in Fig .3.3. The hydrograph for 1989/90 showed the characteristic feature of a hydrograph and the only hydrograph amenable to base flow separation by recession curve method. It cuts across all other hydrographs in such a manner that it appears to be the mean hydrograph for the ten year data. This hydrograph, 1989/90 shown in Fig. 3.4 was then selected on this basis as representative of all hydrographs for base flow separation.

3.6.2 Base flow

The recession curve approach was used in this work. It relates to the use of recession curve equation (3.1) and have been used for the analysis in this work.

Each hydrograph is examined for the flow before the start point of the rising limb. The flow associated with the first point before runoff is experienced is analogous to Q_0 and the corresponding time t_0. The flow at the point of rise of the rising limb is analogous to Q_t and the corresponding time t.

The recession curve equation (3.1) is now applied to the stream flow hydrograph by substituting Q_0 and Q_t to obtain the recession constant K_r.

$$(K_r)^{t-t_0} = Q_0/\,Q_t \qquad \text{.................................3.2}$$

K_r and Q_0 are re-substituted in equation (3.1) and with monthly time units t. Q_t is computed for the monthly intervals after the rise of the hydrograph to obtain the ground water component of the flow. This is plotted as the recession curve or base flow.

The difference between stream flow discharge and base flow is the direct runoff.

3.6.3 Base Flow Separation

The base flow separation was carried out by applying the recession curve equation (3.1) to the 1989/90 stream flow data of Table 3.1 to obtain the recession constant k, in equation (3.2). The base flow Q_b, was then computed for each stream flow discharge Q and shown in Table 3.2. The runoff component Q_r was obtained by subtracting the base flow from the stream discharge ($Q-Q_b$) and also shown in Table 3.2. The base flow curve was then plotted as shown in Fig. 3.5. The contribution of base flow to the total discharge was determined by dividing the total annual base flow $\sum Q_b$ by the total discharge Q_T, ($\sum Q_b/Q_T$) and the contribution of runoff to the total discharge was obtained by dividing the total runoff $\sum Q_r$ by total discharge Q_T, ($\sum Q_r/Q_T$).

3.6.4 Determination of recession constant K_r and the recession curve equation

From the 1989/90 hydrograph, for $Q_0 = 5.36m^3/s$, t_0 = 1month and for $Q_4 = 4.68m^3/s$, t =4 months and substituting these values in the recession curve equation (3.2) we have:

$$Q_4 = Q_0 K_r^{(t_4 - t_0)} \quad t_4 - t_0 = 4\text{-}1 = 3$$

$$K^3 = 4.68/5.36$$

$$= 0.873$$

$$K = 0.95$$

substituting K= 0.95 in the recession curve equation or base flow equation (3.2) we have:

$$Q_t = (5.36)(0.95)^{(t - t_0)}$$

$$Q_t = 5.09^{(t - t_0)}$$

This equation is now used to compute the base flow for the various time intervals and tabulated in Table 3.2 as Q_b. The difference between this value and the stream flow discharge for the corresponding time interval is the Runoff Q_r. The base flow is plotted on the same axis as the discharge hydrograph and shown in Fig. 3.5. The percentage of base flow to the total discharge is 67.19% while the percentage of runoff to total discharge is 32.81%.

The analysis reveals that the contributions of the base flow to the total discharge for Amanyi stream is about 67% which is substantial while the contribution of the runoff is about 33%. Therefore, Amanyi stream is greatly sustained by the underground flow and runoff does not have significant effect on the river discharge. Amanyi stream is therefore suitable for small hydropower development.

Fig 3.4. 1989/90 Stream Discharge Hydrograph of Amanyi

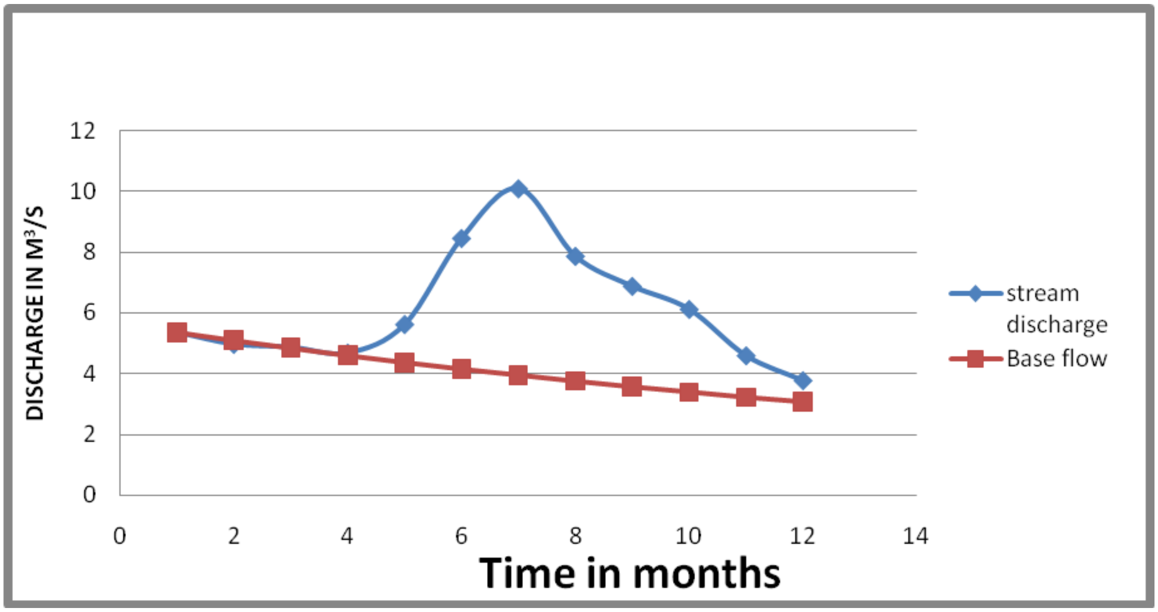

Fig. 3.5 1989/90 Stream Discharge Hydrograph and Base flow of Amanyi

Table 3.1 Mean Monthly Discharge Data (m^3/s)

YR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR
1988/89	3.85	3.65	3.54	3.77	4.09	5.35	7.48	6.56	5.79	4.64	4.07	2.61
1989/90	5.36	4.97	4.87	4.68	5.62	8.46	10.1	7.87	6.88	6.12	4.59	3.77
1990/91	2.49	2.23	3.22	3.85	4.33	5.45	6.14	6.14	5.70	5.55	4.66	4.49
1991/92	4.11	4.23	4.59	4.97	6.15	8.20	8.61	7.64	7.28	7.39	7.16	6.88
1992/93	6.67	6.97	6.93	7.01	6.97	6.97	7.99	8.42	8.68	7.69	7.11	7.33
1993/94	7.32	8.46	10.1	9.83	9.81	11.2	11.8	11.3	11.2	11.2	10.3	9.33
1994/95	6.06	6.35	6.81	7.33	7.87	7.55	8.10	8.53	8.84	7.82	7.47	7.82
1995/96	7.39	7.14	7.24	7.79	8.88	9.25	9.20	8.15	7.95	7.91	7.71	7.73
1996/97	7.84	8.77	8.19	8.01	8.08	8.67	9.23	9.21	9.13	9.08	8.69	8.96
1997/98	8.60	8.81	8.71	8.68	8.63	9.13	9.15	9.28	8.80	8.43	8.09	7.91

Table 3.2 1989/90 Hydrograph and base flow separation

Month	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR
89/90 Mean discharge Q (m^3/s)	5.36	4.97	4.87	4.68	5.62	8.46	10.1	7.87	6.88	6.12	4.59	3.77
Base flow Qb (m^3/s)	5.36	5.09	4.84	4.59	4.36	4.14	3.94	3.74	3.55	3.38	3.2	3.05
Direct runoff (m^3/s)	.	-0.12	0.03	0.09	1.26	4.32	6.16	4.13	3.33	2.74	1.39	0.72

% of base flow to discharge = 0.67185

% of runoff to discharge = 0.32815

3.7 Flow Duration Curve

The flow duration curve (FDC), is a river discharge frequency curve which shows the percentage of time that certain values of discharge weekly, monthly or yearly are equaled or exceeded. The design flow, average flow, installed capacity of turbine, expected annual energy generation and the hydropower potentials of the river can be determined from the FDC.

There are two methods for preparing a flow duration curve:

i) The rank order method and

ii) The group/class interval method

i) The Rank order Method

In the rank order method, the stream flow data is rank-ordered according to magnitude of flow. The rank-ordered values are then assigned other numbers beginning with the largest. The order numbers are then divided by the total number in the record and multiplied by 100 to represent the percent of time interval that a particular average flow has been equaled or exceeded during the period of the record. The flow value is then plotted against the respective "percent exceedence" to obtain the flow duration curve.

ii) Class Interval Method

In class interval method, the procedure according to Rams (1989) is as follows:

a. The total average of discharge is divided into a number of classes.
b. The entire record is scanned day by day for daily flows, month by month for monthly flows, year by year for yearly flows.
c. A tally is made of number of flows in each class for each item in the record.
d. The number of items greater than each class is cumulated starting from the highest item. The accumulated number of items of each class is divided by the total number of items in the record to obtain the "percent exceedence".
e. This percent exceedence is plotted against the upper or (mean) class interval to get the flow duration curve.

The stream flow characteristics can be determined from the slope of the flow duration curve. A flat-sloped curve indicates a river with a few floods with large ground water contribution and natural storage, while a steep slope indicates the flashy nature of the stream, i.e, frequent floods and dry periods with little ground water contribution. The class interval method was used in this work. The discharge data for Amanyi stream presented in Appendix A were analyzed for 10 daily averages and recorded in Table 3.3. The 10 daily average discharge data of Table 3.3 was used for the flow duration curve analysis.

The following steps according to Rams(1989) were followed in carrying out the flow duration curve analysis:

i) The total range of discharge values (2-13m^3/s) were divided into 12 classes and suitable class intervals selected and shown in Table 3.4.

ii) The data was then scanned and the number of occurrences of each discharge value is entered into appropriate class interval and recorded as shown in Table 3.4.

iii) These values were then arranged in ascending order of magnitude and recorded in column 2 of Table 3.5.

iv) The number of times and the percent of time each class interval flow value has been equaled or exceeded in the period of the record is obtained by the cumulative addition of the values in column 2 beginning from the last value and upwards. This is shown in column 3 of Table 3.5.

v) The percentage of time the value of the class interval is equaled or exceeded is computed as shown in column 4 of Table 3.5.

vi) The lower value of each class interval as(y-axis), is then plotted against the percentage of time exceeded by flow as(x-axis) to obtain the flow duration curve FDC and shown in Fig.3.6.

Table 3.3. 10 -Day Average Discharge Measurement of Amanyi Stream

Year	day	APR	MAY	JUN	JUL	AU	SEP	OCT	NOV	DEC	JAN	FEB	MAR
1988/89	10	3.90	3.70	3.60	3.60	3.89	4.29	7.57	7.12	6.16	4.87	4.30	3.11
	20	3.85	3.64	3.79	3.73	4.02	5.79	7.17	6.42	6.04	4.58	4.07	2.48
	30	3.80	3.60	3.96	3.96	4.35	5.96	7.69	6.19	5.24	4.50	3.80	2.27
1989/90	10	5.48	5.10	4.59	4.51	5.33	7.55	9.84	8.16	7.11	6.22	4.96	3.88
	20	5.27	4.91	4.84	4.41	5.08	8.54	10.31	7.83	6.91	6.38	4.61	3.76
	30	5.34	4.90	5.19	5.05	6.38	9.29	10.15	7.61	6.62	5.70	4.17	3.68
1990/91	10	2.68	2.31	2.41	3.67	4.03	5.14	6.06	5.96	5.85	5.80	4.80	4.41
	20	2.49	2.27	3.25	3.76	4.23	5.29	6.13	6.42	5.68	5.56	4.63	4.41
	30	2.30	2.13	3.99	4.08	4.70	5.92	6.23	6.05	5.56	5.33	4.54	4.65
1991/92	10	4.18	4.22	4.27	4.74	5.44	7.44	8.95	8.02	7.28	7.35	7.34	6.82
	20	4.23	4.23	4.57	4.88	6.26	8.62	8.67	7.69	7.24	7.43	7.09	6.92
	30	4.11	4.23	4.92	5.27	6.26	8.54	8.25	7.25	7.33	7.38	7.04	6.91
1992/93	10	6.68	6.65	7.15	6.69	6.72	7.08	7.68	7.95	8.72	8.26	7.00	7.41
	20	6.66	6.86	6.73	7.43	6.25	6.86	8.26	8.66	8.68	7.54	7.09	7.23
	30	6.67	7.36	6.92	6.91	7.30	7.13	8.03	8.66	8.64	7.26	7.26	7.35
1993/94	10	7.53	7.86	10.17	9.98	9.70	10.46	11.57	11.41	11.20	11.20	10.57	10.14
	20	7.36	7.75	10.18	9.97	9.83	11.50	11.74	11.36	11.18	11.17	10.35	9.04
	30	7.09	10.63	10.13	10.52	10.88	11.53	13.39	13.39	12.21	12.21	9.06	9.74
1994/95	10	5.97	6.20	6.57	7.19	7.34	7.87	7.70	8.27	8.68	8.02	7.57	7.70
	20	6.03	6.43	6.87	7.39	8.11	7.36	8.32	8.61	8.53	7.21	7.21	7.73
	30	6.18	6.40	6.98	7.41	8.15	7.41	8.26	8.71	8.26	7.64	7.66	8.01
1995/96	10	7.45	7.20	7.16	7.38	8.55	9.72	9.36	8.33	7.99	7.93	7.73	7.75
	20	7.44	7.11	7.15	7.99	8.99	9.18	9.04	8.16	7.99	8.04	7.74	7.87
	30	7.11	7.11	7.40	7.98	9.07	8.84	9.21	7.98	7.88	7.79	7.64	7.87
1996/97	10	7.95	8.20	8.90	7.22	8.33	8.50	9.31	9.25	9.13	9.10	8.76	8.60
	20	7.68	8.56	7.98	7.66	7.83	8.75	9.20	9.24	9.16	8.64	8.64	9.56
	30	7.88	8.49	7.68	8.68	8.07	8.69	9.18	9.14	9.10	9.05	8.65	8.74
1997/98	10	8.67	8.18	8.86	8.86	8.88	9.30	9.30	9.30	8.90	8.55	8.19	8.40
	20	8.60	8.82	8.65	8.71	8.30	9.43	9.05	9.22	8.82	8.41	8.08	7.90
	30	8.54	8.81	8.58	8.49	8.69	8.69	9.11	9.21	8.68	8.34	7.98	7.47

Table 3.4 Arrangement of flows into class intervals (CI)

CI	2-2.9	3-3.9	4-4.9	5-5.99	6-6.99	7-7.99	8-8.99	9-9.99	10-10.99	11-11.99	12-12.99	13-13.9	Total
1988/89	0	15	4	8	5	4	0	0	0	0	0	0	36
1989/90	0	3	9	9	5	4	2	2	2	0	0	0	36
1990/91	7	4	1	10	5	0	0	0	0	0	0	0	36
1991/92	0	0	0	2	5	12	6	0	0	0	0	0	36
1992/93	0	0	11	0	12	16	8	0	0	0	0	0	36
1993/94	0	0	0	0	0	5	0	0	10	10	2	2	36
1994/95	0	0	0	1	8	15	12	70	0	0	0	0	36
1995/96	0	0	0	0	0	24	6	6	0	0	0	0	36
1996/97	0	0	0	0	0	8	15	13	0	0	0	0	36
1997/98	0	0	0	0	0	3	25	8	0	0	0	0	36
Total	7	22	34	30	40	91	74	36	12	10	2	2	360

Table 3.5: Flow Duration Analysis of 10-Daily Average monthly flow data for Amanyi stream

s/n	10 daily average monthly flow class interval (C.I)	No of occurrences in 10yr period (n)	No of time equaled or exceeded (m)	% of time lower value of C.I equaled or exceeded (m/n) x 100	Monthly power P=7xQxH (kw)
1	2-2.99	7	360	100	42
2	3-3.99	22	353	98.05	63
3	4-4.99	34	331	91.94	84
4	5-5.99	30	297	82.5	105
5	6-6.99	40	267	74.16	126
6	7-7.99	91	227	63.05	147
7	8-8.99	74	136	37.78	168
8	9-9.99	36	62	17.22	189
9	10-10.99	12	26	7.22	210
10	11-11.99	10	14	3.88	231
11	12-12.99	2	4	1.11	252
12	13-13.99	2	2	0.55	273

$\sum N = 360$

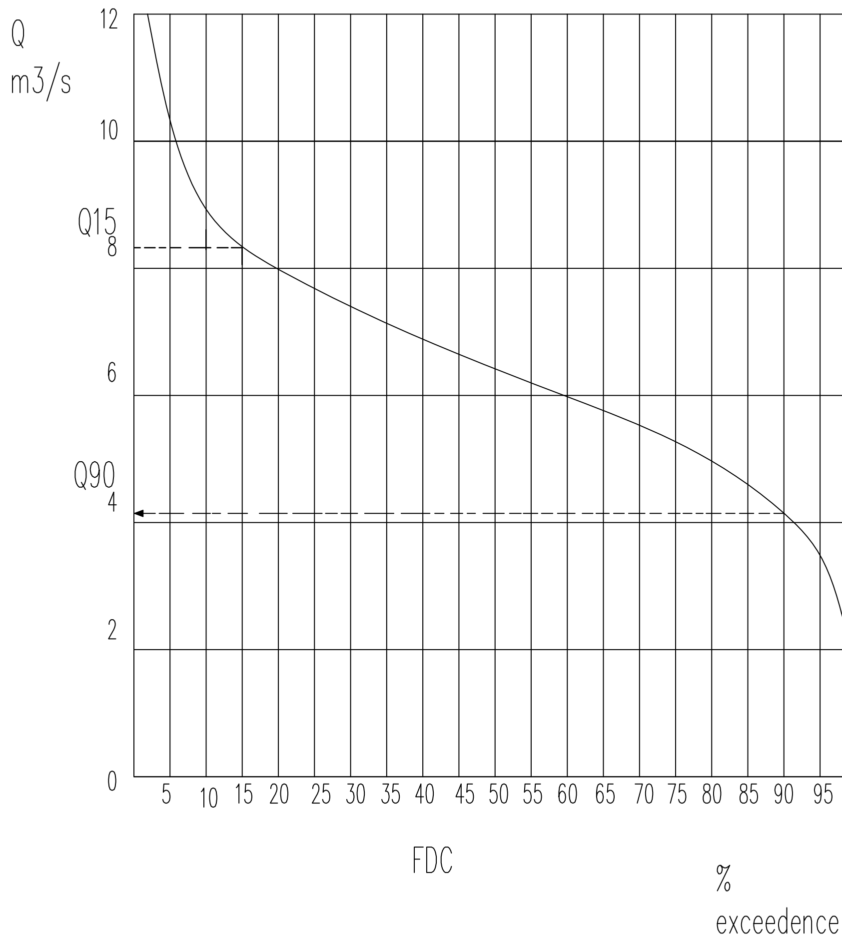

FIG 3.6: FLOW DURATION CURVE

3.7.1 Maximum Flow

Maximum flow Q_{15} is the flow exceeded 15% of the time t. This was determined by reading off the flow corresponding to Q_{15} from the flow duration curve, $Q_{15} = 8.5\ m^3/s$.

3.7.2 Average flow

The mean or average flow, Q_{av}, is gotten by computing the average of the flow data. In this work, this was computed from the mean monthly discharge data (Table 3.1) by dividing the total annual discharge by the total number of years of the data and shown in Table 3.1, $Q_{av} = 6.9\ m^3/s$.

3.7.3 Minimum Flow

The minimum dependable flow Q_{90} is that flow that is available 90% of the time. From the FDC, $Q_{90} = 4.21\ m^3/s$.

3.7.4 Design Flow

The design flow is determined from the techno-economic assessment of the scheme. In this, various flow values greater than the dependable flow are used to compute the corresponding power potential P, the annual energy generated and the cost of turbine. The flow that gives the maximum energy generation with minimum turbine cost is the design flow for the site and the installed capacity is based on this flow (Nwachukwu, 2009).

The availability of a flow is the percentage of time that flow is available for use in a year. The availability of flow therefore corresponds to their percentage exceedence. In computing the design flow, various values corresponding to different percentage exceedences Q_{15}, Q_{20}, Q_{30}, Q_{50}, Q_{60},

Q_{70}, Q_{80} and Q_{90} were read off the FDC (Fig .3.7) to obtain the flow values. This was recorded in the first row of Table 3.6.

The availability of each flow value was determined by selecting the percentage exceedence for that flow and recorded in appropriate column of Table 3.6. These flows were then substituted in the power potential equation to obtain various power potentials. These power potentials were then substituted in the energy equation to obtain the corresponding annual energy generated. These were also recorded as shown in Table 3.6. The flow that gives the maximum energy generation as well as an accepted turbine cost is the design flow Q_d.

3.8 Power potential

Any hydropower scheme, mini, micro, small or large requires water flow, Q, and a drop in height, H,(referred to as a 'head') to produce useful power. It is a power conversion system, absorbing in the form of head and flow, and delivering power in the form of electricity or mechanical shaft power with losses in the form of friction, heating, noise, etc. The conversion equation for power estimation is:

Power input = power output + loss or

Power output = power input x conversion efficiency.

The power input, or total power absorbed by the hydropower scheme, is the gross power, P_{gross} and the power usefully delivered is the net power, P_{net} given by:

$$P_{net} = P_{gross} \times \eta \text{ kw} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.3)$$

where,

η = the overall efficiency of the scheme

$P_{gross} = \rho \times g \times Q \times H_{gross}$ (Kw) ……………………………….(3.4)

where,

ρ = density of water 1000kg/ m^3.

g = acceleration due to gravity, 9.81m^2/s.

Q = flow in m^3/s

H_{gross} = gross head in m.

Substituting the values for ρ, g in the equation (3.4) above, we have:

$P_{net} = 1000 \times 9.81 \times Q \times H_{gross}$

$P_{net} = \eta \times 9810 \times Q \times H_{gross}$ (Watts)

The power produced at the turbine is less than the gross power, because of friction losses in the penstock and the turbine. The power output of the generator is less again, because of inefficiencies in the drive system and generator. Further, transmission involves losses, with the result that the consumer receives only about half the gross power capacity of the site. The overall efficiency of the scheme, η, tends to vary between 0.7 and 0.8

Substituting this range in the above equation gives:

$P_{net} = (6.87 - 7.85) \times Q \times H_{gross}$

The power received by the consumer, P_{net} is given by:

$P_{net} = (7\text{-}8) \times Q \times H_{gross}$ (Kw) …………………………………(3.5)

The above equation is known as the power equation and is used to estimate the power potential of any river.

The power potential is therefore directly proportional to discharge Q and gross head H.

The power potential P, is an indication of the average power production capability of a river and is computed from the power equation

$$P = 7 \times Q \times H \qquad \text{..(3.6)}$$

for this site, H = 3.63m fixed by the maximum head achievable at the culvert location.

$Q_{av} = 6.9\ m^3/s$

$P = 7x\ 6.9 \times 3.63$

$P = 175.33kw$

3.9 Installed Capacity

The installed capacity of a plant is based on the maximum flow which is usually taken as $Q_{15,}$ which is flow exceeded 15% of the time ($Q_{ax} = Q_{15}$) (Makanju, 2003).

The installed capacity for this river at the site selected is estimated on the basis of maximum flow Q_{max}, and this is available flow 15% of the time. $Q_{max} = Q_{15} = 8.5m\ ^3/s$, and by substituting Q with Q_{15} in the power equation above, and H = 3.63m, we obtain:

$P_{ic} = 7x\ Q_{15} \times H$

$P_{ic} = 7x\ 8.5 \times 3.63$

$P_{ic} = 215.98kw$

3.10 Plant capacity factor

The theoretical plant capacity factor is a ratio of the average power production to the installed capacity. This is practically equal to the ratio Q_{av}/ Q_{15}.

The firm or dependable power of a run-of-river (without storage) power plant is estimated on the basis of the flow available at 90% of the time from an unregulated flow duration curve (Nwachukwu, 2009).

3.11 Firm or Dependable Power

This is estimated on the basis of Q_{90}, flow available 90% of the time. $Q_{90} = 4.21$, m^3/s

H = 3.63, Substituting these values in power equation we have,

$P_f = 7 \times 4.21 \times 3.63$

$P_f = 106.97$kw

3.12 Energy Generation

Annual energy

The annual energy generation from a power plant is given by:

$$E = 7 \times Q_{av} \times H \times 8760 \text{ (KWh)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.7)$$

where E = annual energy in KWh

Q_{av} = average discharge, m^3/s.

H = head in meters

From the above equation, annual energy generated will be:

$7 \times 6.9 \times 3.63 \times 8760 = 1,535,882.4$

E = 1,535,882.4kwh.

Table 3.6 Determination of design flow

Discharge (m^3/s)	Q_{15}	Q_{20}	Q_{30}	Q_{40}	Q_{50}	Q_{60}	Q_{70}	Q_{80}	Q_{90}
	8.9	8.6	8.0	7.92	7.52	7.12	6.37	5.30	4.21
Gross head(m)	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63	3.63
Power potential kw	206.53	201.60	198.46	177.41	168.45	159.49	142.69	118.72	94.30
Annual energy (kwh)	267,660.29	348,364.80	514,418.69	613,122.05	727,695.36	826,785.79	862,977.02	820,592.64	733,307.90

3.13 Flood

Flood is an observed increase of stream discharge of a river due to intense rainfall runoff within the catchment of the river.

Flood may inundate farm land, destroy villages and bring about great losses of lives and property if not predicted and controlled.

In mini-hydropower, the flood component of a stream flow is not used for power generation but have to be safely sluiced through the spillway without causing damage to the components of the scheme.

The effect of the peak flood is assessed in terms of the risk and damage it could cause to the scheme and environment if not considered in the design.

The average life span of a small hydropower scheme is between 20 – 30 years and a flood that has the probability of occurrence of once in 10years is selected as the peak flood and used to size the spillway.

A flood flow series is derived from the stream flow data by scanning the data for flood peaks (5 peaks per month). The resulting flood flow series is subjected to frequency analysis log –Pearson III type distribution to determine the frequency factors and estimate the peak flood.

Log- Pearson type III distribution is a distribution in three parameter, coefficient of variation, coefficient of skewness and standard deviation with a limited range in the left direction, unbounded to the right and has a large skew(Nwaogazie, 2006). Since flood flow series commonly indicate considerable skew, this is used as the distribution of flood peaks. The distribution is usually fitted to the logarithms of flood values because this results in lesser skewness. The log-Pearson type III distribution has been adopted as a standard by U.S. Federal Agencies for flood analysis.

A simplified approach suggested by Chow (1951) is used in the frequency analysis. He suggested that most frequency functions applicable to hydrologic sequences, can be resolved to the linearized form

$$X = X_m + KS \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.8)$$

where,

X = flood of a specified probability

X_m = mean of the sample (observed data)

S = standard deviation of the sample

K = frequency factor

The frequency factor is a property of a specific probability distribution at a specified probability level (Nwaogazie, 2006). For the log-Pearson type III distribution a relationship has been developed between the frequency factor and the corresponding return interval and is shown in Appendix B, frequency factors for log-Pearson type III Distribution.

The procedure for analysis according to Nwaogazie (2006) is as follows:

(i) The data are first converted to log values

(ii) Compute the statistical parameters (mean, standard deviation and skewness coefficient, if necessary) from the flood flow series.

(iii) Use the frequency factors for log-Pearson type III distribution, Table A-2(Appendix B)

(iv) For a given return interval, determine the corresponding frequency factor from Table A-2(Appendix B).

(v) Compute the magnitude of the flood using equation (3.8)

(vi) Repeat steps (iv) and (v) for various return intervals and make a frequency plot on a log-normal paper. The flood with a return period of 1 in 10 years is determined. This is the design flood.

The river discharge data were scanned for monthly peak flows to form the flood flow series in column (3) of Table 3.7.Thereafter the statistical parameters mean X_m, standard deviation s and skew coefficient g of the flood flow series were computed and also shown in Table 3.7. For the skew coefficient of -0.570339 (approximately- 0.600) calculated in Table 3.7, the corresponding k- values for log-Pearson type III distribution of specified probabilities of occurrence – 50, 20, 10, 4, 2, 1, 0.5(Table A2, Appendix B) and return periods of 2,5, 10, 25, 50, 100, 200 were selected and recorded in columns 2 and 1 of Table 3.8 respectively.

The flood discharge series was then generated using these k values and recorded in columns 5 of Table 3.8. The flood discharge values Q were then plotted against the probability of occurrence to obtain the log-Pearson type III frequency curve of Fig.3.7.

The design flood of one in ten years is considered adequate for this work and was determined from the frequency curve of Fig.3.7

Table 3.7 Computation of statistical parameters from flood flow series

s/n	Year	Max annual Q	Log Q	LogQ-X_m	(LogQ-$X_m)^2$	(Log Q-$X_m)^3$
1	1988/89	7.48	0.8739	-0.0704	0.0050	-0.000348914
2	1989/90	10.1	1.0043	0.0596	0.0036	0.000211709
3	1990/91	6.14	0.7882	-0.1561	0.0244	-0.003803721
4	1991/92	8.61	0.9350	-0.0093	0.0001	-8.04357E-07
5	1992/93	8.68	0.9385	-0.0058	0.0000	-1.95112E-07
6	1993/94	11.84	1.0734	0.1290	0.0166	0.002146689
7	1994/95	8.53	0.9309	-0.0134	0.0002	-24061E-06
8	1995/96	9.25	0.9661	0.0217	0.0005	1.02183E-05
9	1996/97	9.23	0.9652	0.0208	0.0004	8.99891E-06
10	1997/98	9.28	0.9675	0.0232	0.0005	1.24872E-05

Sum 9.4431 0.0513 -0.001765939

Mean = X_m $\sum(\text{LogQ})/n$ = 0.9443

STD = S = Sqrt $(\sum(\text{logQ}-\text{logQ}_m)^2)$ = 0.075467425

Skew coefficient $g = \dfrac{n\sum(\text{logQ}-\text{logQ}_m)^3}{(n-1)(n-2)(\sigma \text{logQ})^3}$ = -057033972

Table 3.8 Computation of peak flows of Amanyi stream

Return period T years	Percent probability	K	$X=X_m+ks$	$Q=\log^{-1} X$ m^3/s
2	50	-0.099	0.9368	8.6457
5	20	0.8	1.0047	10.1088
10	10	1.328	1.0445	11.0789
25	4	1.939	1.0906	12.3196
50	2	2.359	1.1223	13.2526
100	1	2.755	1.1522	14.1971
200	0.5	3.132	1.1806	15.1565

$S = 0.07546$ $X_m = 0.9443$

FREQUENCY CURVE FIG 3.7

3.14 HYDRAULIC DESIGN

3.14.1 Lay-out of Scheme

The scheme lay out is as shown in Fig.3.2. The scheme consists of a gated spillway at the existing culvert, intake structure, penstock and power house.

3.14.2 Basis of Design

The design principle is to maximize the achievable head at the intake by closing the gates on the spillway. The control sluice gates at the penstock are then opened to allow water flow through the penstock to the turbine. The water exits the power house after turning the turbine through the tailrace. In a low head scheme of this type where every head available is required for power generation, reducing head loss at the intake and penstock as much as possible is of prime importance for maximum power generation. This is overcome in this design by incorporating sluice valves of spherical shape with 0 loss coefficient at the penstock.

Turbulence is usually experienced at the entrance to the penstock and entrance losses will be high if not properly designed.

The hydraulic design therefore involves the sizing of the penstock and trash rack and intake and establishing their relative positions such that the total system head loss is minimized. This increases the efficiency of operation of the scheme.

3.14.3 Design Procedure

The following step by step procedure was adopted in the hydraulic design of the scheme:

1. The components of the scheme were identified and their position located on the topographical map to obtain the layout map of the scheme.

2. A longitudinal section through the intake structure, spillway (culvert), penstock, power house and tail race canal was drawn and the elevations of these structures were established.
3. The maximum upstream water elevation at the intake and the downstream water surface elevation at the tailrace canal were determined as UWL and DWL respectively.
4. The gross head H_g, was determined from the difference between the UWL and DWL, (UWL –DWL) and the net head H_n, determined by limiting the head loss in penstock to 5% of H_g.
5. The power that can be generated by utilizing the design flow Q_d was then determined by substituting $H_n = 0.95H_g$ and $Q = Q_d$ in the power equation,

   $P = 7 \times Q_d \times 0.95\ H_g$.
6. Two number hydroelectric turbines (2no.) were selected to generate the power computed in 5 above by applying a diversity factor of 2 for operational efficiency.
7. The power generated in (1 no.) turbine, PT, was determined rounding off the value to the nearest 25KW.
8. The flow through one turbine, Q_T was then determined from the power equation rearranged: $Q_T = P_T/(7xH_n)$ and substituting $H_n = 0.95H_g$.
9. The head loss per unit length of penstock was determined from $h_f/L = h_f/h_g \times h_g/L$ by substituting $h_t/h_g = 5\%$, h_g and L.
10. The penstock diameter, D, that will convey Q_T to the turbine was then determined by applying re-arranged Manning's equation.

$H_f/L = (g \times n^2 \times Q^2)/D^{5.3}$ and substituting the value for h_f/L computed in 9 above. The next practicable standard pipe size was selected as penstock diameter D.

11. The head loss per unit length of the selected penstock size was again computed as in step (10) and the total head loss in penstock is determined.
12. The permissible velocity through the trash rack was determined from $v = 0.12\sqrt{(2gh)}$ and the head loss across the trash rack determined from $h_{tr} = k_t(t/b)^{4/3}v_0^{\,2}/2g \sin\Phi$.
13. The head loss at the entrance to the penstock was determined by selecting the bell mouth entrance type with the least loss coefficient K = 0.04 (fig. 3.10) and substituting in $h_T = (kxV^2)2g$.
14. The total head loss H_t was then determined as the sum of head losses at intake h_T, trash rack h_{tr} and penstock h_f.
15. The actual net head H_n was now determined from $h_g - H_t$ and actual plant capacity determined from the power equation.
16. $P = 7 \times Q_T \times H_n$.

3.15 Penstock Pipe

The penstock conveys water under pressure to the turbine in the power house. Its hydraulic design therefore, is limited to the determination of the appropriate pipe size that will carry the required quantity of water at a specified elevation to generate the power potential with minimal head loss.

Head loss in penstock is determined from Manning's equation.

Manning's head loss equation is given by:

$$h_f = (Lgn^2Q^2)/D^{5.3} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.9)$$

where,

h_f = head loss in (m)

L = pipe length (m)

g = acceleration due to gravity ($9.81 m/s^2$)

D = pipe internal diameter (m)

Generally, turbines for small, mini and micro hydropower have been standardized and are manufactured in multiples of 25Kw by most turbine manufacturers (Chukwujekwu, 2003). Therefore, by applying a diversity factor of 2, the turbine configuration for this site is 2 x 75Kw.

The flow required to generate 75Kw depends on the net head, H_n, under which it operates and can be determined from the power equation.

$$Q = P/(7xH_g) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.10)$$

Since the scheme is of a very low head the anticipated head loss in the penstock is limited to 5% of the gross head h_g by design.

Therefore,

$H_n = (1\text{-}0.05)H_g$

$= 0.95x3.63$

$H_n = 3.440m$

By substituting H_n= 3.440m in equation (3.3), we have:

$Q = 75/(7x3.440)$

$Q = 3.115$ m3/s.

The head loss per unit length in penstock, h_f/L, can be expressed as:

$H_f/L = h_f/h_g \times h_g/L$, but $h_f/h_g = 0.05$, $L = 50m$ and $h_g = 3.63m$

Therefore,

$H_f/L = 0.05 \times 3.63/50$

$= 0.00576.$

Manning's head loss equation is given by:

$$h_f = (Lgn^2Q^2)/D^{5.3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.11)$$

$n = 0.012$ for welded steel pipe, $h_f/L = 0.00579$

$h_f/L = (9.81x(0.012)^2 \text{ x } (3.12)^2)/D^{5.3}$

$D^{5.3} = 1.9678$

$D = 1.13m$

A 1.2m diameter welded steel pipe penstock is recommended for this site.

Substituting D = 1.2m in equation (3.4), we have:

$h_f/L = (9.81 \text{ x}(0.012)^2 \text{ x}(3.12)^2/(1.2)^{5.3}$

$= 0.0043m$

head loss in 50m length pipe = 0.0043 x50

$h_f = 0.216m.$

3.16 Intake Design

The intake is incorporated with a trash rack to protect the turbine from trash carried in the water. This introduces some losses if not properly aligned and designed.

In order to overcome the effect of turbulence and hence reduce the entrance losses to acceptable levels, the bell mouth shaped intake which has the lowest loss coefficient of 0.004 is adopted in this design. Fig.3.8 shows the loss coefficient for various types of entrance.

The turbulence loss is determined from:

$$h_t = KV^2/2g \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.12)$$

where:

h_t = head loss due to turbulence (m)

K = loss coefficient

V = velocity at the entrance to the penstock (m/s)

Fig 3.8 **Entrance loss coefficient**

3.17 Trash Rack

The trash rack strains damaging materials and yet has enough openings to allow the design flow to pass through without significant head loss. This is achieved by limiting the design velocity of trash rack to the permissible velocity.

The permissible velocity through the trash rack is given by:

$$V = 0.12\sqrt{(2gh)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.13)$$

where:

V = permissible velocity (m/s)

g = acceleration due to gravity (m/s^2)

H = maximum head of water at inlet to penstock (m)

Velocities in the range of 0.2 – 0.5 m/s have been found to result in minimal head loss across the trash rack and have been adopted in sizing the trash rack.

The design area A_{dt} is the clear opening area in the rack through which water passes.

The trash rack is sized from:-

$A_{dt} = Q/V$

where:

A_{dt} = design area of trash rack (m^2).

Q = design flow (m^3/s).

The trash rack is designed based on maximum flow Q_{max}.

Maximum water elevation at culvert = 49.45m

Invert elevation of intake structure = 45.82m

H = (49.45 - 45.82)m.

$H = 3.63$

Substituting this value in equation (3.5) above we have:

$V = 0.12 \text{ x}(2\text{x}9.81\text{x}3.63)^{0.5}$

$V = 0.46$ m/s.

Velocities in the range of 0.2 to 0.5m/s have been found to result in minimal head loss across the trash rack and have been adopted in sizing the trash rack.

The design area A_d , is the clear open area in the rack through which water passes.

The trash rack is sized from:-

$A_d = Q/V$ The trash rack is designed based on maximum flow Q_{max}.

$Q_{max} = 8.9\ m^3/s$

$V\ \ = 0.5m/s$

Therefore,

$A_{dt} = 8.9/0.5m$

$A_{dt} = 17.84m^2$.

3.17.1 Trash Rack Head Loss

The head loss across the trash rack H_{tr} is given by:

$$H_{tr} = K_t(t/b)^{4/3}\ v_o^2/2g\sin\Phi \quad \text{……………………………………}(3.14)$$

where:

H_{tr} = head loss across the trash rack (m)

K_t = trash rack loss coefficient.

t/b = ratio of maximum bar thickness to space between bars

v_o = approach velocity(m/s)

g = gravitational constant (m/s^2)

Φ = angle of bars with the horizontal.

In this design, t =6mm, b =30mm , K_t =0.8, t/b =6/30 =0.2, Φ =75^o.

The head loss h_{tr} = 0.8x $(0.2)^{4/3}$ x($(0.46)^2$/2x9.81)x sin(75^o)

h_{tr} = 0.000387m

This is minimal and can be neglected.

3.18 Entrance Losses

In order to overcome the effect of turbulence and hence reduce the entrance losses to acceptable levels the bell mouth shaped intake which has the lowest loss coefficient of 0.04 is adopted in this design. Fig.3.10 shows the loss coefficient for various types of entrances.

The turbulence losses are determined from:

$H_t = KxV^2/2g$ ………………………………………… (3.15)

$Q_d = 3.11m^3/s$, A = x-sectional area of penstock

A = 22/7 x $(1.2)^2/4 = 1.131m^2$.

Velocity in the pipe is given by:

V = Q/A

= 3.11/1.131

V = 2.74m/s

Substituting this value of V and K = 0.04, in equation (3.15) we have:

$h_t = 0.04x\ (2.74)\ K^3/2\ x\ 9.81$

h_t = 0.013m

Total head loss $H_t = h_f + h_{tr} + h_t$

H_t = 0.216 + 0.000387 + 0.013

H_t = 0.23m

Actual net head $H_n = H_g$ - 0.23 = 3.63 - 0.23

H_n =3.40m

Plant capacity P = 7 x 3.11 x 3.40 = 74.01KW

Therefore, the capacity of (1 no.) turbine of 75Kw is adequate.

3.19 Actual power generated

From Fig. 3.1, Design layout of Small Hydropower Scheme:

Maximum water elevation at culvert = 49.45m

Downstream water surface elevation = 47.26.

Gross head H_g = 49.45 – 45.82

H = 3.63m

From the power equation (3.2) and substituting $H_g = H_n$,

P = 7 x Q x H_n.

P = 7 x 6.37 x 0.95 x 3.63

P = 154.0KW

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Discussion of Results

The flow characteristics of Amanyi stream are presented in Table 4.1.

The maximum flow for Amanyi stream is $8.5m^3/s$. It is used to compute the upper limit range of the installed capacity of the turbine that can be sustained by the stream. The stream discharge beyond this value is not used for power production and therefore is allowed to spill.

The average flow for Amanyi stream is $6.9m^3/s$. This is the average of all the discharge data in the 10year record. It is used to compute the power potential of the river.

The minimum flow for Amanyi stream is $4.21m^3/s$. This flow is available for use 90% of the time and is used to compute the firm/dependable power that can be generated by the river.

The design flow of $6.37m^3/s$ is the turbine flow that will generate maximum annual energy. It is used to compute the installed capacity of the turbine required to generate maximum annual energy.

The power and energy generation capacity of Amanyi stream are represented in Table 4.2.

The maximum achievable head at this site is 3.63m and is fixed by the elevation of the top slab of the existing double cell culvert. The river has a power potential of 175.38kw which is the maximum power that can be generated from the river. The firm/dependable power of 106.41kw can be generated throughout the year and will satisfy over 50% of Neke Uno load demand. The average annual energy generation is the energy generated by the exploitation of the full power potential. The design installed capacity is the capacity of the standardized hydroelectric turbine which when installed will generate power potential. The layout of the Scheme (Fig.3.1) shows the various components of the scheme and their designed locations for the efficient operation of the scheme.

The components represented are, gated spillway (existing culvert), intake structure, penstock, power house, and gate valve. The achievable head for power generation without topographic modification is 3.63m achieved by closing the gate at the spillway to build up the water level to this desired height. Water is allowed to enter the penstock by opening the gate valve at the intake structure. Water then flows through the turbine, generates power and exits the power house through the tail race canal. Another gate valve is introduced at the power house to enable the turbine to be shut down in cases of emergency and a by- pass channel to conduct water away to the tail race is provided. The penstock diameter D = 1.2m.

The 1989/90 hydrograph and base flow separation presented (Table 3.2 and Fig. 3.5) shows the base flow as well as the runoff components of the river discharge.

The percentage contribution of base flow and runoff to the total discharge is 67% and 33% respectively suggesting that the Amanyi stream is substantially sustained by base flow (underground water contribution) and that seasonal variation will not adversely affect the river discharge and power generated especially in the dry season. Sustainability of flow is therefore guaranteed.

Table 4.1: Stream flow characteristics of Amanyi stream

s/n	Description	Output
1	Maximum flow Q_{max}	8.5 m^3/s
2	Average flow Q_{av}	6.9 m^3/s
3	Minimum dependable flow Q_{min}	4.21 m^3/s
4	Design flow Q_d	6.37 m^3/s

Table 4.2: Power and Energy Generation Capacity of Amanyi stream

s/n	Description	Output
1	Firm/dependable Power	102.41kw
2	Power Potential	175.38kw
3	Average Annual Energy Generation	1,536,328.8kwh
4	Actual Power Generated	154kw

4.2 MATCHING DEMAND WITH SUPPLY

4.2.1 Domestic Load Demand

The observed load center within 1.5km radius of the proposed site is Neke Uno community. However, load survey for the entire community was not carried out but was done for only one village which is closest to the proposed project site. A load survey for Umuegwu village which is one of the four villages that make up Neke Uno community revealed that the power demand for the village (Umuegwu) is about 128kw (this is the only domestic load survey that was carried out) .This constitutes about 83% of the actual power generated.

4.2.2 Industrial Load Demand

A survey for the proposed cottage industry for Neke Uno community as shown in Table 4.3 revealed a power requirement of 16kw.

Table 4.3 Industrial load demand for Neke Uno

s/n	Type of industry	Quantity	Power requirement (Kw)
1	Garri processing machine	4	4.5
2	Grinding machine	1	2.5
3	Palm kernel cracker	1	4
4	Maize thresher	1	3
5	Lighting	-	2

From the surveys and analysis, the power generated from Amanyi stream can adequately serve the proposed cottage industry as well as supply 100% of the power requirement for Umuegwu village (one of the villages in Neke-Uno).

However, with scheme improvement (modification of the existing topography), more power can be generated which could possibly serve the remaining three villages.

4.3 Project Cost Estimate

The issue of cost will always be of interest to any project developer or proposer. The developer or proposer would want to know how much the proposed project is expected to cost in order to plan effectively for its development. A small hydropower scheme is one of such projects that require careful planning prior to implementation especially due to the huge financial resources required for its development.

At the conception stage of the project development, assessing the overall project cost is usually an uphill task especially when historical cost data are unavailable. This is the case at this point in time in small hydropower development in Nigeria.

The proposer would want to know for any identified site;

- What power potential it has.
- What is the preferred harnessing method?
- Who are the beneficiaries and how much will it cost to develop the scheme to effectively serve the beneficiaries?

These questions can only be fully answered when the various cost elements associated with development of small hydropower are assessed.

The cost elements associated with small hydropower development are classified into direct cost which include cost of civil works, mechanical/electrical, accessories, miscellaneous equipment and indirect cost which include engineering planning, design, legal cost and interest on loan during construction if applicable (Chukwujekwu, 2003).

The overall project cost is the aggregation of the associated cost of each cost element.

Typical cost models have been developed by UNDP and GEF consultants for Alternate Hydropower Energy Centre (AHEC) of the Indian Institute of Technology, IIT Roorkee.

Based on the outcome of consultancy work on cost of developing small hydropower plants in India and China. The cost elements that make up this model and their percentage contribution to the overall cost of the project are shown in Fig.4.1 The typical cost model approach of project cost assessment is applicable to small hydropower schemes especially at the conception/reconnaissance stage of project development when the total investment cost and project viability is required in order to take a decision on whether to go on with the project or not. It gives the total cost of development of a small hydro scheme especially at the reconnaissance and pre-feasibility stages. This approach has been adopted in estimating the overall project cost in this work.

The cost assessment procedure according to (AHEC) is as follows:

i) Estimate the Design flow Q, Gross head, H, and calculate the power potential from the power equation $P = g \times \rho \times Q \times H \times \eta$(Watts).

ii) Select the type of turbine for the configuration of H and Q above from standard shell diagram Fig. 4.2 and determine the cost of turbine from Table 4.4.

iii) The appropriate typical cost model is selected. High head schemes, are associated with maximum civil works while low head schemes, are associated with minimum civil works.

iv) The cost of the various elements are then computed from the typical cost model by proportion based on the cost of the turbine.

The total cost of the project is the sum of all the cost elements.

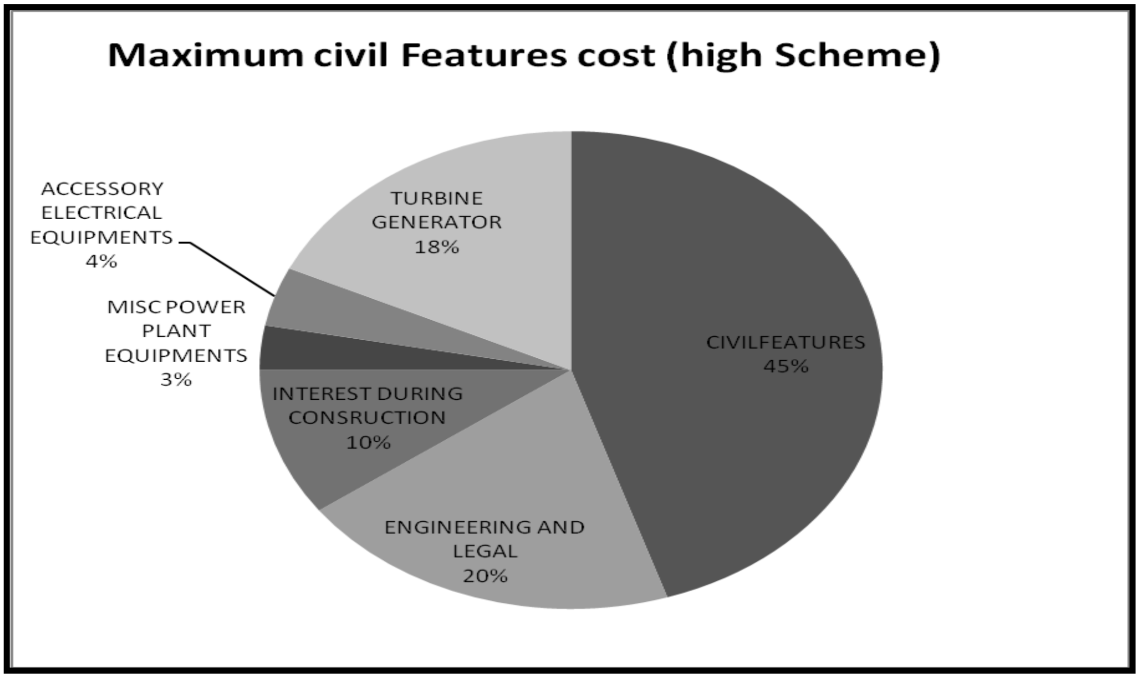

Fig. 4.1a Maximum civil features cost (high head scheme)

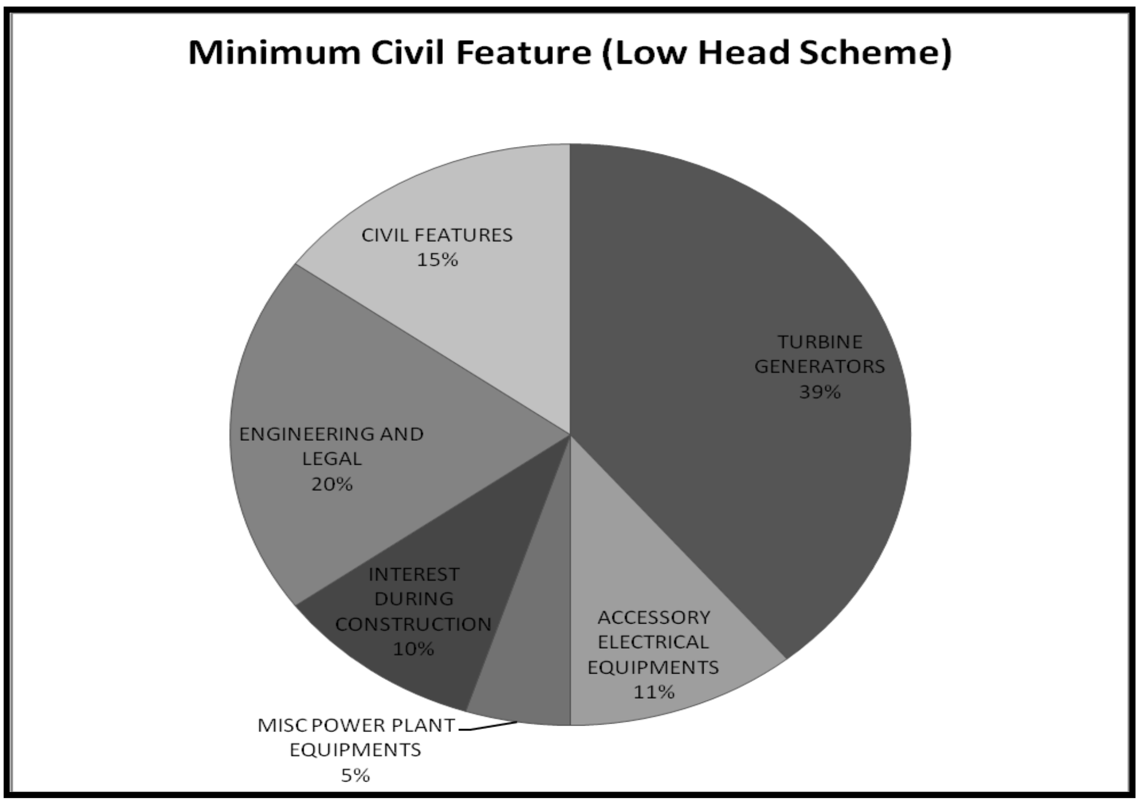

Fig. 4.1b Minimum civil features cost (low head scheme)

(Source; Alternate Hydro energy Centre, 2007)

FIG 4.2 CHART 4 TURBINE

4.4 Actual Project Cost

From standard shell diagram Fig .4.2, for Q = 6.37m^3/s and H =3.5m, and P =150kw, the turbine type is axial flow (propeller). Provide 2No.x75kw hydroelectric turbines.

From Table 4.4, the cost of (1No.) 75kw turbine by interpolation and taking the upper limit of cost is $85,000.00. Therefore, (2No.) turbines will cost 2x 85,000.00 = $170,000.00. From the typical cost model associated with low head, i.e. with minimum civil features, *the turbine and generator cost is 39% of the total project cost. The total cost by proportion is $170,000.00/0.39 = $435,897.43 as shown in Table 4.5. The cost of the various components were determined by applying their percentage contribution to the total cost of the project.

Table 4.4 Costs of turbines in units of $1000 US (excludes alternator cost)

Shaft Power Kw	Cross flow	Francis	Single-jet pelton	Multi-jet pelton	Turgo	Propeller
2	1 -2	4-6	1-4	1-3	2-4	4-6
5	2-6	8-10	2-8	2-6	5-8	8-10
10	2-10	15-20	2-15	2-10	8-14	15-20
20	5-30	25-70	5-20	5-15	12-20	20-30
50	5-30	25-70	5-50	5-30	35-50	25-70
100	30-50	40-100	40-80	15-60	55-80	40-100
150	50-80	60-120	30-80	80-100	60-120	

(Source: Alternate hydro energy center , 2007)

Table 4.5 Total Cost of proposed Project.

Description	% contribution to Total Cost	AMOUNT	
		$	₦
Civil features	15	65,384.61	9,807,691.50
*turbine and generator	39	170,000.00	25,500,000.00
Accessory Electrical Equipment	11	47,948.72	7,192,308.00
Miscellaneous Power Plant Equipment	5	2,397.43	359,614.50
Interest during construction	10	43,589.74	6,538,461.00
Engineering and legal	20	87,179.49	13,076,923.50
Total cost		435,897.43*	65,384,614.50

Conversion rate: 1$ =N150, *O & M Cost is tied to life span of turbine and generator.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Amanyi stream is located on a flat terrain with practically very low head drop. The stream is perennial in nature and greatly sustained by underground water recharge. The average width of the river is large such that the cost of introducing a civil structure across the river for small hydropower development will be uneconomically high. The existing triple cell box culvert at the site of study is amenable to modification and has been adopted as the spillway for the scheme by providing a gate at the culvert.

This would result in an overall reduction in cost of project when compared to a similar site without an existing culvert or similar structure.

The topography of the study area is generally flat and higher heads required for higher power generation can only be achieved by modifying the topography.

Amanyi stream has a power potential of 175Kw which is less than 1Mw, and therefore classified as a mini hydropower. It has a maximum discharge of 8.5m^3/m, average discharge of 6.9m^3/s, minimum discharge of 4.27m^3/s and a design discharge of 6.37m^3/s. These flows can be harnessed to generate power.

The power demand for Umuegwu village (one of the villages in Neke Uno) is 128kw and the industrial demand is 16kw, which is about 83% and 10% respectively of the actual power generation. The maximum achievable power without modifying the topography at a head of 3.63m is 154kw.

The project cost estimate for developing the hydropower potential of Amanyi stream is ₦65, 384, 614.50.
The load centers according to International Standard must be within 1.5km radius of the scheme in order to reduce to the barest minimum, transmission losses.

Generally, higher powers are possible downstream due to envisaged higher catchment area and resulting higher river discharge.
Amanyi stream is suitable for a Low-Head Mini Hydropower Development.

5.2 Recommendations

The current power crisis in Nigeria can be tackled by the development of small/mini and micro hydroelectric schemes especially where the resources are available. Small hydropower schemes guarantee uninterruptible power supply because of the sustainable flow of the water resource on which the design is based.
Amanyi River is substantially sustained by ground water flow and the nearest load centre, Neke Uno community, is within 1.5km radius of the scheme. It is therefore recommended as follows:
That:

1. The Mini Hydropower Potential of Amanyi stream at the selected study site be developed.
2. Extensive load survey should be carried out for Neke Uno and the nearest community, Mbu, to determine the load requirements and allocation plan.

3. Project Cost Estimate for the development of Small Hydropower should include operation and maintenance cost tied to the life span of turbine and generator to be used.
4. The United Nations Industrial Development Organization, UNIDO, could be approached for assistance with funding for the development of the scheme through public private partnership agreement.
5. Further study should be carried out on Amanyi stream using the method of extrapolation to obtain more data.

5.3 Contribution to knowledge

The study assessed the flow characteristics of Amanyi stream and established its potential for hydropower development for Neke Uno community which has not been done before now. This work serves as a pilot scheme and any stream anywhere with similar features as Amanyi stream will also have similar hydropower potentials.

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APPENDIX A

TABLE A1 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1988/89**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	3.90	3.85	3.60	3.90	3.80	5.10	6.30	6.00	5.90	5.40	4.15	4.05
2	3.90	3.70	3.60	3.90	4.00	5.00	6.30	6.00	5.90	5.40	4.15	4.05
3	3.90	3.70	3.60	3.85	4.00	5.20	6.30	5.90	5.80	5.20	4.10	4.05
4	3.90	3.70	3.60	3.60	4.00	5.00	6.30	5.90	5.90	5.20	4.10	3.90
5	3.90	3.70	3.60	3.60	4.10	5.00	6.30	5.90	5.90	5.20	4.10	4.05
6	3.90	3.70	3.60	3.60	4.10	5.00	6.30	5.90	5.90	5.15	4.10	3.90
7	3.90	3.70	3.60	3.60	4.10	5.20	6.00	5.90	5.80	5.10	4.10	3.90
8	3.90	3.70	3.60	3.60	3.80	5.10	5.90	6.00	5.60	5.00	4.10	3.90
9	3.90	3.70	3.60	3.60	4.00	5.70	6.00	6.00	5.60	4.90	4.10	3.90
10	3.60	3.70	3.60	3.50	4.00	5.10	6.00	6.00	5.60	4.90	4.15	4.05
11	3.85	3.70	3.60	3.70	4.10	5.10	6.00	6.10	5.60	4.90	4.10	4.05
12	3.80	3.65	3.60	3.70	4.10	5.10	6.30	5.90	5.60	4.90	4.10	3.90
13	3.50	3.65	3.60	3.70	4.10	5.10	6.30	5.90	5.60	4.90	4.10	3.90
14	3.50	3.60	3.60	3.50	4.10	5.30	6.30	5.90	5.80	4.88	4.10	3.85
15	3.85	3.60	3.60	3.50	4.10	5.30	6.30	5.90	5.60	4.88	4.10	3.85
16	3.85	3.64	3.65	3.50	4.10	5.40	6.30	5.90	5.60	4.90	4.20	3.85
17	3.50	3.64	3.65	4.10	4.10	5.30	6.30	5.85	5.60	4.70	4.10	3.85
18	3.85	3.64	3.65	4.10	4.10	5.30	6.30	5.85	5.60	4.70	4.10	3.85
19	3.85	3.64	3.70	4.10	4.10	5.40	6.20	5.85	5.60	4.70	4.10	3.80
20	3.85	3.64	3.60	4.10	4.20	5.60	6.10	5.85	5.60	4.70	4.10	3.80
21	3.80	3.64	3.70	4.10	4.30	5.80	6.10	5.85	5.60	4.60	4.30	3.80
22	3.80	3.64	3.90	4.00	4.55	5.60	6.00	5.85	5.60	4.60	4.10	4.80
23	3.85	3.64	3.90	4.10	4.60	5.80	6.00	5.85	5.60	4.50	4.10	4.80
24	3.85	3.64	3.90	4.00	4.65	5.60	6.00	5.85	5.60	4.60	4.00	4.80
25	3.85	3.64	3.90	4.10	5.00	5.40	6.00	5.85	5.60	4.30	4.00	4.70
26	2.30	3.64	3.90	4.10	4.90	5.90	6.10	5.80	5.40	4.30	4.10	4.70
27	3.85	3.64	4.00	4.10	4.65	6.40	6.00	5.85	5.40	4.20	4.10	4.70
28	3.85	3.60	3.90	4.10	4.90	6.30	6.00	5.85	5.60	4.20	4.00	4.70
29	3.85	3.60	3.90	3.80	4.60	6.30	5.90	5.90	5.40	4.20	-	4.70
30	3.80	3.60	3.90	3.70	4.60	6.30	5.90	5.90	5.40	4.15		4.70
31	-	3.60	-	3.80	4.95	-	6.00	-	5.40	4.15		4.70

TABLE A2 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1989/90**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	4.70	5.20	3.20	400	3.70	5.10	6.20	6.00	5.90	5.40	4.15	4.05
2	4.75	5.20	3.30	4.00	4.00	5.00	6.60	6.00	5.90	5.40	4.15	4.05
3	4.75	5.10	3.10	4.00	4.10	5.20	10.0	6.90	5.80	5.20	4.20	4.05
4	4.75	3.20	3.30	4.10	4.40	5.00	10.2	6.90	5.90	5.20	4.20	3.90
5	4.75	4.40	4.50	4.10	4.10	5.00	6.20	7.90	5.90	6.20	4.20	4.05
6	4.75	4.30	4.70	4.10	4.10	5.00	6.40	7.90	5.90	6.15	4.20	3.90
7	4.75	4.20	4.65	4.15	4.10	5.20	7.00	7.00	5.80	6.10	4.20	3.90
8	4.80	4.20	4.50	4.15	3.80	5.10	7.90	7.00	6.80	6.00	4.20	3.90
9	3.80	4.20	4.50	4.15	4.00	5.70	7.70	7.00	6.80	6.90	4.20	3.90
10	4.80	4.10	4.10	4.00	4.00	5.10	7.70	7.00	6.80	6.90	4.15	4.05
11	4.80	4.20	4.30	4.00	4.10	5.10	7.30	7.10	6.80	6.90	4.10	4.05
12	4.80	4.10	4.50	3.90	4.10	5.10	7.40	7.20	6.80	6.90	4.10	3.90
13	5.80	4.20	4.80	3.90	4.10	5.10	7.80	7.10	6.80	6.90	4.10	3.90
14	5.80	4.10	3.80	3.90	4.50	5.30	8.10	7.90	6.80	6.88	4.10	3.85
15	5.80	4.30	4.10	4.10	5.50	5.30	8.20	7.00	6.60	6.88	4.10	3.85
16	5.60	4.20	4.10	4.10	5.50	5.40	8.60	7.20	5.60	6.90	4.20	3.85
17	5.50	4.20	4.20	4.10	5.10	5.30	8.60	6.70	5.60	6.70	4.10	3.85
18	5.40	4.20	4.10	4.10	5.10	5.30	8.70	6.00	5.60	6.70	4.10	3.85
19	5.40	4.20	4.20	4.10	5.10	5.40	8.70	6.00	5.60	6.70	4.10	3.80
20	5.50	4.10	4.10	4.10	5.20	5.60	8.10	6.00	5.60	6.70	4.10	3.80
21	5.40	3.10	4.00	4.10	5.30	5.80	9.10	5.80	5.60	6.60	4.30	3.80
22	5.30	3.10	3.90	4.00	5.55	5.60	9.00	6.90	5.60	6.60	4.10	3.80
23	5.30	3.10	4.10	4.10	5.60	5.80	9.00	6.00	6.60	6.50	4.10	3.80
24	5.30	3.10	4.70	4.00	4.65	5.60	9.00	6.00	6.60	6.60	4.00	3.80
25	5.30	3.10	4.00	4.10	5.00	5.40	9.00	5.80	6.60	6.30	4.00	3.70
26	5.30	3.00	3.90	4.70	4.90	5.90	9.10	6.90	6.40	4.30	4.10	3.70
27	5.30	3.22	4.00	4.40	4.65	6.40	9.00	6.00	6.40	4.20	4.10	3.70
28	5.30	3.10	3.90	4.10	4.90	6.30	9.00	6.00	6.60	4.20	4.00	3.70
29	5.30	3.10	3.90	3.80	4.60	6.30	9.90	5.90	6.40	4.20	-	3.70
30	5.20	3.10	3.90	3.70	4.60	6.30	5.90	5.90	6.40	4.15		3.70
31	-	3.20	-	3.80	4.95	-	6.00	-	6.40	4.15		3.70

TABLE A3 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1990/91**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	3.70	2.20	3.00	3.90	3.80	5.10	6.30	6.00	5.90	4.40	4.15	4.90
2	3.50	2.20	3.30	3.90	4.00	5.00	6.30	6.00	5.90	4.40	4.15	4.90
3	3.50	2.10	3.10	3.00	4.10	5.20	6.10	5.90	5.80	5.20	4.20	4.90
4	3.50	2.20	3.30	3.70	4.40	5.00	6.20	5.90	5.90	5.20	4.20	3.90
5	2.70	2.40	3.50	3.60	4.10	5.00	6.20	5.90	5.90	5.20	4.20	3.60
6	2.70	2.30	3.70	3.60	4.20	5.00	6.20	5.90	5.90	5.15	4.20	4.00
7	2.70	2.20	3.70	3.60	4.20	5.20	6.00	6.00	5.80	5.10	4.20	4.90
8	2.70	2.20	3.20	3.60	4.20	5.10	5.90	6.00	5.80	5.00	4.20	4.40
9	2.60	2.20	3.20	3.60	4.20	5.70	5.90	6.00	5.80	5.90	4.20	4.40
10	2.60	2.10	3.10	3.50	4.20	5.10	5.90	6.00	5.80	5.90	4.15	4.05
11	2.50	2.20	3.30	3.60	4.20	5.10	5.90	6.10	5.80	5.90	4.10	4.05
12	2.50	2.10	3.50	3.50	4.20	5.10	5.80	7.20	5.80	5.90	4.10	3.90
13	2.50	2.20	3.80	3.50	4.20	5.10	5.80	7.10	5.80	5.90	4.60	3.90
14	2.50	2.10	3.80	3.50	4.20	5.30	5.80	6.90	5.80	5.88	4.60	3.85
15	2.50	2.30	3.10	3.10	4.20	5.30	5.80	6.00	5.60	5.88	4.60	3.85
16	2.60	2.20	3.10	3.10	4.50	5.40	5.80	6.20	5.60	5.90	4.60	3.85
17	2.50	2.20	3.20	3.10	4.50	5.30	5.80	6.70	5.60	5.70	4.60	3.85
18	2.40	2.20	3.10	3.10	4.50	5.30	5.80	6.00	5.60	5.70	4.60	3.85
19	2.40	2.20	3.20	3.10	4.50	5.40	5.70	6.00	5.60	5.70	4.60	3.80
20	2.50	2.10	3.10	3.10	4.70	5.60	6.10	6.00	5.60	5.70	4.60	3.80
21	2.40	2.10	3.00	3.10	4.70	5.80	6.00	5.80	5.60	5.60	4.60	3.80
22	2.30	2.10	3.90	3.00	4.80	5.60	6.00	6.90	5.60	5.60	4.60	3.80
23	2.30	2.10	3.10	4.10	4.80	5.80	6.00	6.00	5.60	5.50	4.90	3.80
24	2.30	2.10	3.70	4.00	4.85	5.60	6.00	6.00	5.60	5.60	4.90	3.80
25	2.30	2.10	3.00	4.10	5.00	5.40	6.10	5.80	4.60	5.30	4.90	3.70
26	2.30	2.00	3.90	4.70	5.15	5.90	6.00	6.90	4.40	5.30	4.90	3.70
27	2.30	2.22	2.00	4.40	5.15	6.40	6.00	6.00	4.40	5.20	4.90	3.70
28	2.30	2.10	2.90	4.10	5.15	6.30	5.90	6.00	4.60	5.20	4.90	3.70
29	2.30	2.10	2.90	3.80	5.18	6.30	5.90	5.90	4.40	5.20	-	3.75
30	2.20	2.10	2.90	3.80	5.20	6.30	6.00	5.90	4.40	5.15		3.75
31	-	2.20	-	3.80	5.10	-		-	4.40	5.15		3.75

TABLE A4 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1991/92**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	3.75	3.20	4.00	4.80	5.80	7.10	8.20	7.90	7.90	7.40	7.15	7.05
2	3.75	3.20	4.30	4.85	5.00	7.00	8.60	7.90	7.90	7.40	7.15	7.05
3	3.75	3.10	4.10	4.70	5.10	7.20	8.10	7.90	7.80	7.20	7.20	7.05
4	4.75	4.20	4.30	4.70	5.40	7.00	8.20	7.90	7.90	7.20	7.20	7.90
5	4.75	4.40	4.50	4.85	6.10	7.00	8.20	7.90	7.90	7.20	7.20	7.05
6	4.75	4.30	4.70	4.85	6.10	8.00	8.40	7.90	7.90	7.15	7.20	7.90
7	4.80	4.20	4.70	4.90	6.10	8.20	8.00	7.00	7.80	7.10	7.20	7.90
8	4.80	4.20	4.20	4.90	6.80	8.10	8.90	7.00	7.80	7.00	7.20	7.90
9	4.80	4.20	4.20	4.00	6.00	8.70	8.70	7.00	7.80	7.20	7.20	7.90
10	4.80	4.10	4.10	4.00	6.00	8.10	8.70	7.00	7.80	7.20	7.15	6.05
11	4.80	4.20	4.30	4.00	6.10	8.10	8.30	7.10	7.20	7.20	7.10	6.05
12	4.50	4.10	4.50	4.00	6.10	8.10	8.40	7.20	7.20	7.20	7.10	6.90
13	4.50	4.20	4.80	4.10	6.10	8.10	8.80	7.10	7.20	7.20	7.10	6.90
14	4.50	4.10	4.80	4.10	6.50	8.30	8.10	6.90	7.20	7.28	7.10	6.85
15	4.50	4.30	4.10	4.10	6.50	8.30	8.20	7.00	7.20	7.28	7.10	6.85
16	4.60	4.20	4.10	4.10	6.50	8.40	8.60	7.20	7.20	7.20	7.20	6.85
17	4.50	4.20	4.20	4.10	6.10	8.30	8.60	7.70	7.20	7.70	7.10	6.85
18	4.40	4.20	4.50	4.10	6.10	8.30	8.70	7.00	7.20	7.70	7.10	6.85
19	4.40	4.20	4.50	4.10	6.10	8.40	8.70	7.00	7.20	7.70	7.10	6.80
20	4.50	4.10	4.50	4.10	6.20	8.60	8.10	7.00	7.20	7.70	7.10	6.80
21	4.40	4.10	4.50	4.10	6.30	8.80	8.10	7.80	7.20	7.60	7.30	6.80
22	4.30	4.10	4.50	4.00	6.55	8.60	8.00	7.90	7.20	7.60	7.10	6.80
23	4.30	4.10	4.10	4.10	6.60	8.80	8.00	7.00	7.20	7.50	7.10	6.80
24	4.30	4.10	4.70	4.00	6.65	8.60	8.00	7.00	7.20	7.60	7.00	6.80
25	3.30	4.10	4.00	4.10	6.00	8.40	8.00	7.80	7.20	7.30	7.00	6.70
26	3.30	4.00	4.90	4.70	6.90	8.90	8.10	7.90	7.20	7.30	7.10	6.70
27	3.30	4.22	4.00	5.40	6.65	8.40	8.00	7.00	7.20	7.20	7.10	6.70
28	3.30	4.10	4.90	5.10	6.90	8.30	8.00	7.00	7.20	7.20	7.00	6.70
29	3.30	4.10	4.90	5.80	6.60	8.30	8.90	7.90	7.20	7.20	-	6.70
30	3.20	4.10	4.90	5.70	6.60	8.30	8.90	7.90	7.20	7.15		6.70
31	-	4.20	-	5.80	6.95	-	8.00	-	7.20	7.15		6.70

TABLE A5 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1992/93**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	6.70	6.20	7.20	6.90	6.60	6.90	6.60	8.20	8.90	8.40	7.05	7.00
2	6.70	6.20	7.30	6.90	6.60	6.90	6.60	8.20	8.90	8.40	7.05	7.00
3	6.70	6.10	7.10	6.90	6.50	6.90	6.60	8.20	8.80	8.20	7.05	7.05
4	6.70	6.20	7.30	6.90	6.50	6.90	6.60	8.20	8.40	8.20	7.05	7.90
5	6.70	6.70	7.50	6.90	6.50	6.90	6.60	8.20	8.40	8.20	7.05	7.05
6	6.70	6.70	7.60	6.90	6.50	6.90	7.60	8.60	8.40	8.15	7.05	7.90
7	6.70	6.70	7.60	6.60	6.50	7.20	7.50	8.60	8.40	8.10	7.05	7.90
8	6.70	6.70	7.60	6.60	6.50	7.10	7.50	8.60	8.40	7.00	7.00	7.90
9	6.60	6.70	7.50	7.00	6.45	7.10	7.50	8.60	8.40	7.10	7.00	7.90
10	6.60	6.70	7.50	7.50	6.45	7.10	7.50	8.60	8.40	7.10	7.05	7.05
11	6.50	6.70	7.50	7.60	7.10	7.10	7.50	8.60	8.40	7.10	7.00	7.05
12	6.50	6.70	7.50	7.50	7.40	7.10	7.45	8.60	8.40	7.10	7.00	7.90
13	6.50	6.70	7.80	7.50	7.50	7.10	7.45	8.60	8.40	7.10	7.00	7.90
14	6.50	6.70	7.80	7.50	7.50	7.30	7.40	8.60	8.40	7.18	7.00	7.85
15	6.50	6.70	7.00	7.10	7.50	7.30	7.40	8.60	8.40	7.18	7.00	7.85
16	6.60	6.70	7.00	7.10	7.10	7.40	7.40	8.60	8.40	7.10	7.00	7.85
17	6.50	6.70	6.00	7.10	7.10	7.30	8.40	8.60	8.40	7.10	7.00	7.85
18	6.40	6.70	6.10	7.10	7.10	7.30	8.40	8.60	8.40	7.10	7.00	7.85
19	6.40	6.70	6.10	7.10	7.20	7.40	8.40	8.60	8.40	7.10	7.00	7.80
20	6.50	6.70	6.10	6.10	7.30	7.60	8.40	8.60	8.40	7.10	7.95	7.80
21	6.40	6.40	6.10	6.10	7.55	7.80	8.40	8.60	8.40	7.10	795	7.80
22	6.30	6.40	6.10	6.00	7.60	7.60	8.40	8.60	8.40	7.10	7.95	7.80
23	6.30	7.70	6.10	6.10	7.65	7.80	8.35	8.60	8.40	7.10	7.95	7.80
24	6.30	7.70	6.10	6.00	7.00	6.60	8.20	8.60	8.40	7.10	7.95	7.80
25	6.30	7.70	6.00	6.70	6.90	6.60	8.20	8.65	8.40	7.10	7.00	7.70
26	6.30	7.70	6.90	6.60	6.65	6.60	8.20	8.95	8.40	7.10	7.00	7.70
27	6.30	7.72	6.00	6.60	6.90	6.60	8.20	8.95	8.40	7.10	7.00	7.65
28	6.30	7.70	6.90	6.60	6.60	6.60	8.20	8.90	4.40	7.00	7.00	7.65
29	6.20	7.10	6.90	6.60	4.60	6.60	8.20	8.90	8.40	7.00	-	7.65
30	-	7.10	6.90	6.60	6.95	6.60	8.20	-	8.40	7.05		3.65
31		7.20	-			-			8.40	7.15		7.60

TABLE A6 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1993/94**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	7.60	8.40	3.60	9.90	9.80	11.10	6.30	6.00	5.90	5.40	4.15	4.05
2	7.60	8.40	3.60	9.90	9.80	11.30	6.30	6.00	5.90	5.40	4.15	4.05
3	7.30	8.40	3.60	9.80	9.80	11.3	6.20	5.90	5.80	5.20	4.20	4.05
4	7.30	8.40	3.60	9.80	9.80	11.3	6.20	5.90	5.90	5.20	4.20	3.90
5	7.30	8.45	3.50	9.80	9.80	11.3	6.20	5.90	5.90	5.20	4.20	4.05
6	7.30	8.45	3.70	9.80	9.80	11.3	6.40	5.90	5.90	5.15	4.20	3.90
7	7.30	8.45	3.70	9.80	9.80	11.3	6.00	6.00	5.80	5.10	4.20	3.90
8	7.30	8.45	3.70	9.80	9.85	11.3	5.90	6.00	5.80	5.00	4.20	3.90
9	7.30	8.50	3.70	9.80	9.85	11.3	6.70	6.00	5.80	4.90	4.20	3.90
10	7.30	8.50	3.70	9.80	9.85	11.3	6.70	6.00	5.80	4.90	4.15	4.05
11	7.30	8.50	3.70	9.80	9.80	11.3	6.30	6.10	5.80	4.90	4.10	4.05
12	7.30	8.50	3.75	9.80	9.80	11.3	6.40	6.20	5.80	4.90	4.10	3.90
13	7.30	8.50	3.80	9.80	9.80	11.3	5.80	6.10	5.80	4.90	4.10	3.90
14	7.30	8.50	3.80	9.80	9.80	11.3	6.10	6.90	5.80	4.88	4.10	3.85
15	7.30	8.50	4.10	9.10	9.80	11.3	6.20	6.00	5.60	4.88	4.10	3.85
16	7.30	8.50	4.10	9.10	9.80	11.3	6.60	6.20	5.60	4.90	4.20	3.85
17	7.30	8.40	4.20	9.10	9.80	11.0	6.60	6.70	5.60	4.70	4.10	3.85
18	7.30	8.40	4.10	9.10	9.95	11.0	6.70	6.00	5.60	4.70	4.10	3.85
19	7.30	8.5	4.20	9.10	9.95	11.0	6.70	6.00	5.60	4.70	4.10	3.80
20	7.30	8.45	4.10	9.10	9.95	11.0	6.10	6.00	5.60	4.70	4.10	3.80
21	7.30	8.45	4.00	9.10	9.95	11.0	6.10	5.80	5.60	4.60	4.30	3.80
22	7.30	8.45	3.90	9.00	9.95	11.0	6.00	6.90	5.60	4.60	4.10	3.80
23	7.30	8.40	4.10	9.10	9.90	11.0	6.00	6.00	5.60	4.50	4.10	3.80
24	7.30	8.40	4.10	9.00	9.95	11.0	6.00	6.00	5.60	4.60	4.00	3.80
25	7.30	8.40	4.00	9.10	9.00	11.0	6.00	5.80	5.60	4.30	4.00	3.70
26	7.30	8.40	3.90	9.10	9.90	11.0	6.10	6.90	5.40	4.30	4.10	3.70
27	7.30	8.40	4.00	9.40	9.95	11.0	6.00	6.00	5.40	4.20	4.10	3.70
28	7.30	8.40	3.90	9.40	9.95	11.0	6.00	6.00	5.60	4.20	4.00	3.70
29	7.30	8.40	3.90	9.40	9.95	11.0	5.90	5.90	5.40	4.20	-	3.70
30	7.30	8.40	3.90	9.45	9.90	11.0	5.90	5.90	5.40	4.15		3.70
31	-	8.40	-	9.40	9.10	-	6.00	-	5.40	4.15		3.70

TABLE A7 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1994/95**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	6.00	6.20	6.00	7.80	7.70	7.95	8.20	8.00	8.90	8.40	7.15	8.05
2	6.00	6.20	6.30	7.80	7.00	7.00	8.00	8.00	8.90	8.40	7.15	8.05
3	6.00	6.10	6.10	7.70	7.10	7.20	8.10	8.90	8.80	8.20	7.20	8.05
4	6.00	6.20	6.30	7.70	7.40	7.00	8.00	8.90	8.90	8.20	7.20	8.90
5	6.00	6.40	6.50	6.60	7.10	7.00	8.20	8.90	8.90	8.20	7.20	8.05
6	6.00	6.30	6.70	6.60	7.10	7.00	8.00	8.90	8.90	8.15	7.20	8.90
7	6.00	6.20	6.70	6.60	7.10	7.20	8.00	8.00	8.80	8.10	7.20	8.90
8	6.00	6.20	6.20	6.60	7.80	7.10	8.10	8.00	8.80	7.00	7.20	8.90
9	6.00	6.20	6.20	6.60	7.90	7.70	8.10	8.00	8.80	7.90	7.20	8.90
10	6.00	6.10	6.10	6.50	7.90	7.10	8.10	8.00	8.80	7.90	8.15	8.05
11	6.00	6.20	6.30	6.60	7.90	7.10	8.10	8.10	8.80	7.90	8.10	8.05
12	6.00	6.10	6.50	7.50	7.90	7.10	8.10	8.20	8.80	7.90	8.10	8.90
13	6.05	6.20	6.80	7.50	7.90	8.10	8.10	8.10	8.80	7.90	8.10	8.90
14	6.05	6.10	6.80	7.50	7.90	8.30	8.10	8.90	8.80	7.88	8.10	8.85
15	6.00	6.30	6.90	7.10	7.90	8.30	8.10	8.00	8.60	7.88	8.10	7.85
16	6.00	6.20	6.90	7.10	7.90	8.40	8.10	8.20	8.60	7.90	8.20	7.85
17	6.00	6.20	6.90	7.10	7.90	8.30	8.10	8.70	8.60	7.70	8.10	7.85
18	6.00	6.20	6.90	7.10	7.90	8.30	8.10	8.00	8.60	7.70	7.10	7.85
19	6.00	6.20	6.90	7.10	7.90	8.40	8.10	8.00	8.60	7.70	7.10	7.80
20	6.00	6.10	6.90	7.10	7.90	7.60	8.00	8.00	8.60	7.70	7.10	7.80
21	6.00	6.10	6.90	7.10	7.90	7.80	8.00	8.80	8.60	7.60	7.30	7.80
22	6.00	6.10	6.90	7.00	7.95	7.60	8.00	8.90	8.60	7.60	7.10	7.80
23	6.05	6.10	6.90	7.10	7.6\0	7.80	8.00	8.00	8.60	7.50	7.10	7.80
24	6.05	6.10	6.90	7.00	7.65	7.60	8.00	8.00	8.60	7.60	7.00	7.80
25	6.05	6.10	6.00	7.10	7.00	7.40	8.00	8.80	8.60	7.30	7.00	7.80
26	6.05	6.00	6.90	7.70	7.90	7.90	8.00	8.90	8.40	7.30	7.10	7.80
27	6.05	6.22	6.00	7.40	7.65	7.40	8.00	8.00	9.40	7.20	7.10	7.80
28	6.00	6.10	6.90	7.10	7.90	7.30	8.00	8.00	9.60	7.20	7.00	7.80
29	6.00	6.10	6.90	7.80	7.60	6.30	8.00	8.90	9.40	7.20	-	7.80
30	6.00	6.10	7.90	7.70	7.60	6.30		8.90	9.40	7.15		7.80
31	-	6.20	-	7.80	7.95	-		-	9.40	7.15		7.80

TABLE A8 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1995/96**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	7.40	7.00	7.20	7.00	7.80	910	9.20	9.00	9.90	7.40	8.15	8.65
2	7.40	7.00	7.20	7.00	7.00	9.00	9.60	9.00	9.90	7.40	8.15	8.05
3	7.40	7.00	7.20	7.10	7.10	9.20	9.10	8.90	9.80	7.20	8.20	8.05
4	7.40	7.00	7.20	7.10	7.40	9.00	9.20	8.90	8.90	7.20	8.20	8.90
5	7.40	7.00	7.10	7.15	7.10	9.00	9.20	8.90	8.90	7.20	8..20	8.05
6	7.40	7.00	7.10	7.15	8.10	9.00	9.40	8.90	8.90	7.15	820	8.90
7	7.40	7.00	7.10	7.20	8.10	9.20	9.00	88.00	8.80	7.10	8.20	8.90
8	7.40	7.00	7.10	7.20	7.80	9.10	9.90	8.80	8.80	7.00	8.20	8.90
9	7.40	7.20	7.10	7.20	7.60	9.70	9.70	8.00	8.80	7.90	8.15	8.90
10	7.40	7.10	7.10	7.20	8.00	9.10	9.70	8.00	6.80	7.90	8.10	8.05
11	7.40	7.20	7.10	7.15	8.10	9.10	9.30	8.10	6.80	7.90	8.10	8.05
12	7.40	7.10	7.10	8.15	8.10	9.10	9.40	8.20	6.80	7.90	8.10	8.90
13	7.40	7.20	7.30	8.15	8.10	9.10	9.80	8.10	6.80	7.90	8.10	8.90
14	7.45	7.10	7.30	8.10	8.50	9.30	9.10	8.90	6.80	7.88	8.10	8.85
15	7.35	7.30	7.30	8.10	8.50	9.30	9.20	8.00	6.60	7.88	8.20	8.85
16	7.35	7.20	7.30	8.10	8.50	9.40	9.60	8.20	6.60	7.90	8.80	8.85
17	7.35	7.20	7.30	8.10	8.10	9.30	9.60	8.70	6.60	7.70	8.80	8.85
18	7.35	7.20	7.30	8.10	8.10	9.30	9.70	8.00	6.60	7.70	8.80	8.85
19	7.35	7.20	7.30	8.10	8.10	9.40	9.70	8.00	6.60	8.70	8.80	8.80
20	7.35	7.10	7.30	8.10	8.20	9.60	9.10	8.00	6.60	8.70	8.80	8.80
21	7.35	7.10	7.30	7.10	8.30	9.80	9.10	8.80	6.60	8.60	8.80	8.80
22	7.35	7.10	7.30	7.00	8.55	9.60	9.00	8.90	6.60	8.60	8.80	8.80
23	7.30	7.10	7.30	7.10	8.60	9.80	9.00	8.00	6.60	8.50	8.80	8.80
24	7.30	7.10	7.30	7.00	8.65	9.60	9.00	8.00	6.60	8.60	8.80	8.80
25	7.30	7.10	7.30	7.10	8.00	9.40	9.00	8.80	6.60	8.30	8.80	8.70
26	7.30	7.00	7.30	7.70	8.90	9.90	9.10	8.90	6.40	8.30	8.70	8.70
27	7.30	7.22	7.30	7.40	8.65	9.40	9.00	8.60	7.40	8.20	8.60	8.70
28	7.30	7.10	7.30	7.10	8.90	9.30	9.00	8.90	7.60	8.20	-	8.70
29	7.30	7.10	7.30	7.80	8.60	9.30	9.90	8.90	7.40	8.20		8.85
30	7.20	7.10	7.30	7.70	8.60	9.30	9.90	-	7.40	8.15		8.85
31	-	7.20	-	7.80	8.95	-	9.00		7.40	8.15		8.85

TABLE A9 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1996/97**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	7.85	7.20	8.75	8.90	8.80	8.00	6.30	6.00	5.90	5.40	4.15	7.05
2	7.85	7.20	8.75	8.90	8.00	8.00	6.30	6.00	5.90	5.40	4.15	7.05
3	7.85	7.10	8.75	8.90	8.10	8.50	6.30	5.90	5.80	5.20	4.20	7.05
4	7.85	7.20	8.75	8.90	8.40	9.00	6.30	5.90	5.90	5.20	4.20	7.90
5	7.80	7.40	8.80	8.95	8.10	9.00	6.30	5.90	5.90	5.20	4.20	7.05
6	7.80	7.30	9.80	8.95	8.10	9.00	6.40	5.90	5.90	5.15	4.20	7.90
7	7.80	8.20	9.80	8.95	8.10	9.20	6.20	6.00	5.80	5.10	4.20	7.90
8	7.85	8.20	9.75	8.90	8.80	9.10	5.90	6.00	5.80	5.00	4.20	7.90
9	7.85	8.20	9.75	8.86	8.00	9.20	6.00	6.00	5.80	4.90	4.20	7.90
10	7.90	8.10	9.75	8.85	8.00	9.20	6.00	6.00	5.80	4.90	4.15	7.05
11	7.90	8.20	9.80	8.84	8.10	9.20	6.30	6.10	5.80	4.90	4.10	7.05
12	7.90	8.10	8.80	8.80	8.10	9.20	6.40	7.20	5.80	4.90	4.10	7.90
13	7.90	8.30	8.80	8.80	8.10	9.20	6.80	7.10	5.80	4.90	4.10	7.90
14	7.90	8.30	8.80	8.85	8.50	9.20	6.10	6.90	5.80	4.88	4.10	7.85
15	7.90	830	8.10	8.90	8.50	9.20	6.20	6.00	5.60	4.88	4.10	7.85
16	7.90	8.20	8.10	8.10	8.50	9.40	6.60	6.20	5.60	4.90	4.20	7.85
17	7.90	8.20	8.20	8.10	8.10	9.30	6.60	6.70	5.60	4.70	4.10	7.85
18	7.40	8.20	8.10	8.10	8.10	9.30	6.70	6.00	5.60	4.70	4.10	7.85
19	7.40	8.20	8.20	8.10	8.10	8.40	6.70	6.00	5.60	4.70	4.10	7.80
20	7.50	8.10	8.10	8.10	8.20	8.60	6.10	6.00	5.60	4.70	4.10	7.80
21	7.40	8.10	8.00	8.10	8.30	8.80	6.10	5.80	5.60	4.60	4.30	7.80
22	7.30	8.40	8.90	8.00	8.55	8.60	6.00	6.90	5.60	4.60	4.10	7.80
23	7.30	8.40	8.10	8.10	8.60	8.80	6.00	6.00	5.60	4.50	4.10	7.80
24	7.30	8.70	8.70	8.00	8.65	8.60	6.00	6.00	5.60	4.60	4.00	7.80
25	7.30	8.70	8.00	8.10	8.00	8.40	6.00	5.80	5.60	4.30	4.00	7.70
26	7.30	8.70	8.90	8.20	8.90	8.90	6.10	6.90	5.40	4.30	4.10	7.70
27	7.30	8.70	8.00	8.40	8.65	8.40	6.00	6.00	5.40	4.20	4.10	7.70
28	7.30	8.70	8.90	8.30	8.90	8.30	6.00	6.00	5.60	4.20	4.00	7.70
29	7.30	8.70	8.90	8.90	8.60	8.30	5.90	5.90	5.40	4.20	-	7.70
30	7.20	8.75	8.90	8.90	8.60	8.30	5.90	5.90	5.40	4.15		7.70
31	-		-	8.80	8.95	-	6.00	-	5.40	4.15		7.70

TABLE A10 WATER DISCHARGE (m^3/s)

River system: Anambra **Name of Stream: Amanyi**

Catchment Area 100Sqkm **Hydrological Year 1997/98**

DATE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MA
1	7.70	7.20	8.20	7.90	8.80	9.10	6.20	9.00	9.90	5.40	8.15	7.05
2	8.70	7.20	8.20	7.90	8.00	9.00	6.30	9.00	9.90	5.40	8.15	4.05
3	8.70	7.10	8.10	7.90	8.10	9.20	6.20	9.90	9.80	5.20	8.20	7.05
4	8.70	7.20	8.30	7.90	8.40	9.00	7.20	9.90	9.90	5.20	8.20	7.90
5	8.70	7.40	850	7.90	8.10	9.00	7.20	9.90	9.90	5.20	8.20	7.05
6	8.70	7.30	8.70	8.90	8.10	9.00	7.40	9.90	9.90	5.15	8.20	7.90
7	8.70	7.20	8.70	8.90	8.10	9.20	7.00	9.00	8.80	5.10	8.20	7.90
8	8.70	7.20	8.20	8.60	8.80	9.10	7.90	9.00	8.80	5.00	8.20	7.90
9	8.60	7.20	8.20	8.90	8.00	9.70	7.70	9.00	8.80	4.90	8.20	7.90
10	6.60	7.10	8.10	8.90	8.00	950	8.70	9.00	8.80	4.90	8.15	7.05
11	6.50	7.20	8.30	8.90	8.10	9.10	8.30	9.10	8.80	4.90	8.10	7.05
12	6.50	7.10	8.50	8.90	8.10	9.50	8.40	9.20	8.80	4.90	8.10	7.90
13	6.50	7.20	8.80	8.90	7.10	9.50	8.80	9.10	8.80	4.90	7.10	7.90
14	8.50	7.10	8.80	8.90	7.50	9.50	9.10	9.90	8.80	4.88	7.10	7.85
15	8.50	7.30	8.10	8.10	7.50	9.50	6.20	9.00	7.60	4.88	7.10	7.85
16	8.60	8.20	7.10	8.10	7.50	9.00	9.60	9.20	7.60	4.90	7.20	7.85
17	8.50	8.20	7.20	8.10	7.10	8.00	9.60	9.70	7.60	4.70	7.10	6.85
18	8.40	8.20	7.10	8.10	7.10	8.60	9.70	9.00	7.60	4.70	7.10	6.85
19	8.40	8.20	7.20	8.10	7.10	8.60	9.70	9.00	7.60	4.70	7.10	6.80
20	8.50	8.10	7.10	8.10	7.20	5.60	9.10	9.00	7.60	4.70	7.10	6.80
21	8.40	8.10	7.00	8.10	7.30	5.80	9.10	9.80	7.60	4.60	7.30	6.80
22	8.80	8.10	7.90	8.00	7.55	5.60	9.00	9.90	7.60	4.60	7.10	6.80
23	8.30	8.10	7.10	8.10	8.60	5.80	9.00	9.00	8.60	4.50	7.10	6.80
24	8.30	8.10	7.70	8.00	8.65	5.60	9.00	9.00	8.60	4.60	7.00	6.80
25	8.30	8.10	7.00	8.10	8.00	5.40	9.00	9.80	8.60	4.30	7.00	6.70
26	7.30	8.00	7.90	8.70	8.90	5.90	9.10	9.90	8.40	4.30	7.10	6.70
27	7.30	8.22	7.00	8.40	8.95	6.40	9.00	9.00	8.40	4.20	7.10	7.70
28	7.30	8.10	7.90	8.10	8.90	6.30	9.00	9.00	8.60	4.20	7.00	7.70
29	7.30	8.10	7.90	8.80	8.90	6.30	9.90	9.90	8.40	4.20	-	7.70
30	7.20	8.10	7.90	8.70	8.90	6.30	9.90	9.90	8.40	4.15		7.70
31	-	8.20	-	8.80	8.95	-	9.00	-	8.40	4.15		7.70