WATER RESOURCES POTENTIALS OF ALA RIVER FOR POSSIBLE WATER SUPPLY TO OBA – ILE HOUSING ESTATE, AKURE.

BY

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CERTIFICATION

This is to certify that this research was carried out by Omoya, David Folorunso (CVE/98/1875) and submitted through the Department of Civil Engineering, the Postgraduate school, Federal University of Technology, Akure, in partial fulfillment for the award of the degree of Masters of Engineering (M.Eng.) in Civil Engineering.

13 Nov. 2006

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Supervisor	*Date*

DEDICATION

This research effort is dedicated to my great grand father, Chief Ologun Ala (late), my late father, Chief Joseph Omoyajowo, my nuclear family, and the glory of God the Omniscient.

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The contributions of my supervisor, Dr. (Engr.) A.M. Oguntuase MNSE to this project is hereby acknowledged and deeply appreciated. He haven been in the research world for such a long time brought to bear his wealth of experience in giving the proper orientation and guidance for the overall success of this programme.

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ABSTRACT

A lot of the water and other environmental resources of many communities in Nigeria are not properly utilized while such communities lack essential infrastructures which could have been developed from those resources through proper harnessing and prudent management by the Government and the respective communities. Oba – Ile Housing Estate, Akure is a typical growing modern urban community with a critical water supply problem despite the presence of Ala River which passes through the Estate.

This project examines the water resources potentials of Oba – Ile Housing Estate and the use to which River Ala; the main water resource may be put for the improvement of the community.

The Ala River discharges was monitored for a period of two years with a locally fabricated staff gauge at a suitable gauging station which is by the bridge linking the southern and the northern parts of the estate. Suitable floats were timed through known displacements to determine the velocity of flows across predetermined segmental areas within the channel for the accurate determination of discharges across the channel.

The mathematical model given by the time series $y = 37.88\left(Sin\left(\frac{2\pi(2n+t+7)}{12}\right)+1\right)$ was developed for the mean staff gauges while the rating equation $Q = 5.307(h-0.2378)^{1.0216}$ was developed for the river discharges at the gauging site. The coefficient of correlation of the rating equation with the observed data was +1.00.

The rating equation was used for the evaluation of the raw water yield of Ala River which was discovered to be capable of serving Oba-Ile Estate, Oba-Ile Town, Eleyowo and part of Akure if properly harnessed. From the result of the studies, a simple water supply scheme and a water reticulation system were designed for Oba-Ile Estate.

The Ala stream corridor was discovered to present a veritable space for fish farming, coconut and plantain plantations, dry season farming, recreation and tourism development.

CHAPTER ONE

INTRODUCTION

1.1 General Background

The Oba-Ile Housing Estate is a typical growing modern urban community. Established in 1979 by the Ondo State Government, it has a total of 1,205 plots, 90% of which have been developed.

The community suffers untold hardship for potable water supply particularly during the dry season when most shallow wells have dried up. The situation is unwholesome and pathetic particularly when viewed against the background that Ala River, the main water resource within the estate has not been developed for the use of the community.

The reticulation network of the first phase of the estate (233 plots) was constructed in the early 80's but the oral evidence from residents shows that over 40% of it was damaged during the initial pressure testing and that the project was subsequently abandoned.

Successive Governments have not really improved the situation as only 5% of the estate has been connected to public water supply ever since. In fact, only two taps receive potable water within the entire estate presently and only at infrequent intervals.

Water is life. It is the most important requirement for human survival next only to air. It is also critical for the health of humans and ecological systems. It is the focus of this thesis to give a critical appraisal to the effective development of appropriate water supply option for the estate.

1.2 Objectives of the Study

The specific objectives of these studies are to:

- Determine a hydrologic and hydraulic characteristics of River Ala;.
- Determine the water quality and quantity of River Ala at the desired location.;
- Design a water supply scheme and a reticulation system for the Estate, and
- Propose effective water resources utilization for River Ala within its catchment areas.

1.3 SCOPES OF WORK

River Ala is an ungauged river, despite its passing through a major town, (Akure) and several communities, with bridges and culverts across it. It becomes necessary to embark on installing a gauge at a suitable location on the river within the Oba-Ile estate to collect discharge data.

Thereafter, the project becomes that of planning and design, for effective utilization of the water resources. Water quality samples were collected at three locations in August, considered the worst period due to heavy rain storm and sediment laden river flow.

1.4 RELEVANCE OF THE WORK

The relevance of this work derives from the need for the effective prediction of the Ala River water discharges for both low flows in the dry season where in minimum flows are critical for water supply purposes as well as the peak of the raining season when high flows and discharges with possibilities of flooding becomes critical for all hydraulic structures like dams, spillways and bridges that may be constructed across the river.

There is also the need to explore the beneficial uses of the Ala stream corridor with its invaluable ecosystem .

CHAPTER TWO

LITERATURE REVIEW

2.1 Accessibility to Freshwater

The quality of life as well as life itself depends on adequate supply of freshwater (Fredericks, 1991). Water and sanitation constitute the primary drivers of public health. Once we can secure access to clean water and adequate sanitation facilities for all people irrespective of the difference in their living conditions, a huge battle against all kinds of diseases will be won (Lee, 2004). The need for accessibility of all to clean potable water for domestic and other uses can therefore not be overemphasized.

Globally, over 1.2 billion people are at risk because they lack access to safe freshwater (Khan, 1997; UN, 1997). In the year 2000, 2.4 billion people lacked access to improved sanitation with 81% of these in rural areas while 86% of the 1.1 billion people that lacked access to improved water sources in the same period were rural dwellers (WHO, 2004). It is also reported that only 61% of people in developing countries have access to adequate water supply while 36% had access to sanitation facilities (WHO, 1998). Within Oba-Ile Housing Estate, the sources of raw water are ground water, rainwater and the Ala River water. In agreement with the position of Obatayo, 2002, the preponderance of these sources not withstanding, drinking water is still another scarce commodity.

In Nigeria, as much as 59% of the households in the rural areas depend on streams and 30.4% on private wells. In addition, 2% draw their water from boreholes. The streams and private wells particularly stand the danger of contamination especially during the raining season and this exposes a large proportion of the rural population to diseases which quite often reduce their productivity (The Federal Office of Statistics, 1987).

The situation of Oba-Ile Housing Estate is even more severe as over 95% of the Residents rely on untreated well water and rainwater, 4.5% have access to boreholes while less than 0.5% have access to treated water. This calls for urgent actions on the issue of water supply to the whole estate.

2.2 Water Supply, Health and Productivity

According to the 1948 constitution of WHO, health is defined as a state of complete well being and not merely absence of physical, mental and social diseases (WHO, 2004). The economic potency of any country is a direct derivative of the health status of the populace. The economies of most advanced countries therefore take seriously the impact of preventive measures on the Gross National Product (GNP) and spend a significant proportion of it in providing clean safe drinking water (Egbo, 2002).

Cholera, typhoid fever, bacillary dysentery, grandasis, dracunculiasis, poliomyelitis and hepatitis, are major water borne diseases. Besides, mineral contaminants such as lead, arsenic, cadmium, etc. could be carcinogenic and/or mutuatory thus permanently damaging human body and ultimately causing untimely death (Idika, 2001).

Diarrhea, a major water borne disease is responsible for 4 billion cases globally every year causing 3-4million deaths especially in children and infants (Olshanky et al; 1997, USAID, 1990). WHO, 2004, quoting the UN Secretary General, Kofi Annan, stated that we shall not finally defeat AIDS, tuberculosis, malaria or any other infectious diseases that plague the developing world until we have also won the battle for safe drinking water, sanitation and basic health care. A study conducted in Karachi, Pakistan, revealed that the poor living in areas without sanitation or hygienic education spent six times more on medical care than people living in areas with proper sanitation and hygiene (Khan, 1997).

The UNICEF collaboration with the Federal Government on the Assisted Water and Sanitation project between 1991 and 1995 made a landmark in the eradication of guinea worms in many rural communities in several states of the Federation. It certainly eradicated the inception of cholera outbreak (Njoku, 2002).

The West African Technical Review, 1982 reports of a Nigerian town where 60% of the 30,000 strong population was suffering from guinea worm, installed a water pipe system in an attempt to combat the disease. The result was better than anyone had dared hoped as guinea worm, a disease transmitted by drinking infested water was completely wiped out.

In Nigeria, the increased accessibility to potable water contributed to decreasing the incidence of water borne diseases. For example, guinea worm has virtually disappeared due to improved supply of potable water and hygienic education (Idika, 2003).

While the cost of building potable water systems and sanitation facilities is high, the cost of not doing so can become staggering (Idika, 2003). It is therefore expedient that the issue of potable water supply should be taken seriously so that potable water that is sparkling may be produced and distributed to the people. The overall effect would be better health for all, reduction in health care delivery costs, improved living standards as well as increased productivity by the residents.

2.3 Sources of Water

Providing potable water supply for a community involves tapping the most suitable source of water, ensuring that it is safe for domestic consumption and supplying it in adequate quantities. The viable water supply option must be carefully examined as to its completeness, effectiveness, efficiency and acceptability (USADIBR, 1987).

The main sources of water that may be developed for Oba-Ile Housing Estate are rainwater, ground water, and surface water. Rainwater is absent during the dry season necessitating the need for expensive storage facilities for reserve against the dry season. Groundwater in the basement complex formation is virtually absent and when it is available it occurs in very small yield often rather difficult to locate. The feasible option for adequate water supply in such environ is through surface water abstraction whenever such water is present in sufficient quantity and treatable quality.

2.4 Water Quality Requirement

Schalekemp (1982) observed that the problems in the world with regards to water supply are that either there is too little of it available or if there is sufficient it is polluted. Adequate water supply is one that provides safe water in quantities sufficient for drinking and for culinary, domestic and other household purposes so as to make possible the personal hygiene of members of the household. A sufficient quantity should be available on a reliable, year round basis near to or within the household where the water is to be used. Unfortunately however, clean water only exists briefly in nature and is immediately polluted by prevailing environmental factors and human activities. Even rain coming from the sky picks up contaminants such as carbon dioxide and particulate matter. The available fresh waters must therefore be treated to achieve desirable qualities. The World Health Organization has given the minimum water standards for potable water supply as given in Table 2.1 (Obatayo, 2002).

2.5 Hydrology

Water is one of the earth's greatest resources. Globally, the total quantity available is essentially constant. Driven by energy from the sun, water constantly circulates from rivers, lakes, streams (through evaporation) by plants (through transpiration) to the atmosphere and back to the earth (through precipitation). Salts and other impurities that may be picked either naturally or as a result of human use are dropped in the process making it possible to use water indefinitely (Frederick and Sedjo, 1991). However, engineering hydrology is mainly concerned with the magnitude and risk of drought. Familiarity with low flows needs tempering with healthy respect for floods and the potential they have for damage (Twort et al, 1986); That is, one should be equally concerned with the low flow from droughts and high flows of flood during heavy rains. Almost two billion people were affected by natural disasters in the last decade of the 20th century, 86% of them by floods and droughts (WHO, 2004). There is thus the need to take the issue of hydrology very seriously in the new world order.

TABLE 2.1: POTABLE WATER W.H.O. STANDARD

(As reported by Obatayo, 2002)

SUBSTANCE	MAX. ACCEPTANCE	MAX. ALLOWANCE
Total Solids	500mg/l	1500mg/l
Colour	6 Units	50 Units
Turbidity	<1 Unit	25 Units
Taste	Unobjectionable	-
Odour	Unobjectionable	-
Iron (Fe)	0.3mg/l	1.0mg/l
Manganese (Mn)	0.1mg/l	0.5mg/l
Copper (Cu)	1.0mg/l	1.5mg/l
Zinc (Zn)	5.0mg/l	15mg/l
Calcium (Ca)	75mg/l	200mg/l
Magnesium (Mg)	50mg/l	150mg/l
Sulphate (SO4)	200mg/l	400mg/l
Chloride (Cl)	200mg/l	250mg/l
PH Range	7.0	6.5 to 9.2
Magnesium + Sodium Sulphate	500mg/l	1000mg/l
Phenolic Substance (as Pheno)	0.001mg/l	0.02mg/l
Nitrates	45mg/l	-
Carbon Chloroform Extract CCE (Organic Pollutants)	0.2mg/l	0.5mg/l
Alkyl Benzyl Sulphonents ABS.	0.5mg/l	1.0mg/l
Fluorides	1.0mg/l	1.5mg/l

2.6 Hydraulics

The channel characteristics controlling the channel conveyance of a river includes the cross sectional area, shape of the stream channel, the sinuosity of the stream course, the expansion and restrictions of the channel, the stability and roughness of the stream bed and banks, and the vegetable cover (Herschy, 1995).

The pressure on the surface of a flowing stream in an open channel is atmospheric and the motivating force establishing flow is predominantly the gravity force component acting parallel with the bed slope but net pressure forces may also be present (Featherstone and Nalluri, 1985).

Over long reaches of river channels, the stream flow at various points along the stream course may be unsteady due to changes in inflow flood for example. The velocity at a particular point will not be constant under such condition but will vary with time. The flow in any stream is rarely uniform over long reaches due to the variation of the channel characteristics along the direction of flow.

Flows in the Ala river channel appear tranquil during the dry season when flow discharges are small. The flows become highly turbulent during the raining season particularly during floods.

2.7 Flow Measurements

With the present worldwide need to better manage our precious water resources, an accurate assessment of the flows of the World rivers is crucial (Herschy, 1995).

It is highly desirable to obtain the right quality of data at the right place and the right time. These data are useful for water management covering water supply, pollution control, irrigation, flood control, energy generation and industrial water use.

The methods of flow measurements are velocity area methods using current meters and floats; weirs and flumes, ultrasonic and electromagnetic methods, and dilution/dye tracing technique (Herschy, 1995, Linsley, et, al, 1982).

2.8 Water Demand

Water demand is the expected daily rate of water use in the community. The determination of the per capita daily demands involves estimating the population to be served and their per capita consumption together with an analysis of the factors that may operate to affect consumption (Steel and Mcghee, 1979).

The World Health Organisation (WHO) has stipulated a per capital demand of 120 lcd for rural areas. In Nigeria, this requirement is far from being met and in urban areas, it has been shown to be lesser than WHO's rural area stipulation. For instance, Areola and Akintola, 1980 concluded that the basic per capita consumption for planned residential area of Ibadan was 89 litres per capita per day.

2.9 Stream Corridor Development

There is more to a stream corridor than a meandering water. A stream corridor or stream valley is a complex and valuable ecosystem which includes the land, plants, animals, and network of streams within it (FISRWG, 1998). Due recognition of the value of Ala stream corridor will give an insight into what has been lost through uninformed and misguided action on Ala stream corridor and the watershed that nourishes it.

2.10 Water Reticulation System

The oldest systematic method for solving the problem of steady flows in pipes is the Hardy Cross method which is itself an early adaptation of the method of moment distribution from structural Engineering in 1936. The method is well suited to hand computations. However, when large network of pipes are involved, computer solution offers the best option (Larock, et, al, 2000).

Usually, the systems of pipes are adapted by skeletonisation to reduce the large network of pipes to manageable sizes. Engineering experience based on sound judgement is invaluable in defining the most appropriate piping system problems to analyse.

The design of complex water supply distribution systems requires adequate conception, planning and design. This can be achieved by the use of network hydraulic modelling software that facilitates the development of functional, safe and economic pipe network (Ekenta, 2002).

Improper design of a water reticulation system may bring about insufficient pressure in the distribution system at some points with the possibility of the contamination and economic loss due to pipe bursts resulting from excessive pressures at other points. Several pipe bursts will impose serious interruptions to water supply and must therefore be prevented.

2.11 Earth Dam and Other Water Retaining Structures

When preliminary studies have confirmed the choice of a flowing river as the most feasible option of water supply to a design population but the quantity of water required cannot be supplied by the flowing river throughout the year, then there is the need for a barrier like a dam or a dyke across the river to impound enough water for the period in which the flows would be unable to sustain demand.

For small water supply schemes, where cost consideration plays an important role, a small earth dam may be adequate. For bigger schemes and in particular depending on cost, geological and geophysical considerations, other types of dams like concrete gravity dams, and cantilevered reinforced concrete dams may be more suitable.

The issue of water retaining structures should be taken seriously because of the propensity they have for damage at failure. Zoppou and Roberts, 1999 reported that over 200 failures of dams greater than 15metres high were recorded in the last century while Billions of dollars damages and a loss of more than 8000 lives were recorded. An earth dam must thus be safe and stable during all phases of the construction and the operation of the reservoir.

The embankment, foundation, abutments and reservoir rim must be stable and must not develop unacceptable deformation under all loading conditions. Seepage must be controlled to avoid internal erosion while overtopping of the embankment during floods must be prevented by the provision of adequate spillway capacity and other outlets.

The best and the most experienced effort is required at the planning stage because a well conceived project can easily be detailed whereas a poorly conceived project often requires herculian effort to consummate. Early design conception must be based on the most realistic rather than the most optimistic or pessimistic evaluation of site conditions and parameters.

The disciplines of hydrology, geology, cartography, geophysics, civil engineering and other technical disciplines have major roles to play in the process of planning, development, and construction of a dam. Invariably, electrical and mechanical Engineers and other technical personnel under them would be required if the dam would be integrated to a water supply or an irrigation system.

Normally, an earth dam is usually constructed of an inner impervious clay core properly compacted at the optimum water content. Compaction in this regard is the act of artificially densifying the soil by pressing soil particles together into close state of contract resulting in the expulsion of air and/or water from the soil mass thereby producing an increased strength of the earth structure (Singh, 1982). The outer shell which is the embankment is constructed of more stable soils that form the pervious layer. The shell must be properly compacted to form adequate protection for the inner clay core.

The upstream may be protected against wave action at the upstream by laying dumped rip-rap at the upstream to specifications. The down stream embankment may be protected against erosion by grassing.

2.12 Spillway and Bottom Outlet

Spillway is the component of the dam where excess water in the allotted storage is released via the spill tail channel to the river channel.

Spillways may be gated or ungated depending on design considerations and cost. Maintenance of gated spillways is definitely higher. The spillway discharges must be dissipated in the stilling basins constructed of concrete or dumped rip-raps in a safe manner that will avoid scouring of the stilling basin bottom.

The ogee (over flow) spillways usually constructed of concrete has a control weir that is ogee shaped. The upper curve of the ogee spillways produces a near maximum efficiency as the water glides over the spill weir without allowing the access of air to the underside of the flowing sheet of water. Sharp crested weirs and other suitable weirs may be used for small earth dams.

The Bottom outlet is used to draw out water from the impounding reservoir to required level to permit maintenance and sometimes to release water to a basin downstream during the dry season. For example, Egbe dam at the Little Osse headworks, Egbe-Ekiti, releases water via a sluice valve to Osse dam, Owo, which is located several kilometres down stream, at the peak of the dry seasons when the latter's raw water requirement cannot be met without augmentation.

CHAPTER THREE

METHODOLOGY

3.1 DATA COLLECTION

The following data were collected:

(i) Survey Data

(ii) Population Data

(iii) Water use Data

(iv) Ala River Gauge Data

(v) Water Quality Data

The methodologies employed in the collection of the relevant data are enumerated here under. However, the analyses of the collected data as well as the results of the analysis are all indicated in the next chapter.

3.2 Survey Works

Topographical maps of Akure, Akure North East, Akure North West, Akure South East, Akure South West and Akure Street Guide were procured from the Ondo State Ministry of Works, Lands and Housing, Akure, while the site layout plans of phases I and II of Oba-Ile Housing Estate were obtained from the Ondo State Property Development Corporation (formerly Ondo State Housing Corporation). The maps and site plans were used to plan the survey works covering the entire estate.

The reconnaissance survey of the estate was undertaken to ascertain the salient features and complete the planning of the survey programmes. The Ala river corridor was studied in detail to determine the beneficial purposes to which it may be put. The theodolite traversing and levelling survey of the entire estate were carried out to determine the relative positions and elevations of the various parts for use in the design of a water supply scheme and the water reticulation system. The levelling survey of the proposed impounding lake area was carried out to evaluate the practicable impoundment that may be achieved. The levelling survey of the cross section of the stream channel at the gauging station was also carried out for the determination of the overall cross sectional area as well as the area of the component segments.

3.3 Questionnaire

Questionnaires on water use, population, and sanitation were prepared and distributed to fifty (50) Residents of Oba-Ile Estate ensuring a good spread. Forty (40) completed questionnaires were returned. These were collated and analysed. A sample of the questionnaire is shown in Appendix I.

3.4 Ala River Gauging

Ala River being the greatest water resource in the estate was studied in detail. The flow pattern was monitored for 2 years (November, 2001 to November, 2003) with a locally fabricated staff gauge. Constructed of 2mm thick flat steel sheets, the staff gauge was 75mm by 50mm in cross section and 4.0metres long. It was painted and calibrated for installation by the bridge across Ala River which links the northern and the southern parts of the estate. The staff gauge was properly positioned on rock to prevent dislocation by water current of the flowing river and was chained to the 16mm plain steel reinforcement projecting from the bridge deck to avoid disturbance by pedestrians.

Reading of the staff gauge was taken at 600Hours and 1800Hours everyday for the two years duration of the studies. Functional flow meters could not be obtained for the monitoring of the flows. However, velocities of surface floats at the respective segments across the cross section were determined by timing the floats though known displacements. Plate 3.1 shows the picture of the installed staff gauge at station 1 which is at the bridge linking the Northern and Southern parts of the estate.

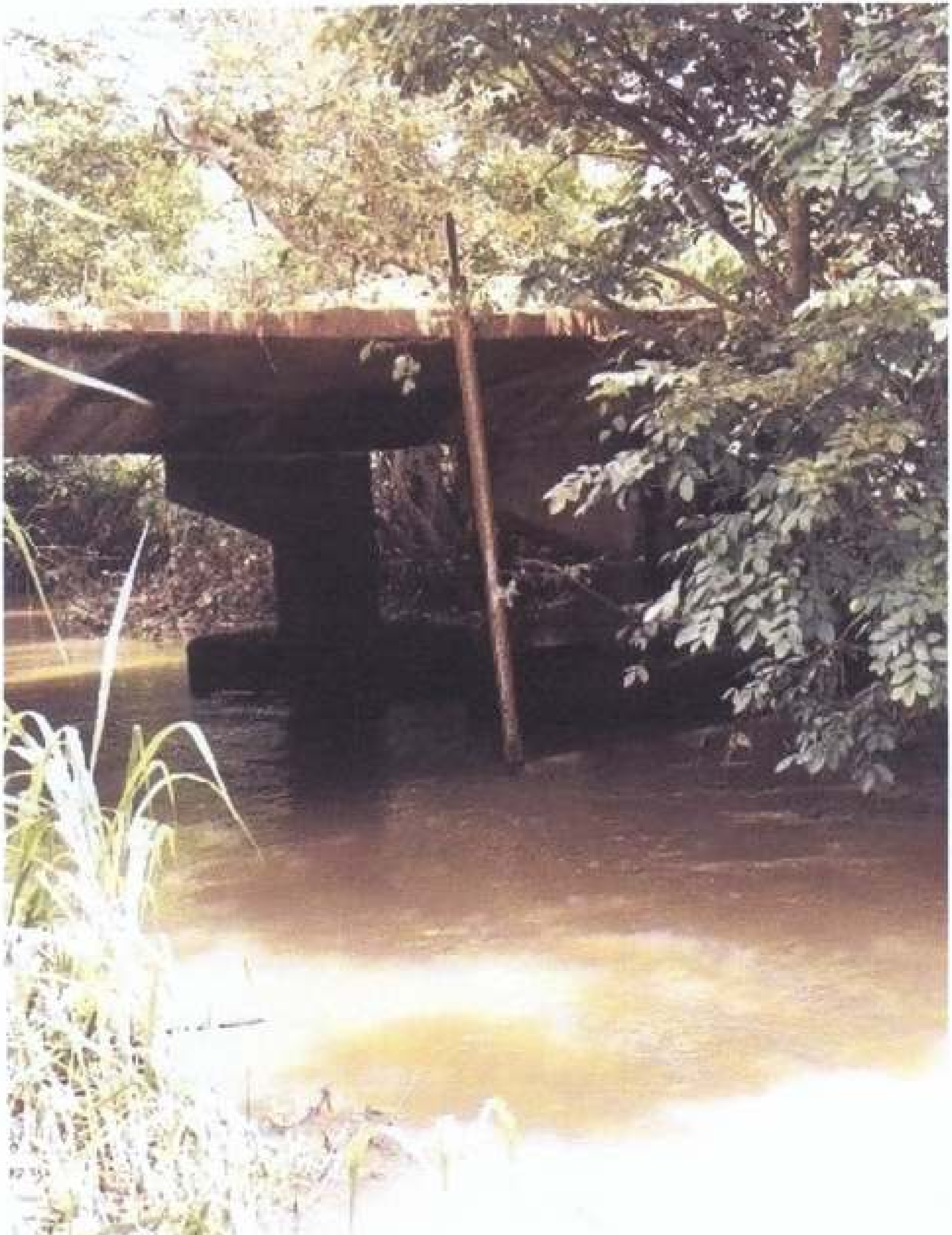

Plate 3.1: Staff Gauge at the Gauging Station

3.5 Water Quality Tests

Raw water samples were taken during normal flows from three locations namely the main outpour of Ala River near Fiwasaye Secondary School, Akure; the gauging station, as well as the proposed impoundment location. The samples were taken between 9.00 Hours to 10.00 Hours on the 11th of August, 2003. The samples were collected during normal flows to avoid excessive sediment loading that occur during floods. The samples were collected with sterilized bottles earlier supplied for that purpose by the laboratory of the Ondo State Water Cooperation. The samples were clearly marked by letters A, B and C respectively to avoid any mix up.

These were analysed for the chemical and microbiological properties of the river to determine the suitable treatment required for the water to make it potable and useful for the community.

3.6 Reticulation System Design

The rather cumbersome printed copies of the manually draughted layout plans of the estate procured from the Ondo State Property Development Corporation were redraughted on AutoCAD and merged together as a single drawing to facilitate a proper adaptation and application of the layout to computer aided designs (CAD).

The water reticulation system was subsequently designed through a computerized visual basic programme (based on the Hardy Cross method) to guarantee accuracy, precision, and economy. The resulting distribution network was draughted on the AutoCAD.

CHAPTER FOUR

DATA ANALYSIS AND RESULTS

4.1 Topographical and Geological Description of the Catchmen Area

From the topographical map of Akure, Oba-Ile Housing Estate was estimated to fall between latitudes 7^0 14^1 30^{11} N and 7^0 16^1N and longitudes 5^0 15^1 E and 5^0 16^1E and located five (5) kilometres east of Akure, the Ondo State capital. The contours on this map revealed a vertical interval of 100ft (30metres) and the estate was estimated to lie between elevation 1000ft (300m) O.D. and 1,100ft (330m) O.D. The tops of some of the inselbergs however were on elevations of up to 1300ft (390m) O.D.

The features on the topographical map of Akure were confirmed on the topographical maps of Akure North West, Akure North East, Akure South West and Akure South East. However, the contours on the latter maps were drawn on 50ft (15m) vertical intervals and a scale of 1:150,000 as against the Akure map which was on a scale of 1:250,000.

The reconnaissance survey of the estate confirmed its relative flatness and the prevalence of igneous rock outcrops at high reliefs at the northwestern part and southwestern parts of the estate. Two inselbergs were observed to be directly opposite the space between the first and second entrance gates of the estate.

The top of the igneous rock at the northwestern part of the estate present a suitable base for the proposed ground level reservoir because of the high relief which would provide enough hydraulic head for a water reticulation system for the estate. Besides, they could sustain a bearing pressure of up to 10,000kN/m^2 (Tomlinson and Boorman, 1995) thus providing the desired stability for the base of the reservoir.

Ala River that drains the estate has a dendritic pattern, flows essentially eastwards within the estate and discharges into Ogbesse River basin about 40 kilometres south east of the estate.

The watershed of River Ala is Akure. There is no inflow river outside the town. River Ala and other streams such as Elegbin, Omiyeye and Osise that are tributaries to River Ala take their sources within Akure town (Aduwo and Jemilugba, 2000). The topographical maps, revealed the presence of heavy forest along the Ala River through out its course. However, urbanization and human activities had virtually destroyed the forest along the river within Akure town and had significantly depleted it within Oba-Ile Housing Estate.

The features of the upland course of a river generally characterized by steep slopes, high velocity of river flows, water falls, splashing sounds and erosion were not evident along the course of River Ala within the estate due to the flatness of the terrain. However, the features of the middle course were observed. The presence of some ox-bow lakes were noticed while the flooding of the banks were observed in the peak of the raining season along the flat segments of Ala River course within the estate.

Some rock outcrops were observed along the river channel within the estate. These together with the inselbergs dotting the landscape confirm the presence of the basement complex geological formations of the southwestern Nigeria.

Fig. 4.1 shows the layout of the Oba-Ile Housing Estate as reproduced in AutoCAD after merging the two existing plans of phases I and II obtained from the Ondo State Property Development Corporation.

4.2 Questionnaire Analysis

Table 4.1 shows the analysis of the data collected through the forty (40) questionnaires returned by the residents. The mean number of persons per building is 7 with the percentages of men: women: children being 25%: 26%: 49% respectively. 92.5% of the sampled population wanted government to provide potable water for the estate while 77.5% wanted a separate water supply scheme for the estate. The distribution pattern of water closet facilities shows an impressive figure of 3 water closets per building which translates to less than three persons per toilet. It appears that faecal disposal within the estate is effective since all homes sampled had water closet facilities. The locations of the soak away pits relative to the wells may however not be completely satisfactory.

Table 4.1: Analysis of the Forty (40) Completed and Returned Questionnaires on Water Resources Utilization for Oba-Ile Housing Estate, Akure.

Item No.	Description	Options Under Description	Total	Average Per House	Total %
1.	Population	Number of Women	67	1.68	25
		Number of Men	69	1.73	26
		Number of Children	130	3.25	49
		Total	266	6.65	100
2.	Sources of Water	Public Tap	-	-	-
		Borehole Only	4	0.10	10.0
		Borehole + Wellwater + Rainwater	2	0.05	5.0
		Well Water Only	28	0.70	70.0
		Well Water + Rainwater	4	0.10	10.0
		Rainwater Only	2	0.05	5.0
		Total			100
3.	How Frequent is Public tap running	Everyday	-	-	-
		Once a Week	2	0.05	5
		Once a Month	1	0.025	2.5
		Never	33	0.825	82.5
		Unaware	4	0.100	10.0
		Total			100
4.	Type of Toilet	Pit Toilet	-	-	-
		W. C.	120	3.00	100
		Others	-	-	-
		Total			100
5.	Government to provide potable water for the Estate.	Yes	37	0.925	92.5
		No	3	0.075	7.5
		Total			100
6.	Separate Water Supply System For Oba-Ile Housing Estate	Yes	31	0.775	77.5
		No	9	0.225	22.5
		Total			100
7.	Type of Water Supply to be provided by Government.	Borehole	2	0.05	5.0
		Tap Water	36	0.90	90.0
		Well Water	-	-	-
		Any Good Water	2	0.05	5.0
		Total			100
8.	Willing to Get Connected to Public Water Supply	Yes	39	0.925	97.5
		No	1	0.025	2.5
		Total			100

4.3 Water Quality Analysis

The results of the chemical and microbiological tests on the raw water samples at three (3) different locations are shown in Table 4.2. There were no observable variations in the raw water qualities from the 3 samples. Normally, one would have expected improvement due to self purification process over a distance of 5kilometres from Akure town. The fact that there is no appreciable difference points to the fact that some pollution due to the effect of urbanization must be taking place within the estate hence the need for treatment of the water to make it potable.

4.4 Hydrograph Analysis

The mean daily staff gauges for November, 2001 to November, 2003 (25months) are shown in Tables 4.3 to 4.6.

The mean staff reading in the middle third of January, 2002 was practically nil as shown in Table 4.3. This does not support the general held belief that River Ala is a perennial River.

The peak mean daily staff gauge of 302.6cm was observed on the 24th of September, 2002. River Ala overflowed its banks during this period. However, there were no destructive effects on properties located within the estate by reason of the 30metres minimum offset enforced by Government and complied with by developers.

It was observed that the staff gauges were generally low between December, 2001 and February, 2002 as well as December, 2002 to February, 2003. However, because of rainfall on the 17th of February, 2003, a mean daily staff gauge of 70.4cm was recorded on the 18th of February. This quickly reduced to 7.7cm on the third day which was the 20th of February, 2003.

Table 4.7 shows the mean monthly staff gauges for November, 2001 to November , 2003 (25months).

Table 4.2: RAW WATER PHYSICAL AND CHEMICAL ANALYSIS OF RIVER ALA

Laboratory: Ondo State Water Corporation Akure.

Sample: A-Fiwasaye, B-Bridge, C-Proposed impounding Reservoir area

Date Collected: 7Th Aug., 2003.

Collected By: Engr. D. F. Omoya

Time Collected: 9 - 10.00am

Date of Completion of Analysis: 12th Aug., 2003.

NOS	TEST	A	B	C
1.	Appearance	Brownish	Brownish	Brownish
2.	Colour, Hazen	70	74	76
3.	Temperature (^{0}C)	25	25	25
4.	Turbidity, NTU	20	25	27
5.	P^H	6.8	7.0	6.8
6.	Dissolved Solid mg/l	246	242	244
7.	Suspended solids, mg/l	46	44	46
8.	Total solids mg/l	292	286	290
9.	Total Alkalinity mg/l	195	192	174
10.	Total Hardness mg/l $CaCO_3$	154	146	136
11.	Calcium Hardness, mg/l $CaCO_3$	98	95	117
12.	Magnesium Hardness, mg/l $CaCO_3$	56	51	19
13.	Chloride, mg/l C1	95	69	67
14.	Nitrate, mg/l	0.75	0.74	0.74
15.	Total Iron, mg/l Fe	1.50	1.20	1.20
16.	Manganese, mg/l Mn	ND	ND	ND
17.	Sulphate, mg/l SO_4	17.5	17	16.8
18.	Phosphate, mg/PO_4	0.5	0.4	0.45
19.	Fee Carbon Dioxide mg/l CO_2	25	23	24
20.	Dissolve Oxygen mg/l O_2	4.6	4.8	4.6
21.	Bio. Chem. Oxygen Demand (BOD) mg/L O_2	-	-	-
22.	Chemical Oxygen Demand COD mg/l O_2	-	-	-
23.	Silica as S_iO_3, mg/l SO_3	Traces	Traces	Traces
24.	Organic Matter mg/l C	-	-	-

Table 4.3: Staff Gauging of Ala River for November, 2001 to April, 2002 at Station 1

Day	Mean Daily Staff Readings (cm)					
	November 01 (cm)	December 01 (cm)	January 02 (cm)	February 02 (cm)	March 02 (cm)	April 02 (cm)
1.	11.0	6.8	5.0	4.0	4.5	4.0
2.	12.3	8.0	2.0	8.8	5.3	5.0
3.	12.2	7.8	1.5	6.0	64.0	4.3
4.	6.9	7.8	1.0	7.1	24.5	3.5
5.	9.0	7.3	1.0	4.1	10.5	1.5
6.	12.4	7.8	2.5	8.4	29.5	2.5
7.	14.7	6.5	0.5	6.0	43.0	2.5
8.	17.4	10.0	0.0	25.5	17.5	37.5
9.	18.7	6.5	0.0	17.0	9.0	25.0
10.	16.3	6.3	0.5	13.0	23.0	4.0
11.	18.5	5.7	0.0	8.5	13.5	3.0
12.	32.3	5.3	0.0	7.5	24.0	3.5
13.	30.7	4.3	0.0	6.5	13.0	41.5
14.	28.5	4.3	0.0	6.5	13.0	60.0
15.	25.3	5.5	0.0	6.0	11.0	62.5
16.	18.5	5.3	0.0	29.0	26.5	52.5
17.	17.8	4.8	0.0	27.0	37.5	15.0
18.	15.3	4.5	0.0	28.5	6.5	10.0
19.	14.0	4.8	0.0	14.0	41.0	103.0
20.	11.5	4.8	1.0	13.0	64.0	34.5
21.	11.3	6.8	3.4	12.0	24.5	87.5
22.	12.5	8.0	2.5	10.0	10.5	55.2
23.	15.5	7.8	3.1	9.5	6.0	35.4
24.	15.5	7.8	2.5	10.0	6.5	30.8
25.	11.9	7.3	2.0	7.0	26.5	32.6
26.	12.7	7.8	2.0	5.0	25.5	22.6
27.	10.5	6.5	5.5	5.0	12.0	20.0
28.	11.4	10.0	4.0	8.5	10.0	15.0
29.	10.5	9.9	3.5	43.5	10.0	12.0
30.	11.7	8.8	4.5	-	4.5	13.5
31.	-	8.3	4.5	-	5.3	-

Table 4.4: Staff Gauging of Ala River for May, 2002 to October, 2002 at Station 1

Day	Mean Daily Staff Readings (cm)					
	May 02 (cm)	June 02 (cm)	July 02 (cm)	August 02 (cm)	September 02 (cm)	October 02 (cm)
1.	14.1	55.5	14.2	85.3	65.0	25.0
2.	15.1	76.0	80.5	137.9	60.1	20.1
3.	11.0	82.6	113.3	107.7	55.2	12.7
4.	10.0	54.7	60.6	95.2	45.3	47.7
5.	9.6	140.5	92.6	90.1	39.3	50.0
6.	9.1	177.8	48.1	120.2	52.8	18.8
7.	8.5	67.6	37.8	100.1	72.8	187.6
8.	8.1	13.4	18.1	95.2	70.2	180.4
9.	6.6	9.9	10.4	72.9	90.1	90.5
10.	5.3	8.7	62.1	50.1	69.1	75.1
11.	85.3	7.2	12.6	200.5	120.0	95.4
12.	46.6	49.7	82.5	97.6	100.2	140.
13.	40.6	19.6	37.6	185.3	83.0	130.1
14.	79.6	11.6	20.7	117.6	215.0	80.2
15.	38.7	9.4	28.0	150.3	245.2	77.7
16.	29.4	157.5	130.1	195.1	155.3	52.8
17.	19.0	93.0	122.9	102.7	92.7	25.5
18.	16.1	107.7	172.8	75.2	112.7	15.2
19.	9.8	148.3	193.1	85.5	92.7	16.2
20.	14.3	100.2	135.2	74.6	160.2	20.0
21.	20.6	78.6	170.1	82.6	225.2	36.5
22.	32.7	37.1	187.7	110.4	262.6	54.7
23.	37.0	22.9	120.4	115.1	302.5	33.5
24.	15.4	44.1	105.0	62.6	302.6	36.5
25.	7.4	67.7	67.8	30.2	280.0	42.1
26.	7.6	43.0	62.7	130.0	250.1	75.1
27.	5.0	35.1	40.4	90.5	255.2	95.4
28.	4.6	17.2	43.0	117.7	200.4	185.2
29.	5.7	13.1	57.8	101.6	160.2	185.1
30.	4.9	11.7	48.1	75.1	170.3	90.0
31.	5.2	-	49.1	70.4	165.2	51.8

Table 4.5: Staff Gauging of Ala River for November, 2002 to April, 2003 at Station 1

Day	Mean Daily Staff Readings (cm)					
	November 02 (cm)	December 02 (cm)	January 03 (02)	February 03 (cm)	March 03 (cm)	April 03 (cm)
1.	12.8	10.0	10.2	5.2	6.5	4.0
2.	10.3	10.2	15.3	5.1	5.2	3.5
3.	5.3	10.1	20.2	5.0	5.2	4.0
4.	0.5	12.8	20.4	5.2	5.1	2.5
5.	48.0	11.7	15.1	4.5	5.0	2.5
6.	117.6	15.3	12.8	4.0	5.0	4.5
7.	62.7	12.5	10.3	4.5	5.2	4.5
8.	40.3	11.5	15.7	5.1	5.0	3.0
9.	32.5	10.5	12.7	4.7	5.1	5.2
10.	37.7	12.5	10.2	4.5	5.1	4.6
11.	32.8	15.5	10.2	5.1	7.7	4.0
12.	55.2	15.5	10.1	3.0	7.7	3.0
13.	65.1	11.9	10.2	4.6	5.0	7.7
14.	35.2	12.7	10.4	15.2	5.0	5.1
15.	40.0	10.5	20.4	7.6	5.0	5.0
16.	17.9	11.8	15.1	7.7	5.0	5.1
17.	12.8	12.3	10.4	17.8	5.0	12.7
18.	15.0	12.0	7.6	70.4	10.1	12.9
19.	10.7	11.9	13.5	22.6	10.2	7.7
20.	10.5	14.5	10.2	7.7	12.6	25.2
21.	7.6	14.1	7.7	5.0	15.3	22.7
22.	7.9	14.8	7.8	4.5	10.4	12.6
23.	12.7	12.8	7.0	4.5	10.1	7.8
24.	15.1	12.4	7.1	5.2	7.6	15.2
25.	17.7	10.5	7.9	5.1	5.2	15.2
26.	12.7	13.1	7.6	3.5	5.1	7.5
27.	10.0	14.8	7.6	3.5	5.0	3.0
28.	12.7	12.6	7.8	4.5	10.6	7.6
29.	10.1	14.5	7.5	-	10.3	3.0
30.	12.6	13.8	9.2	-	10.3	4.0
31.	10.7	13.0	10.2	-	5.1	3.0

Table 4.6: Staff Gauging of Ala River for May, 2003 to November, 2003 at Station 1

Day	Mean Daily Staff Readings (cm)						
	May 03 (cm)	June 03 (cm)	July 03 (cm)	August 03 (cm)	September 03 (cm)	October 03 (cm)	November 03 (cm)
1.	2.5	7.7	40.4	37.7	15.2	30.0	99.0
2.	2.0	7.9	102.5	42.8	12.7	25.2	57.2
3.	1.0	7.7	37.6	62.9	17.6	27.5	67.7
4.	1.0	6.6	25.2	37.6	17.5	34.0	122.7
5.	4.0	5.1	20.1	20.2	17.7	47.0	83.1
6.	3.0	5.0	22.7	27.6	15.2	51.5	50.5
7.	4.0	5.1	47.9	27.6	15.0	71.5	58.4
8.	3.0	7.2	72.7	17.6	10.4	95.5	67.6
9.	2.5	10.2	42.7	35.2	17.7	162.5	52.7
10.	2.5	5.1	25.2	22.8	26.0	172.5	45.0
11.	10.2	5.2	40.1	10.4	20.2	142.5	50.0
12.	7.6	5.1	25.3	12.6	58.2	148.2	52.7
13.	5.1	5.0	15.1	20.1	57.6	192.5	60.3
14.	5.0	5.0	5.4	15.3	43.7	205.5	73.0
15.	4.5	17.7	35.2	17.7	32.7	90.1	57.6
16.	4.6	12.7	22.7	73.6	31.1	119.5	50.1
17.	5.2	5.1	105.2	60.1	37.0	62.6	45.0
18.	5.1	5.2	82.7	35.2	42.6	67.6	42.9
19.	4.5	5.3	47.7	100.2	53.0	68.0	45.0
20.	110.3	12.8	72.8	51.6	53.2	135.1	40.0
21.	102.6	55.2	130.2	75.0	36.2	78.0	37.0
22.	30.2	40.3	155.2	75.2	40.5	107.5	38.0
23.	10.2	30.3	175.3	52.6	30.2	87.6	40.1
24.	5.2	70.4	75.3	37.7	20.4	70.1	40.0
25.	17.7	72.8	55.1	67.6	32.6	69.0	39.1
26.	7.8	37.9	90.3	50.1	54.0	57.6	39.0
27.	7.8	37.5	112.7	37.7	50.1	58.1	32.9
28.	5.1	20.3	82.6	20.1	48.6	71.5	36.5
29.	5.0	17.9	152.6	22.6	38.5	80.1	22.5
30.	10.3	18.1	65.1	20.2	39.5	65.5	15.0
31.	-	14.9	-	17.7	28.0	-	-

Table 4.7: Mean monthly staff gauges for Ala River at Station 1 for November, 2001 to November, 2003

Months	Mean of the Monthly Staff Gauges (cm)
November – 01	15.54
December – 01	6.84
January – 02	1.69
February – 02	12.3
March – 02	20.06
April – 02	26.66
May – 02	20.07
June – 02	58.68
July – 02	78.20
August – 02	104.02
September – 02	147.43
October – 02	72.47
November – 02	25.54
December – 02	12.63
January – 03	11.21
February – 03	8.75
March – 03	7.11
April – 03	7.35
May – 03	12.97
June – 03	18.11
July – 03	66.10
August – 03	38.85
September – 03	32.66
October – 03	89.78
November – 03	52.01

This shows a value of 6.84cm, 1.69cm and 12.3cm for December, 2001, January, 2002 and February, 2002 respectively. The figures were 12.63cm, 11.21cm, and 8.75cm for December, 2002, January, 2003 and February, 2003 respectively. This indicates a pattern of low flows in December of a particular year through January, to February of the following year.

In the year 2003, low mean monthly values of 7.11cm, 7.35cm and 12.97cm were observed in March, April, and May respectively. This is possibly due to the fact that year 2003 was a relatively dry year giving a peak mean monthly staff reading of 89.78cm in October of that year compared with a peak value of 147.43cm in September, 2002.

Fig. 4.2 shows the graphical representation of the mean monthly staff gauges at the gauging station for the entire gauging period. The resulting hydrograph has been fitted with a sinusoidal curve as shown below:

$$y = 37.88\left(\sin 2\pi \frac{(12n + t + 7)}{12} + 1\right)$$

Where n is the number of years and t is the number of months with January to December taking values of 1 to 12 respectively. The amplitude 37.88 is the mean of the mean monthly staff gauges in the whole 25months gauging period.

The fitted curve can be used for the prediction of staff gauges and discharges in subsequent years in the absence of additional data. Besides, there is a reasonable measure of correlation between the actual curve and the fitted curve as given by a correlation coefficient of +0.74 and a coefficient of determination of 0.55 as obtained using a Microsoft Excel Software Programme. However, the detail of the computation is as given in appendix II.

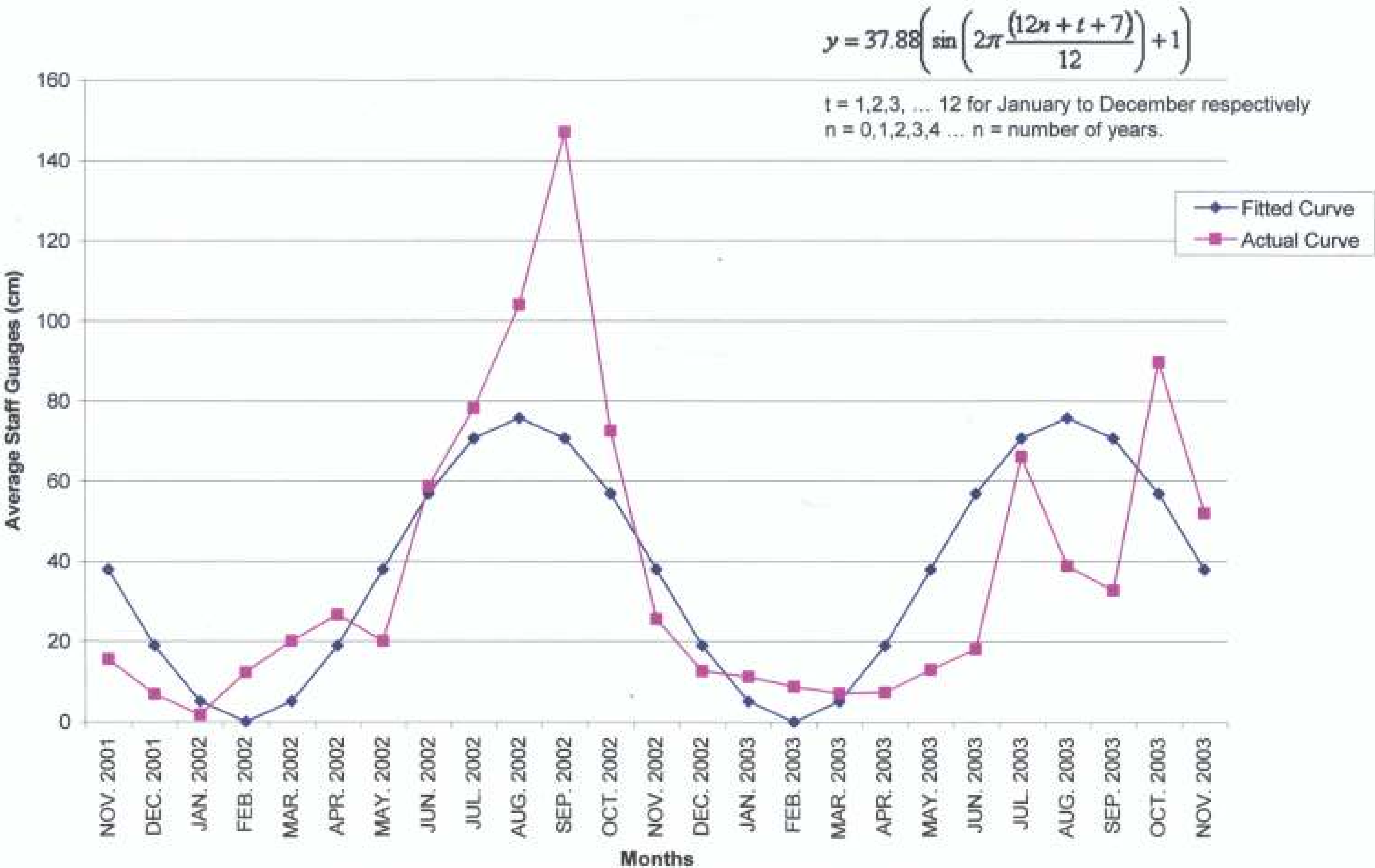

Fig. 4.2: Discharge Time Series for River Ala at Station 1

4.5 Rating Curve

Fig.4.3 shows the layout of the cross section of the stream channel at the gauging station which is by the upstream side of the bridge linking the northern and southern parts of the estate. Table 4.8 shows the computation of the discharges against the staff gauges. In the table, surface velocity was taken as 1.2 x mean velocity as proposed by (Linsley et al, 1982). At any gauging height, Q was obtained by multiplying the area of the respective segments with the associated mean velocities and summing up at that level.

Fig. 4.4 shows the plot of the staff gauges versus the discharges. A smooth curve through the plotted points gives a rating curve for River Ala at the gauging station. The equation of the rating curve is governed by the exponential function $Q = 5.307(h - 0.2378)^{1.0216}$. The development of the equation is shown is appendix III. Table 4.9 shows the comparison of the values of the actual computed discharges and the values obtained from the fitted curve.

The coefficient of correlation between the actual discharges and discharges obtained from the rating curve is +1.00 while the coefficient of determination is 1.00. This shows near perfect fit of the equation to the observed data and guarantees the accuracy of the prediction of discharges at various depths at the gauging station. The coefficient of correlation may be obtained from a simple excel programme but the development is shown in appendix IV.

4.6 Ala Stream Corridor

Fig. 4.1 shows that at least 100 plots are adjacent to River Ala within the Estate. The 30 metres minimum offset given for each plot is suitable for fish pond development. Throughout the Estate, there were only two fish ponds within the premises of an Ijaw man. His premises is always evergreen with many crops like plantain, bananas, oil palm and cocoyam.

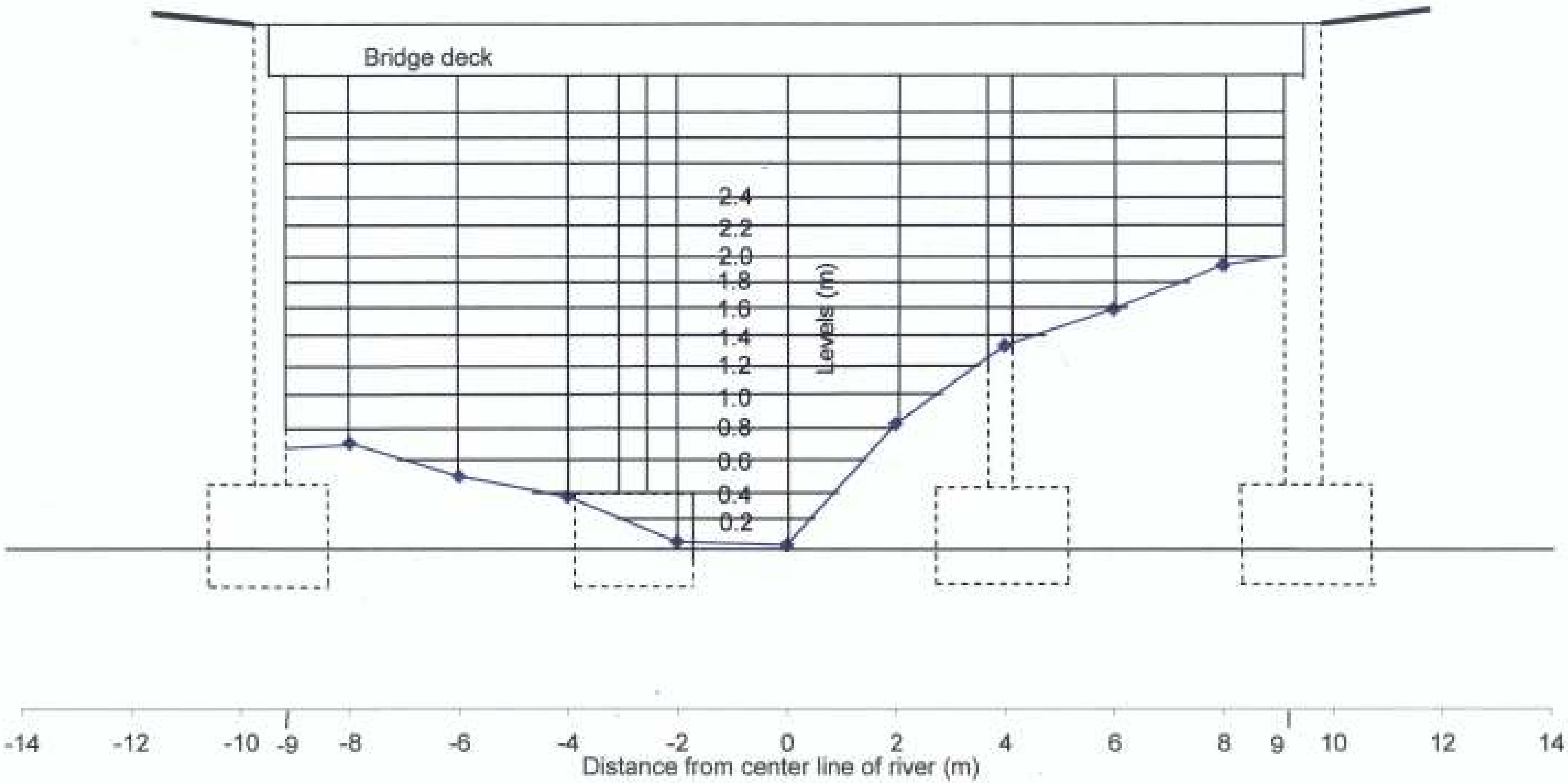

Fig. 4.3: Cross section of Ala River at gauging site for discharge monitoring

Table 4.8: Variation of Discharges Q(m^3/s) with Staff Gauges (m) at Station 1 Along Ala River.

H (m)	L (m)	t (s)	A (m^2)	$V = \frac{L}{1.2t}$	Q = A*V (m^3/s)	Q (m^3/s)
	7.4	11.6	0.230	0.53	0.12	
	7.4	11.4	0.700	0.54	0.38	
	7.4	9.9	0.840	0.62	0.52	
0.4	7.4	10.9	0.252	0.57	0.14	1.17
	7.4	18.1	0.532	0.34	0.18	
	7.4	7.6	0.124	0.81	0.10	
	7.4	6.9	1.100	0.89	0.98	
	7.4	6.9	0.630	0.89	0.56	
0.6	7.4	19.3	0.175	0.32	0.06	1.88
	7.4	12.8	1.012	0.48	0.49	
	7.4	10.9	1.640	0.57	0.93	
	7.4	10.9	1.500	0.57	0.85	
	7.4	13.5	1.030	0.46	0.47	
	7.4	14.2	0.595	0.43	0.26	
0.8	7.4	67.3	0.180	0.09	0.02	3.01
	7.4	43.5	0.380	0.14	0.05	
	7.4	15.7	0.995	0.39	0.39	
	7.4	13.0	1.430	0.47	0.68	
	7.4	11.9	1.900	0.52	0.98	
	7.4	10.9	1.412	0.53	1.15	
	7.4	11.7	1.412	0.53	0.74	
1.0	7.4	∞	0.16	0.00	0.00	4.01
	7.4	∞	0.480	0.00	0.00	
	7.4	12.1	1.810	0.51	0.92	
	7.4	10.9	2.440	0.57	1.38	
	7.4	11.9	2.300	0.52	1.19	
	7.4	12.8	1.830	0.48	0.88	
	7.4	14.0	1.400	0.44	0.62	
1.2	7.4	41.1	0.580	0.15	0.09	5.08
	7.4	16.1	0.780	0.38	0.30	
	7.4	15.1	1.800	0.41	0.74	
	7.4	14.2	2.230	0.43	0.97	
	7.4	12.5	2.700	0.49	1.33	
	7.4	12.5	2.840	0.49	1.4	
	7.4	13.7	2.210	0.45	0.99	
	7.4	15.7	0.880	0.39	0.35	
1.4	7.4	∞	0.110	0.00	0.00	6.08
	7.4	29.4	0.430	0.21	0.09	
	7.4	15.8	1.280	0.39	0.50	
	7.4	14.0	2.610	0.44	1.15	
	7.4	12.3	3.240	0.50	1.62	
	7.4	13.7	3.100	0.45	1.40	
	7.4	14.0	2.630	0.44	1.16	
	7.4	15.0	2.200	0.41	0.90	
1.6	7.4	15.4	0.980	0.40	0.39	7.21

H (m)	**L (m)**	**t (s)**	**A (m^2)**	$V = \frac{L}{1.2t}$	**Q = A*V (m^3/s)**	**Q (m^3/s)**
	7.4	44.0	1.180	0.14	0.17	
	7.4	15.0	2.600	0.41	1.07	
	7.4	14.3	3.000	0.43	1.29	
	7.4	13.4	3.500	0.46	1.61	
	7.4	11.9	3.640	0.52	1.89	
	7.4	13.4	2.710	0.46	1.25	
	7.4	15.4	1.680	0.40	0.67	
	7.4	15.8	0.830	0.39	0.32	
	7.4	18.7	0.300	0.33	0.10	
1.8	7.4	∞	0.100	0.00	0.00	8.37
	7.4	∞	0.730	0.00	0.00	
	7.4	11.0	0.700	0.56	0.39	
	7.4	12.3	1.230	0.50	0.62	
	7.4	12.3	2.080	0.50	1.04	
	7.4	12.1	3.110	0.51	1.58	
	7.4	9.8	1.040	0.63	0.65	
	7.4	12.1	3.900	0.51	1.58	
	7.4	12.8	3.400	0.48	1.64	
	7.4	14.0	3.000	0.44	1.32	
2.0	7.4	34.3	1.380	0.18	0.25	9.49

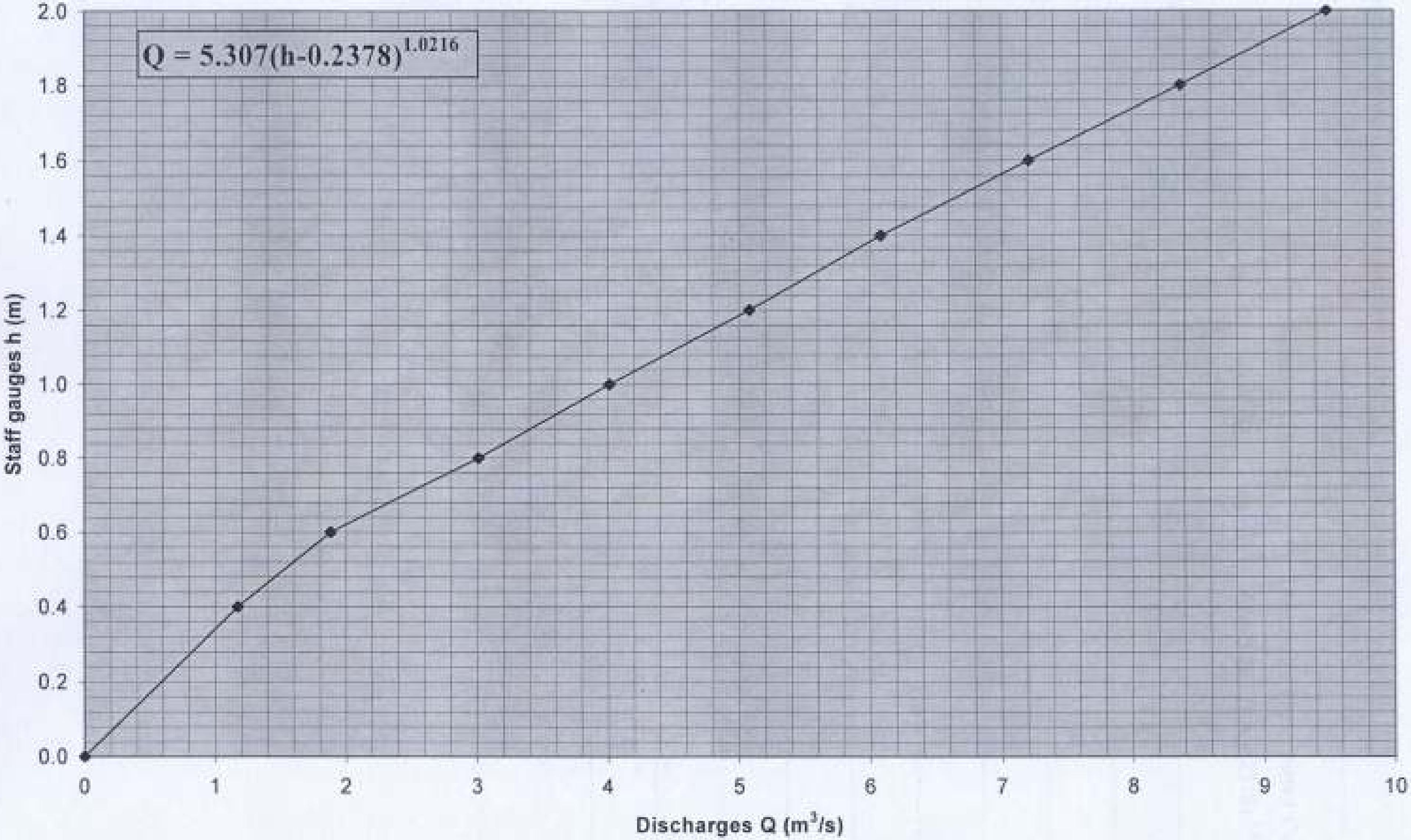

Fig. 4.4: Rating Curve of Ala River at Station 1

Table 4.9: Comparison of the actual computed discharges with discharges from the fitted exponential function at various heights at the gauging station.

Depth (m)	Actual Discharge (m^3/s)	Discharge From Fitted Curve $Q = 5.307\,(h - 0.2378)^{1.0216}$ (m^3/s)
0.4	1.17	0.83
0.6	1.88	1.88
0.8	3.01	2.95
1.0	4.01	4.02
1.2	5.08	5.01
1.4	6.08	6.19
1.6	7.21	7.28
1.8	8.37	8.37
2.0	9.49	9.47

Correlation Coefficient = +0.9993 = +1.00

Coefficient of Determination = 0.9986 = 1.00

According to him, he is going to improve the ponds based on the general encouragement of government though the provision of fingerlings at subsidized rates.

It was discovered that there were no swimming pools, recreational centres, reception, and youth centres throughout the estate. Meanwhile, the Landlords Association was said to be looking at the possibility of having a recreational centre with the support of the government.

The Ala stream corridor provides a lot of opportunities for providing land area for such purposes. The impact of having about 50 fish ponds within the Estate will not only put more animal protein on the tables of residents but will have a better economic, social, recreational and employment opportunities on the residents while providing a greener environment especially in the dry seasons.

CHAPTER FIVE

DESIGN OF WATER SUPPLY SCHEME

5.1 PROCESS DESIGN

5.1.1 Estimate of water demand

The total number of developed plots is 949 plots which is about 90% of the entire plots in the estate. At an average of 7 persons/plot, total present population = 949 x 7 = 6643 persons. At an annual growth rate of 2.5% per annum, the population in the year 2024 shall be

$$p_{2024} = p_i(1+0.025)^{20}$$

$$= 6643(1.025)^{20} = 10{,}886\,persons$$

$$V_{2024} = 10{,}886x0.12 = 1306\,m^3/day$$

$$V_{present} = 6643x0.12 = 797.2\,m^3/day$$

For 8hrs of pumping day at present

$$Q = \frac{797.2}{8} = 99.65\,m^3/hr$$

For 13hours pumping/day in 2024

$$Q = \frac{1306}{13} = 100.4\,m^3/hr$$

Design discharge Q = 100m^3/hr

5.1.2 Sedimentation Tank Design

Total inflow into sedimentation tanks Q = 100m^3/hr.

Using two tanks to allow for alternate cleaning of tanks,

flow though each tank $= \frac{Q}{2} = 50\,m^3/hr$

Assume L:B = 2.0 to 1.0 (Twort, et al, 1985)

Detention time D. T. = 2hrs. (Twort, et al, 1985)

Settling velocity surface loading $V = \frac{Q}{bL}$

Where b = width of sedimentation tank

L = length of sedimentation tank

$V = 1.0 m/hr$ (Twort, et al, 1985)

$$1 = \frac{50}{bl}$$

$$1 = \frac{50}{2.0b^2}$$

$$b = \sqrt{\frac{50}{2}}$$

$= 5.0$m

$l = 2.0b$

$= 2(5.0)$

$= 10.0$m

Horizontal Velocity $V_h = \frac{Q}{bd}$ (Twort et al, 1985)

Where

d = effective depth of sedimentation tank

Q = discharge through sedimentation tank

b = width of sedimentation tank.

V_h is in the range 4-36m/hr (IRC, 1981)

Assume V_h = 5m/hr. for full utilization of tank.

$$5 = \frac{50}{5xd}$$

$d = 2.0m$

Assume a freeboard of 0.3m and a sludge depth of 0.5m

Height of Tank $H = 2.0 + 0.3 + 0.5$

H = 2.8m

5.1.3 Filter Design

For a typical local sand filter medium, sand of effective size 0.95mm and uniformity coefficient of 1.58 could be used (Fakaye and Omoya, 1984).

Where effective size D_{10} is the sieve size through which 10% of the material passed.

And uniformity coefficient $U_c = \frac{D_{60}}{D_{10}}$

Sand Range = 0.5 to 1.5mm (Twort et al, 1985).

And Uc = 1.5 to 2.5 (Twort et al, 1985)

For Rapid rate gravity filter,

surface loading $V = \frac{Q}{A}$

Where Q is total inflow into the filter and A is surface area of filter

Assume two filter units

v = 4m/hr. (Twort et al, 1985)

$A = \frac{50}{4} = 12.5m^2$

$L = B = \sqrt{12.5}$

$L = B = 3.54m$

Water depth above filter = 1.0m

Freeboard = 0.3m

Sand bed = 0.75m

Gravel bed (inclusive of filter under drains)

= 0.45m

Gravel size range = 5 – 60mm. (Henry and Heinke, 1989)

Total depth h = 2.5m. This is okay (Henry and Heinke, 1989)

5.1.4 Backwashing

Upward wash water rates = 0.5 to 6.5mm/s (Twort et al, 1985)

= (1.8 to 23.4m/hr.)

Assuming a rate of 5m/hr.

Area of each filter = $12.5m^2$

$Q = 62.5m^3/hr.$

This Q shall be supplied by the backwashing pump.

5.1.5 Chlorination Chamber

Assuming a chlorine demand of 1.5mg/l and a chlorine residual of 1.0mg/l

This scheme being a small one, chlorine is to be obtained from Hypochlorite (HTH) supplied in drums.

This contains about 60 – 70% available chlorine.

$Q = 100m^3/hr.$

$$\frac{Kg}{Day} = \left\{\begin{array}{c}\text{Litres of water treated}\\ \text{at maximum daily flow rate}\end{array}\right\} x \frac{2.5mg \text{ of chlorine}}{1 \text{ litre}} x \frac{kg}{1x10^6mg}$$

(Henry and Heinke, 1989)

Taking maximum daily rate = 1.5 x average daily rate (Henry and Heinke, 1989).

Required Chorine $= 1306x1.5x10^3 x \frac{2.5mg}{1litre} x \frac{kg}{1x10mg}$ = 4.90kg/day

At the present population, the actual quantity

required per day $= 800 \times 1.5 \times 2.5 \times 10^3 \times \frac{2.5mg}{1litre} \times \frac{kg}{1 \times 10^6 mg}$

= 3kg/day

These values may be altered depending on the raw water qualities.

Total mass of hypochlorite added now

$$= \frac{3}{0.6} = 5.0kg$$

And at 2024, mass of HTH/day

$$= \frac{4.9}{0.6} = 8.17kg$$

Q = $100m^3/hr$.

V = Q x DT.

$$100 \times \frac{1}{3} = 33.3m^3$$

If H = 2.0metres

Consider 2 tanks for ease of dosage and maintenance.

$$A = \frac{33.3}{2} = 16.65$$

$L = B = \sqrt{16.65}$ =4.08m

Freeboard = 0.3m

Dead storage = 0.3m

H = 2 + 0.3 + 0.3 = 2.6m

This unit may be optional in as much as adequate detention time of more than 20minutes would be enhanced in the chlorination chambers.

The schematic diagram of the proposed water treatment plant is shown in fig. 5.1

5.1.7 Ground Level Reservoir

For Q = 100m/hr

For 12 hours reserve

V = $1200m^3$

Dimensions of proposed reservoir are:

h = 7.0m; free board = 0.5

V = $\pi r^2 h$

$$r = \sqrt{\frac{V}{\pi h}} = \frac{1200}{\pi \times 7.0}$$

Provide r = 7.5m

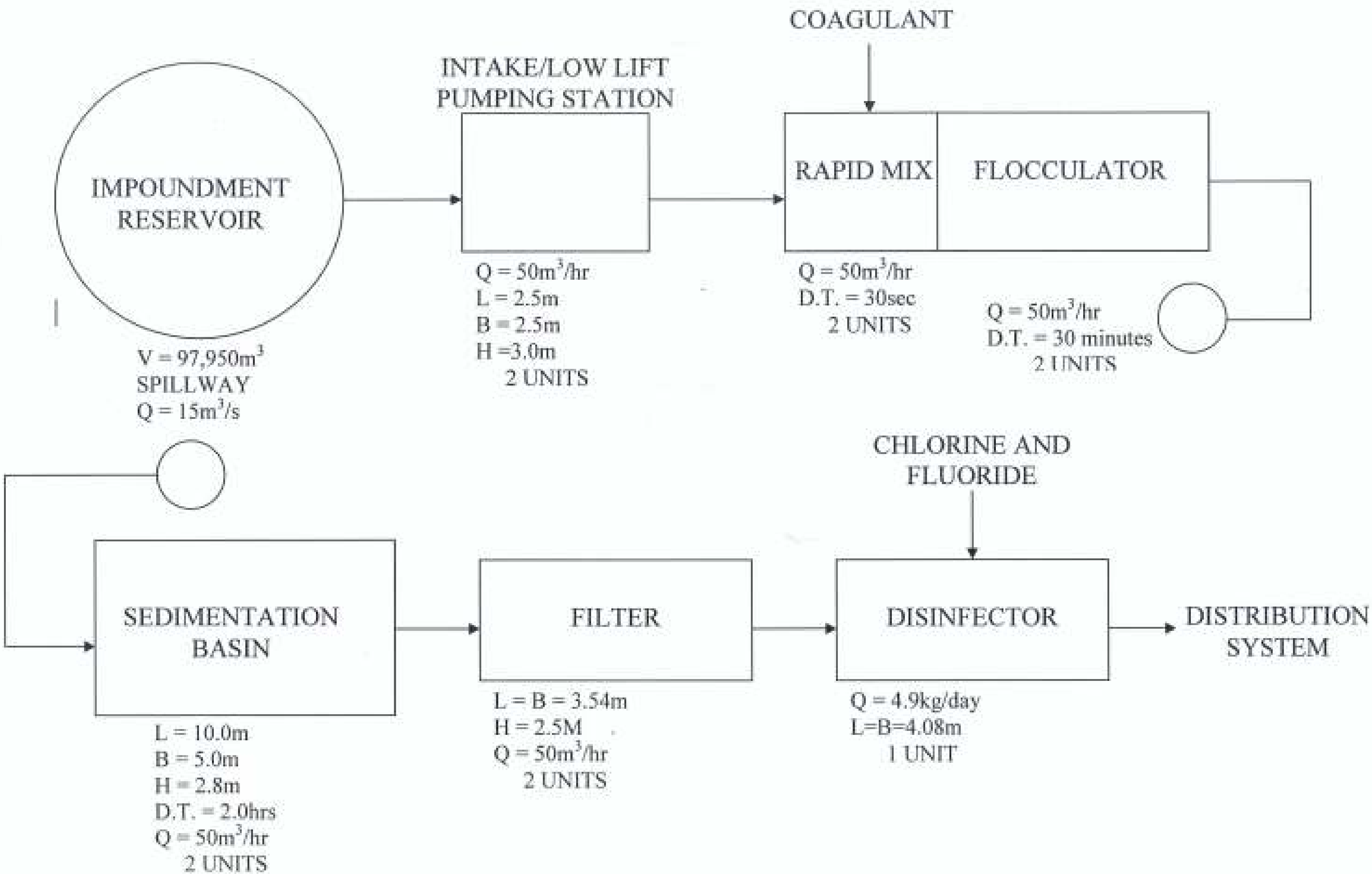

Fig. 5.1: SCHEMATIC DIAGRAM OF THE PROPOSED WATER TREATMENT PLANT.

5.2 STRUCTURAL DESIGN AND OTHER DESIGN CONSIDERATIONS

5.2.1 Structural Design

The structural design of the reinforced concrete reservoir and other water retaining structures are all given in the Appendices VA and VB. Relevant references for the designs are given in the relevant segments of the design calculations.

Radial and tangential moments were considered for the circular reservoir roof while ring tension and bending moments were considered for the circular reservoir wall. Shear considerations were also taken care of in the designs.

The rectangular surface tanks were designed based on bending moments and shear forces imposed by hydrostatic forces from the water when the structures are full. When full, the passive resistance of supporting earth was ignored. When empty, minimum reinforcement was provided as the depth of embedment of the tanks were relatively small without any significant moments on the structures.

5.2.2 Water Reticulation System

The pipe network analysis involves the determination of the pipe flow rates and pressure heads which satisfy the continuity and energy conservation equations.

i.) Continuity:

$$\sum_{I=1}^{I=NP(J)} Q_{IJ} - F_J = 0, \; J = 1, NJ \qquad (1)$$

Where Q_{IJ} = Flow rate in pipe IJ at junction I

NP(J) = No of pipes meeting at junction J

F_J = External flow rate (out flow at J)

NJ = Total no of junction in the network.

(ii) Energy Conservation:

J = NP(I)

$$\sum h_{L},IJ - H_{m,IJ} = O, \quad I = 1, NL$$

$$J = 1$$

Where:

$h_{L,}IJ$ = Head loss in pipe J of loop I

$H_{m,IJ}$ = Manometric head generated by a pump in line I,J.

These equations are non linear and are thus solved by interactive means.

The Darcy-Weisbach equation is given by:

$$h_f = f\frac{L}{D}\cdot\frac{V^2}{2g} = f\frac{L}{D}\cdot\frac{Q^2}{2gA^2} \qquad (3)$$

Where h_f = Friction loss in metres.

L = Length of pipe (metres)

D = Diameter of pipe (metres)

A = Cross sectional Area of pipe (square metres)

Q = Discharge in pipe (cubic metres)

V = Velocity of water flow in pipe (metres per second)

g = Acceleration due to gravity (metres per second squared).

f = Frictional factor and is a function of Reynolds number Re and the relative roughness e/D

e = roughness of pipe.

The Colebrook-White equation is of the form

$$\frac{1}{f^{0.5}} = 1.14 - 2\log_{10}\left(\frac{e}{D} + \frac{9.35}{(\mathrm{Re} * f^{0.5})}\right) \qquad (4) \quad \text{(Larock et al,2000)}$$

$$\frac{1}{f^{0.5}} = 1.14 - 2\log_{10}\left(\frac{e}{D} + \frac{7.3434728VD}{(Q * f^{0.5})}\right)$$

Where $\mathrm{Re} = V\frac{D}{\upsilon} = 4\frac{Q}{(\pi\upsilon D)} = 1.27324\frac{Q}{(\upsilon D)}$

For laminar flow $f = \frac{64}{\mathrm{Re}} = 64\frac{\upsilon}{(VD)} = 81.487\frac{\upsilon D}{Q}$

Equations (3) and (4) are solved simultaneously for D and f through computer iterations. This method of analysis based on a computer visual basic programme was used for the water reticulation system for Oba-Ile estate.

The rising mains and the distribution network of pipes for Oba – Ile estate are shown in fig. 5.2 while the computer programme and the output from the computer programme are given in appendix VI. Appendix VII shows the high lift pumps head calculation and a sample pipeline profile is shown in Appendix VIII.

5.2.3 Raw water Requirement.

A small earth dam is required to impound water from a stream when the available water flow from the stream is unable to sustain demand on regular basis through out the year. The impounded raw water volume must take account of infiltration, evaporation, treatment, transmission and other unaccounted losses as well as the dead storage.

The dead storage is to allow for siltation, of the impounding lake overtime, prevention of mud into the intake, and the preservation of aquatic life.

A fair estimate of the raw water impoundment required is made as follows:

60days volume requirement in 2024		=	78,360m^3
Losses			
i.	Infiltration (5%)		3,918m^3
ii.	Evaporation (5%)		3,918m^3
iii.	Treatment, transmission and other unaccounted losses (2½%)		1,959m^3
iv	Dead storage (10%)		7,836m^3
	Total raw Water Capacity required in 2024		**95,991m^3**

This volume may be impounded at modest flows in less than 4 days.

The details of the computations of the impoundment capacity of the dam are shown in Appendix IX.

5.2.4 Earth Dam Design Considerations

5.2.4.1 A dam must satisfy the stability requirement and prevent piping, seepage or leakages through it in its entire life span. The following elements are very important for the overall safety, stability and functionality of the dam.

(1) **Impervious Clay Core:** This may not be less than a width of 3.6 metres, if enough working room is to be provided for machines and equipment. They must be considered for stable clay compacted at optimum moisture content to the maximum dry density. A practical clay core width for small earth dams is 6 metres.

(2) **Embankment:** The upstream slope (horizontal to vertical) for envisaged materials for the dam is 2.5:1 and a downstream slope of 2.0:1.

(3) **Protection of Embankment:** At the upstream side, dumped rip-rap well positioned, keyed in place and of sizes 200mm to 500mm shall be placed to protect the upstream against wave action and erosion while the down stream embankment shall be protected by grassing to prevent erosion.

4.) **Approach to spillways, stilling basing and dam toe:** The approach to spillways, stilling basin and the toe of the dam shall be protected by dumped rip-rap properly placed and rammed.

5.2.4.2 **Geophysical Investigation and Soil Studies:** The design as well as the construction of any dam must be preceeded by detailed geophysical and soil study. This will provide useful information on the impervious nature of the underlying strata, presence of faults (if any) and the possibility of the reuse of excavated materials

For earth dams, it is actually the characteristics of the soils from the site or approved borrow pits that will determine the slope of the embankment. Besides, the outcome of the studies is important in the derivation of the cost component of the project as haulage consideration significantly affect the cost of earth dams. The earth dam and spillway design considerations are given in Appendix X.

The layout of the dam and the vertical section through the proposed earth dam and its spillway are given in appendices XIA, XIB and XIC respectively.

CHAPTER SIX

6.1 COST ESTIMATES

The total cost of the project as conceived is One Hundred and Twenty Five Million, Eight Hundred and sixty four thousand, Four Hundred and Twenty Four Naira Sixty Five Kobo Only. This cost appears staggering but a breakdown of the cost and the proposed phases of execution show the following:

1.0 Headworks	11,167,500.00
2.0 1200m^3 Ground Level Reservoir	7,195,615.00
3.0 Treatment and Pumping Plant	15,000,000.00
4.0 Pipelines	77,474,500.00
Subtotal	110,837,615.00
Add Consultancy fees (3%)	3,325,128.45
Subtotal 1	114,162,743.45
Contingencies (5%)	5,708,137.17
Subtotal 3	119,870,880.62
Add VAT (5%)	5,993,544.03
Grand Total	**125,864,424.65**

Further details are shown in Appendix XII.

The above cost is for a normal contract through competitive bidding with provision for design, supervision and reimbursable expenses. The cost of the pipelines is about 60% of the proposed project. This is because the design assumed that water will be provided 24 hours per day on year round basis and that the water distribution pipes would be laid to the frontage of each building for easy connection there off. A honest direct labour implementation of the project may bring cost down by at least 25%.

In practice, the Nigerian Government appear to lack the will to provide fund and logistics for water supply projects to be implemented for all communities in the country on such basis. Even when there is a better disposition, paucity of funds had always taken its toll on the projects. The good news however, is that for Abuja and other such sensitive areas, Government strive to make potable water available on a continuous basis.

6.2 Possible sources of fund

The following sources of funding of the project are proposed.

(i) The Landlord Resident Association

(ii) Akure North Local Government Area.

(iii) Ondo State Water Corporation

(iv) Ondo State Ministry of Works Lands and Housing.

(v) Ondo State Ministry of Agriculture.

(vi) Federal Ministry of Water Resources.

(vii) Federal Ministry of Agriculture.

6.3 Important note

In practice, more work is required before arriving at the estimated total cost of the project. This is in the area of soil, geotechnical and geophysical investigations as well as contouring. The results of these investigations more often than not affect the design and the overall cost of the project.

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

1.) There is a need for a separate water supply scheme based on surface water abstraction from Ala River for Oba-Ile Housing Estate, Akure.

2.) Ala River, if properly utilized could provide potable water supply for Oba-Ile Housing Estate, Oba-Ile Community and even major parts of Akure because the raw yield in 2 months would not be less than $2,500,000m^3$. This is about 25 times of Oba-Ile Estate gross raw water requirement for a dry season.

3.) The exponential function $Q = 5.307(h - 0.2378)^{1.0216}$ could best approximate the rating curve of Ala River at the gauging site and as such could be used for the flood prediction of Ala River within the estate.

4.) The developed series model given by the function $y = 37.88\left(Sin2\pi \cdot \frac{(12n+t+7)}{12}+1\right)$ may be used for the predication of water depths and thus the discharges of Ala River at any desired period in the absence of additional data.

5.) A central sewage treatment plant could be designed and constructed by Government for the estate. This is because collection would be easy since virtually all households have water closet systems. The sewage plant could also have an integrated biogas plant to generate cooking gas for the residents.

6.) Some basic construction materials like river sand and granite chippings could be prospected within the estate for the works.

7.) The Ala River corridor is useful for agriculture, fish farming ,wildlife, habitat, lakes development and the development of tourism, youth centres, recreation centres, as well as flood prevention and erosion control.

7.2 Recommendations

Based on these studies, it is recommended that:

1. A separate water supply scheme based on surface water abstraction from Ala River within the Estate should be provided for the Oba-Ile Housing Estate.
2. The service reservoir should be founded on solid rock at a high relief to provide the desirable pressure head to support the reticulation system.
3. Further studies should be carried out on the sewage system for Oba-Ile Estate with the focus of developing a biogas plant to supply cooking gas for the estate.
4. Further works should be carried out on the soil strata of the impounding lake area to determine its reliability for the impoundment of raw water without excessive infiltration or deformation.
5. Further works should also be carried out on the staff gauging of Ala River so that enough statistical records could be available for a better prediction of the hydrological and hydraulic characteristics of Ala River.
6. Dry season farming, fish ponds, coconut and oil palm plantations as well as recreational facilities including swimming pools should be developed by Government in collaboration with the landlord association of the estate along the Ala stream corridor. Employment could be generated for the youths and elderly alike while fund could also be generated for further development and security of the estate.

7.3 Contribution to knowledge

1. The mathematical model given by the time series $y = 37.88\left(Sin\frac{2\pi}{12}(12n+t+7)+1\right)$ developed for river Ala at the gauging location may be used to predict the staff gauges of river Ala at desired periods of time.
2. The rating equation given by $Q = 5.307\ (h - 0.2378)^{1.0216}$ developed for river Ala at the gauging site may be used to predict flows for water supply purposes as well as for the construction of hydraulic structures like spillway, culverts and bridges across the river.

3. The studies shows that at the gauging site, River Ala may not be perennial in contrast to the widely held belief as flows in the middle third of February, 2006 was nil.

4. With an average of three (3) water closets per sampled population of seven persons per household, facael disposal within the Oba-Ile Housing Estate appears effective. However, the relative locations of the soak away systems to the wells may require improvement to prevent contamination.

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APPENDIX I: QUESTIONNAIRE ON WATER SUPPLY/SANITARY DISPOSAL FOR OBA-ILE HOUSING ESTATE, AKURE.

1.) **Name:**... **Occupation:**..

2.) **Address (Number and Street)** ....

....

3.) **Type of House**

i.) Flat	ii.) Duplex	iii.) Face-me-face you	iv.) Others
3 Bed Room ☐ 4 Bed Room ☐	No of Rooms ☐	No of Rooms ☐	

4.) **Population**

a.) Number of Women ☐

b.) Number of Men ☐

c.) Number of Children ☐

5.) **Sources of Water Supply**

a.) i. Public Tap ☐ ii. Borehole ☐ iii. Well Water ☐

iv. Rain Water ☐ v. Others (Specify) ☐

b.) How frequent is public tap running?

Everyday ☐ Once a week ☐ Once a month ☐ Never ☐

6.) **Domestic Water Use**

a.) Drinking water i. Bottled water ☐ ii. Tap water ☐ iii. Well water ☐

iv. Water in nylon packs (pure water) ☐

b.) Type of Toilet

i. Pit toilet ☐ ii. W. C. ☐ iii. Others (Specify) ☐

c.) Number of Toilets ☐

d.) Type of Bathroom

i. Bath-tub ☐ ii. Shower ☐ iii. Others (Specify) ☐

e.) Number of Shower

f.) Do you have Swimming Pool? Yes ☐ No ☐

g.) If Yes, Capacity ☐

7.) **Other Uses of Water**

i. Commercial ☐ ii. Industrial ☐ iii. Watering of Lawn ☐

iv. Gardening ☐ v. Poultry ☐ vi. Piggery ☐

8.) **Type of Storage**

a.) Type of tank(s)

i. Overhead ii. Underground iii. Others (Specify)

b.) Capacity of tank

i. 100gal. ☐ ii. 200gal. ☐ iii. 500gal. ☐

iv. 1000gal. ☐ v. 2000gal. ☐ vi. Other (Specify) ☐

c.) Make of tank

i. Plastic ☐ ii. Metal, Galvanized Iron ☐ iii. Concrete ☐

9.) **Do You Want Government to Give Water?** Yes ☐ No ☐

10.) **Do You Want a Separate Water Supply System for Oba-Ile Housing Estate?**

Yes ☐ No ☐

11.) **What Type of Water Supply System Do You Want from Government?**

i. Borehole ☐ ii. Tap Water ☐ iii. Well Water ☐

iv. Others (Specify) ☐

12.) **Do You Have Permanent Storage for Rainwater** Yes ☐ No ☐

13.) **What Type of Roof Covering is Your Building Made of?**

i. Aluminium Covering ☐ ii. Asbestos Cement ☐ iii. Brick Tiles ☐

iv. Corrugated Iron ☐ v. Others (Specify) ☐

14.) **Are You Willing to Pay Your Water Bill Promptly if Connected to Public Water Supply.**

Yes ☐ No ☐

15.) **How Much Will You be Willing to Pay?**

i. ₦150 ☐ ii. ₦250 ☐

16.) **How Do You Dispose Your Solid Wastes?**

a.) Rubbish.

i. By Subscribing to the waste Management Authority Services ☐

ii. By taking to a dunghill ☐ iii. By Burning ☐ iv. Others ☐

b.) Liquid wastes (Kitchen wastewaters) ☐

i. Released directly to the environment ☐

ii. Released to a soak away ☐

iii. Central sewage disposal system ☐

c.) Faeces Disposal

i. Bushring (Pit toilet) ☐ ii. W.C. system ☐

iii. Is a central waste disposal unit required. ☐

17.) Are You Willing to Get Connected to a Functional Sewage Disposal System

Yes ☐ No ☐

18.) Would You Pay for Such Services if Provided by the Government

Yes ☐ No ☐

APPENDIX II:

COEFFICIENT OF CORRELATION OF THE MEASURED TIME SERIES VERSUS THE FITTED CURVE

	X_i	Y_i	$X_i - \underline{X}$	$Y_i - \underline{Y}$	$(X_i - X)(Y_i - \underline{Y})$	$(X_i - \underline{X})$^2	$(Y_i - \underline{Y})$^2
	37.880	15.54	0.000	-22.34	0.000	0.000	499.076
	18.940	6.84	-18.940	-31.04	587.898	358.724	963.482
	5.076	1.69	-32.804	-36.19	1187.177	1076.102	1309.716
	0.000	12.30	-37.880	-25.58	968.970	1434.894	654.336
	5.076	20.06	-32.804	-17.82	584.567	1076.102	317.552
	18.940	26.66	-18.940	-11.22	212.507	358.724	125.888
	37.880	20.07	0.000	-17.81	0.000	0.000	317.196
	56.820	58.68	18.940	20.80	393.952	358.724	432.640
	70.684	78.20	32.804	40.32	1322.657	1076.102	1625.702
	75.760	104.02	37.880	66.14	2505.383	1434.894	4374.500
	70.684	147.43	32.804	109.55	3593.678	1076.102	12001.203
	56.820	72.47	18.940	34.59	655.135	358.724	1196.468
	37.880	25.54	0.000	-12.34	0.000	0.000	152.276
	18.940	12.63	-18.940	-25.25	478.235	358.724	637.563
	5.076	11.21	-32.804	-26.67	874.883	1076.102	711.289
	0.000	8.75	-37.880	-29.13	1103.444	1434.894	848.557
	5.076	7.11	-32.804	-30.77	1009.379	1076.102	946.793
	18.940	7.35	-18.940	-30.53	578.238	358.724	932.081
	37.880	12.97	0.000	-24.91	0.000	0.000	620.508
	56.820	18.11	18.940	-19.77	-374.444	358.724	390.853
	70.684	66.10	32.804	28.22	925.729	1076.102	796.368
	75.760	38.85	37.880	0.97	36.744	1434.894	0.941
	70.684	32.66	32.804	-5.22	-171.237	1076.102	27.248
	56.820	89.78	18.940	51.90	982.986	358.724	2693.610
	37.880	52.01	0.000	14.13	0.000	0.000	199.657
Total	947.000	947.03			17455.881	17218.186	32775.503
Mean	37.880	37.881					

$$R = \frac{\sum (X_i - \underline{X}) * (Y_i - \underline{Y})}{\sqrt{\sum (X_i - \underline{X})^2 * \sum (Y_i - \underline{Y})^2}} \quad = \quad +0.735$$

$$R^2 \quad = \quad 0.540$$

APPENDIX III (A)

DEVELOPMENT OF THE RATING EQUATION

From fig.6,

h_1 = 0.6m Q_1 = $1.88 m^3/s$

h_3 = 1.8m Q_3 = $8.37 m^3/s$

$Q = C(h-a)^n$

$Q_2 = (Q_3 Q_1)^{0.5} = (1.88*8.37)^{0.5}$

$= 3.9668 m^3/s$

$h_2 = 0.99m$

$$a = \frac{0.6 \times 1.8 - 0.99^2}{0.6 + 1.8 - 2 \times 0.99}$$

$$= 0.2378$$

$$\frac{Q_3}{Q_1} = \frac{8.37}{1.88} = \frac{(1.8-a)^n}{(0.6-a)^n}$$

$$4.4521 = \frac{(1.8-0.2378)^n}{(0.6-0.2378)^n}$$

$$n = \frac{\log 4.4521}{\log 4.314}$$

$$= 1.0216$$

$$Q = C(h_3 - a)^{1.0216}$$

$$8.37 = C(1.5773)^{1.0216}$$

$$C = \frac{8.37}{1.5773} = 5.307$$

:- $Q = 5.307(h - 0.2378)^{1.0216}$

This is the rating equation.

APPENDIX III(B)

The Development of Time Series Model

From the plotted data, a sine curve may offer a good fit. However, sine takes a maximum value of +1 and a minimum value of -1. The fitted curve may not take negative values since the gauging station is on rock which is most probably part of the basement complex, hence the proposed equation is of the form $y = A(Sin\theta + 1)$.

But θ is time dependent, therefore θ = Ct. Where C is a constant in degrees

Where A is the amplitude and θ is the varying angle.

Consider time t = 1, 2, …12 for January to December since mean monthly flows is time dependent.

For a complete cycle of 12 months $\theta = 360^0 = 2\pi$

For each month $\theta = \frac{360^0}{12} = 30^0 = \frac{2\pi}{12}$

$$\theta \quad = \quad \frac{2\pi}{12}(1, 2, 3, 4, \ldots 12)$$

To determine the amplitude A of the curve, consider a 24 months duration or any convenient multiples of 12 since each complete season is 12 months.

Integrating the Sine equation over 24 months,

$$\int_0^{24} y dt = \int_0^{24} A(SinCt + 1)dt$$

$$= \quad A\left[\frac{-CosCt}{C} + t\right]_0^{24}, \text{ For } \theta = 30^0$$

$$= \quad A\left[\frac{-Cos24 \times 30^0}{C} + 24 + \frac{Cos0^0}{C} - 0\right]$$

$$= \quad A\left[-\frac{Cos0^0}{C} + 24 + \frac{Cos0^0}{C}\right]$$

$$= \quad 24A = \sum \bar{h}m$$

$$\because \quad A \quad = \quad \frac{\sum \bar{h}m}{24}$$

Take ASinCt + 1 as $A\left(Sin\frac{2\pi}{12}(12n+t+a)+1\right)$

For n = 0

Becomes $A\left(Sin\frac{2\pi}{12}(t+a)+1\right)$

Check for February 2002, where

SinCt + 1 = 0

$Sin270^0$ = -1

$$\Rightarrow \quad \frac{2\pi}{12}(2+a)=270^0$$

$$\frac{360}{12}(2+a)=270^0$$

$$2+a=\frac{270^0}{30^0}=9$$

a = 9 – 2 = 7.

From above:

$$y_{(t)}=A\left(Sin\frac{2\pi}{12}(12n+t+7)+1\right)$$

Where A = $\left(\Sigma\bar{h}\right)_{mean}$

n = 0, 1, 2, 3, 4, 5, ...= No. of years.

t = 1, 2, 3, ... 12 from January to December.

APPENDIX IV

CORRELATION OF ACTUAL DISCHARGES VERSUS DISCHARGES FROM FITTED CURVE.

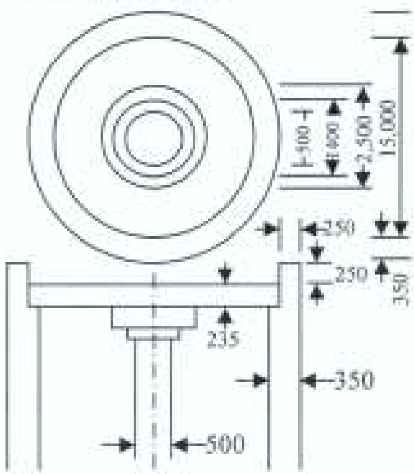
DEPTH	ACTUAL DISCHARGE	DISCHARGE (FITTED CURVE)					
m	$m^3/_s$	$m^3/_s$	$m^3/_s$	$m^3/_s$			
	Xi	Yi	$Xi-\overline{X}$	$Yi-\overline{Y}$	$(Xi-\overline{X})(Yi-\overline{Y})$	$(Xi-\overline{X})^2$	$(Yi-\overline{Y})^2$
0.4	1.17	0.83	-3.97	-4.29	17.061	15.796	18.426
0.6	1.88	1.88	-3.26	-3.24	10.576	10.657	10.496
0.8	3.01	2.95	-2.13	-2.17	4.639	4.556	4.724
1.0	4.01	4.02	-1.13	-1.10	1.247	1.287	1.207
1.2	5.08	5.01	-0.06	-0.02	0.001	0.004	0.000
1.4	6.08	6.19	0.94	1.07	0.999	0.875	1.140
1.6	7.21	7.28	2.07	2.16	4.456	4.267	4.654
1.8	8.37	8.37	3.23	3.25	10.485	10.404	10.507
2.0	9.49	9.47	4.32	4.35	18.890	18.884	18.896
Σ	46.30	40.18	41.16	-0.86	68.354	66.730	70.111
Mean	5.14	4.46					

$$R = \frac{\sum\left(X_i-\overline{X}\right)*\sum\left(Y_i-\overline{Y}\right)}{\sqrt{\sum\left(X_i-\overline{X}\right)^2*\sum\left(Y_i-\overline{Y}\right)^2}} = +0.9993 = +1.00$$

$$R^2 = 0.9986 = 1.00$$

APPENDIX V A

STRUCTURAL DESIGN OF 1200M^3 REINFORCED CONCRETE GROUND LEVEL RESERVOIR

REFERENCE	CALCULATIONS	OUTPUT
	<u>ROOF SLAB</u>	
	<u>Dead Load D. L.</u>	
	(i) Self weight of concrete	
	$= 24 \times 0.235 = 5.6kN/m^2$	
	(ii) Gravel $= 1.80kN/m^2$	
	$\Sigma DL = 7.44N/m^2$	$Gk = 7.44kN/m^2$
	Live Load	
	Live Load $= 1.5kN/m^2$	
	Water $= 1.0kN/m^2$	
	$\Sigma LL. = 2.5kN/m^2$	$Qk = 2.5kN/m^2$
	Design load for ultimate limit state	
Table 2.1	$w = 1.4\ Gk + 1.6\ Qk$	
B88110	$= 1.4 \times 7.44 + 1.6 \times 2.5$	
	$w = 14.42kN/m^2$	$w = 14.42kN/m^2$
		
	All dimensions in mm.	
	Fig 1: Roof plan of reservoir	

REFERENCE	CALCULATIONS	OUTPUT
	Estimation of Effective Depth	
Table 3.3 BS8110	h = 235mm	
	c = 40mm, ϕ = 16mm, t = 235mm	
	d = h – c – ϕ/2	
	d = 235 – 40 – 8 = 187mm	
NCP page 61	Moments in Roof Slab	d = 189mm
	(i) Radial moment Mr = α_1wR	
	(ii) Tangential moment Mt = α_2wR	
	Where α_1 and α_2 are coefficient from table I	
	For $\frac{C}{D} = \frac{1}{10.10} = 0.099 = 0.1$ $wR^2 = 14.42 \times 7.5^2 = 811.13$	
	Table I: Variation of Radial and Tangential Moment at Various Points Along the Radius of the roof slab.	

POINT (m)	α_1	Mr (kNm)	α_2	MT (kNm)
0.1R 0.75	- 0.2487	- 201.00	- 0.0496	- 40.23
0.2R 1.50	- 0.0557	- 45.15	- 0.0684	- 55.48
0.25R 1.88	- 0.0176	- 14.28	- 0.0539	- 43.72
0.3R 2.25	+ 0.0081	+ 6.57	- 0.0394	- 31.96
0.4R 3.00	+ 0.0391	+ 31.71	- 0.0153	- 12.41
0.4R 3.75	+ 0.0539	+ 43.72	+ 0.0020	+ 1.62
0.6R 4.50	+ 0.0578	+ 46.88	+ 0.0134	+10.87
0.7R 5.25	+ 0.0532	+ 43.16	+ 0.0197	+15.98
0.8R 6.00	+ 0.0416	+ 33.74	+ 0.0218	+17.68
0.9R 6.75	+ 0.0237	+19.22	+ 0.0199	+16.14
1.0R 7.50	0	0	+ 0.0145	+11.76

REFERENCE	CALCULATIONS	OUTPUT
	<u>RADIAL MOMENT (Mr)</u>	
	<u>Negative Moment</u>	
	Max Mr occurs at 0.1R = 0.750mm from the centre.	
PCA Page 16	(i) Max Mr = -201.00KNM	
	This theoretical Mr is larger than actual Mr. Therefore allow for 28% reduction	
	Max Mr = 201.00 (1-0.28)	
	= 144.72KNM	
	d $= 335-40-\frac{16}{2}$	
	= 287mm	
Mosley & Bungey fig. A.1	$\frac{M}{fcubd^2} = \frac{144.72\times10^6}{25\times10^3\times287^2}$	
Table 3.1	K = 0.07	
BS8110	$Ia = 0.5 + \left(0.25 - \frac{k}{0.9}\right)^{\frac{1}{2}} = 0.915$	
	$As = \frac{144.72\times10^6}{0.915\times460\times0.95\times287} = 1260mm^2$	
	Provide Y16 – 150T (1340mm^2) radial bars on top of slab up to 0.20R (1500mm)	Provide Y16 – 150T (1340mm^2) radial bars up to 1500mm of the radius of the roof slab
	At Mr = -45.18 KNM	
	d = 235 – 40 – 16/2 = 187mm	
	$\frac{M}{fcubd^2} = \frac{45.18\times10^6}{25\times10^3\times187^2} = 0.010$	
	$Ia = 0.5 + \left(0.25 - \frac{0.052}{0.9}\right)^{\frac{1}{2}} = 0.94$	
	$As = \frac{45.18\times10^6}{0.94\times460\times0.95\times187} = 588mm^2$	

REFERENCE	CALCULATIONS	OUTPUT
Threlfall Table 3	Provide Y16 – 300T (670mm^2)	Provide Y16-300T radial bars up to 2.25m of rooftop.
	Positive Moment	
	(ii) Max (+ve) moment accurs at 0.6R (= 4500mm from centre)	
Threlfall Table 3	Mr = 46.88 x 10^6 KNM $\frac{M}{fcubd^2} = \frac{46.88 \times 10^6}{25 \times 10^3 \times 189^2} = 0.054$	
Table 3.1 BS8110	$la = 0.5 + \left(0.25 - \frac{0.052}{0.9}\right)^{\frac{1}{2}} = 0.94$	
	$As = \frac{46.88 \times 10^6}{0.94 \times 460 \times 0.95 \times 187} = 610mm^2$	
	Provide Y12 – 175B (646mm^2)	Provide Y12 – 175B radial bars at roof slab bottom
	(iii) At 0.9R, Mr = 19.22KNm	
	$\frac{M}{fcubd^2} = \frac{19.22 \times 10^6}{25 \times 10^3 \times 187^2} = 0.02$	
Threlfall Table 3	la = 0.950	
	$As = \frac{19.22 \times 10^6}{0.95 \times 460 \times 0.95 \times 187} = 248mm^2$	
	Provide Min. Reinforcement = 0.0015 x 10^3 x 187 = 281mm^2 Provide Y12 - 300B (376mm^2)	Provide Y12 - 300B radial bars from 6000 to the edge of the wall.

REFERENCE	CALCULATIONS	OUTPUT
	Tangential Moments	
	Max (-ve) moment at 0.2R = -55.48KNM $d = 335 - 40 - 16 - 6 = 273mm$ $\frac{M}{fcubd^2} = \frac{55.48 \times 10^6}{25 \times 10^3 \times 287^2} = 0.03$	
Threlfall Table 3	Ia = 0.95 $As = \frac{55.48 \times 10^6}{0.95 \times 460 \times 0.95 \times 273} = 490mm^2$	
	Provide between centre and 0.4 R i.e. (0 to 3.00m) Y12-200T (566mm^2)	Provide 15Y12-200T circumferential reinforcement between 0m to 3.0m at roof top
	Max (+ve) moment at 0.8R (6.0m) = 17.68KNM $d = 235 - 40 - 12 - 6 = 177$ $\frac{M}{fcubd^2} = \frac{17.68 \times 10^6}{25 \times 10^3 \times 177^2} = 0.02$	
Threlfall Table 3	Ia = 0.950 $As = \frac{17.68 \times 10^6}{0.95 \times 460 \times 0.95 \times 177} = 240mm^2$	
	Provide min As = 0.0015 x 1000 x 187 = 281mm^2 Provide Y10 – 275B (285mm^2)	Provide Y10 – 275B circumferential reinforcement at roof bottom
	Shear Stress Considerations **Drop Panel**	
Mosley & Bungey fig.8.2	Length of critical perimeter = 2π (2.5 + 1.5 x 0.235) = 17.92m Area within critical perimeter	

REFERENCE	CALCULATIONS	OUTPUT
Mosley & Bungey Table 5.1	$= \frac{\pi}{4}(2.5 + 1.5 \times 0.235)^2 = 6.39m^2$ V_u= 1.5 x 14.42 (7.675^2 – 6.38) = 1135.9/KN $v = \frac{Vu}{bd} = \frac{1135.91 \times 10^3}{17920 \times 189} = 0.34 N/mm^2$ % As = 0.44, d = 187 $vc = 0.6 \times \frac{(25)^{1/3}}{25} \bullet \frac{1}{1.06}$ = 0.59N/mm^2 v = 0.34N/mm^2 < vc = 0.59N/mm^2	Shear stress O.K. across drop panel
	Column Head Length of critical perimeter = 2π (1.4 + 1.5 x 0.335) = 11.955m Area within critical perimeter	
Mosley & Bungey Table 5.1	$\frac{\pi}{4}(1.4 + 1.5 + 0.335)^2 =$ = 2.843m^2 $Vu = 1.5 \times 14.42\left(\frac{\pi}{4} \times (7.675^2 - 2.843)\right)$ = 952. 4KN $v = \frac{Vu}{bd} = \frac{952.4 \times 10^3}{11955 \times 287} = 0.43 N/mm^2$ % As = 0.44, d = 287. v_c = 0.55N/mm^2 v= 0.43 N/mm^2< vc = 0.55N/mm^2	Shear stress across column head is okay.

REFERENCE	CALCULATIONS	OUTPUT
	<u>TEMPERATURE STRESSES</u> $Ec = 21000N/mm^2$ t = 350mm $\varepsilon c = 0.00001\ per^0C$ $Ic = \frac{hc^3}{12}$ $T_1 = 20^0C$ (Inside temperature) Compression face $T_0 = 35^0C$ (Outside temperature) Tension face $\varepsilon cEc = 2100 \times 0.00001 = 0.21$ Coefficient of thermal conductivity of concrete kc = 1.5 Resistance at external face % = 0.09m $(w)^{-1}$ Es = $210{,}000Nmm^2$ εc = 0.00001111 per % $T^0 = (T_0 - T_1) \cdot \frac{khc}{kc}$ Where $\frac{1}{k} = a_0 + \frac{hc}{kc}$ $T^0 = 15 \cdot \left\{ \frac{1}{0.09 + \frac{0.35}{1.5}} \right\} \cdot \frac{0.35}{1.5}$ $= 10.82^0C$ $= M_T = T^0 \varepsilon cEc \cdot \frac{Ic}{hc}$ $= 10.82 \times 10^{-5} \times 21 \times 10^3 \left(\frac{350^2}{12}\right)$ = 23.2KNm/mrun	

REFERENCE	CALCULATIONS	OUTPUT
Reynolds & Steedman page 116	$As = \dfrac{M}{0.95fsd}$ $= \dfrac{23.2 \times 10^6}{0.95 \times 130 \times 300} = 626.2mm^2$ Add to the regular tension steel AsT = 626mm^2	
Table 11.2 Mosley & Bungey	$fc_\gamma = 0.5T^{\circ}\varepsilon sEs$ = 0.5 x 10.82 x 10^{-5} x 21 x 10^3 = 1.14N/mm^2 < 1.31N/mm^2	Temperature stresses is okay.

REFERENCE	CALCULATIONS	OUTPUT
	RING TENSION	
	Fixed base free top	
Table A3.1 B S 5337	Ft = Coef*γHr, γ = 9.81KN/m^3	
	H = 7.25m	
	r = 7.5m	
	t = 0.35m	
	Ft = 533.4188 *Coef	
	d = 15.0m (diameter of reservoir)	
	αe = 15	
	Ac = 350000mm^2	
	H^2/td = 10.0119	
	Table XII: Variation of Ring Tension with depth of Reservoir	

h	h(m)	Coeff	Ft (KN/m)
0.0h	0.000	- 0.011	-5.87
0.1h	0.725	0.098	52.28
0.2h	1.450	0.208	110.95
0.3h	2.175	0.323	172.29
0.4h	2.900	0.437	233.10
0.5h	3.625	0.542	289.11
0.6h	4.450	0.608	324.32
0.7h	5.075	0.589	314.18
0.8h	5.800	0.44	234.70
0.9h	6.525	0.179	95.48

REFERENCE	CALCULATIONS	OUTPUT
	A_{ST} = 627mm^2	
	First 3.5m, A_{st} = Ft/fct = 233.10*1000/130 = 1793.077mm^2	First 3.5m Provide 2Y16– 150, one on each face of wall
	Ast+A_{ST} = 1793.08+627 = 2420mm^2	
	Provide 2Y16_150, A_{SP} = 2680mm^2 one on each face	

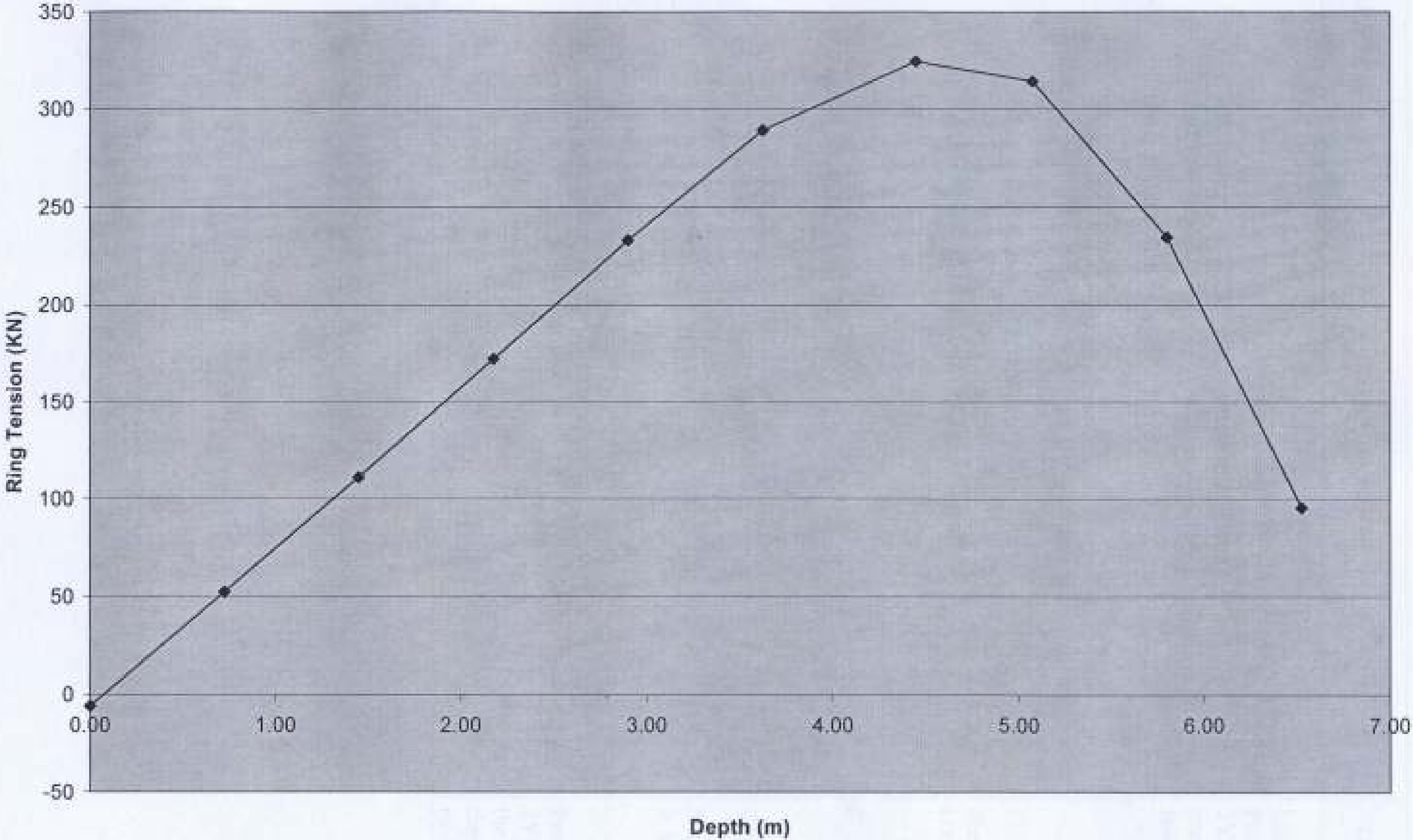


Fig. 8.1: Variation of Ring Tension Vs Depth of the Reinforced Ground Level Reservoir

REFERENCE	CALCULATIONS	OUTPUT
	Last 4m A_{ST} = 324.32x100/130 = 2,494.769, A_{ST} + AST= 3120.7mm Provide 2Y20_200, Asp = 3140mm2 At =Ac+(αe-1)As At_1 = 350000 + 14 x 2680 = 378,520 $fct_1 = \frac{233.1\times10^3}{387{,}520} = 0.60 N/mm^2$ fct_1 = 0.06N/mm^2 < 1.31N/mm^2 At_2 350,000 + 14 x 3140 = 393,960 mm^2	Last 4m Provide 2Y20–200, one on each face of wall. Shear stress is satisfactory
	$fct_2 = \frac{fct_2}{At_2}$ $= \frac{324.2 \times 10^3}{393{,}960} = 0.82N/mm^2$ ft_2 = 0.82N/mm^2 < 1.31N/mm^2	Shear stress is satisfactory
	Last 1.5m Ft = 234.7 Provide 2Y16 – 150	Provide 2Y16-150 in the last 1.5m depth of reservoir

REFERENCE	CALCULATIONS	OUTPUT
Table A3.3 B S 5337	**BENDING MOMENT** d = 350 – 40 –20 – 6 = 284mm M = γh^3*Coef $\frac{\hat{h2}}{td} = \frac{7.25^2}{0.35X15} = 10.0119$	

Table XIII : Variation of bending moment with depth

h	h(m)	γh^3	Coef	γh^3*Coef
0.1h	0.000	0.000	0.0000	0.00
0.2h	0.725	0.381	0.0000	0.00
0.3h	1.450	3.049	0.0001	0.00
0.4h	2.175	10.289	0.0004	0.04
0.5h	2.900	24.389	0.0007	0.17
0.6h	3.625	47.635	0.0019	0.89
0.7h	4.350	82.313	0.0029	2.34
0.8h	5.075	130.710	0.0028	3.59
0.9h	5.800	195.112	-0.0012	-2.30
1.0h	6.525	277.806	-0.0122	-33.25

Vertical reinforcement

Max M+ = 3.59KN/m,

M = -33.25KN/m

$$K = \frac{M}{fcub\hat{d}^2} = \frac{3.59\times10^6}{25\times1000\times284^2} = 0.002$$

$$As = \frac{3.59\times10^6}{0.95\times0.95\times284\times425} = 32mm^2 < As\min$$

Provide min reinforcement = 0.0013 x bh

=0.0013 x 1,000 x 284 = 370mm^2

2Y12 – 250 one on each face

As/face =452mm^2

$$\frac{\hat{H}2}{td} = 10.0119$$

V = Coef x γh ^2

$= 0.105\times7.25^2\times9.81$

V = 54.14KN

v V/bd = 0.19N/mm^2

A_{sp}= 15%, vc = 0.36N/mm^2

v = 0.19N/mm^2 <vc 0.36N/mm^2

OUTPUT:

Provide Y12– 250 vertical reinforcement on each face of wall.

Shear stress at the base of Wall is okay.

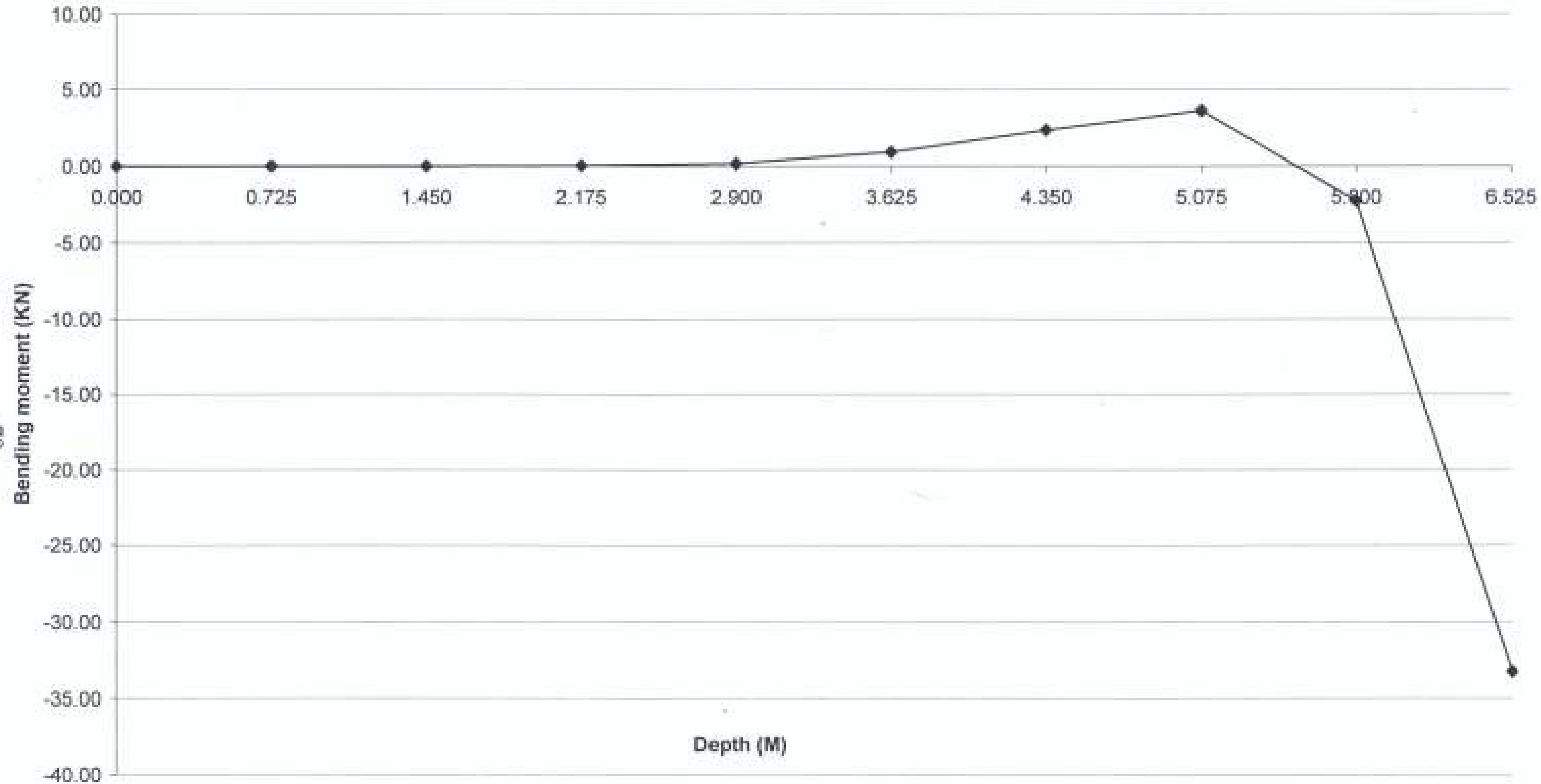


Fig 8.2 : Variation of ring bending moment vs depth of wall of reinforced concrete ground level reservior

REFERENCE	CALCULATIONS	OUTPUT

Wall Foundation

		FMN/m	r (m)	Fr KNm
1.Wall	40.28 x 0.35 x 7.5 x 24	103.28	7.675	792.67
2. Foundation	(0.165 x 0.06 x 0.1375)24= 0.3625 x 24	8.7	7.49	65.16
3. Water	0.65 x 7.25 x 9.81	46.23	7.175	331.70
Full	ΣF =	158.21	ΣF=	1,189.53
Empty	ΣF =	111.98	ΣF=	857.83

Gk = 7.44kN/m^2
Load from roof =14.42 x π x 7.85^2

Load on Column = 2,791.61kN

DL = 1.4 x 7.44 x 7.675^2 = 613.56KN

L.L = 1.6 x 2.5 x 7.675^2 = $\frac{235.62}{849.18KN}$

Load on Wall = 2791.61 – 849.18 = 1942.43 KN

Load Carried per metre of wall = $\frac{1942.79}{\pi(15.35)} = 40.28\,KN/m$

If reservoir is full $\frac{\sum Fr}{\sum F} = \frac{1189.53}{158.21} = 7.52$

If reservoir empty $\frac{\sum Fr}{\sum F} = \frac{857.83}{111.98} = 7.66$

$f_{base} = \frac{8.7}{1.2\times 1} = 7.25\,KN/m^2$

$f_{max} = \frac{F}{bL}\left(1+\frac{6e}{L}\right) = 7.25\,KN/m^2$

$= \frac{158.21}{1.2\times 1}\cdot(1+6\times 0.155)$

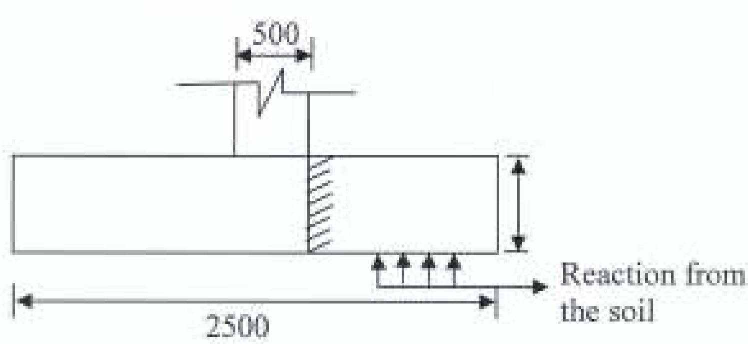
$= 234.2$KN/m^2

REFERENCE	CALCULATIONS	OUTPUT
	$f_{min} = \frac{158.21}{0.95}\left(1-\frac{6\times0.155}{1.2}\right)$	
	$= 29.66KN/m^2$	
	$f_{xx} = f_{min} + \left(\frac{l_1+a}{L}\right)\cdot(f_{max}-f_{min})$	
	$= 29.66 + \frac{(0.2+0.35)}{1.2}(234.02-29.66)$	
	$= 123.325KN/m^2$	
	$M_{xx} = \frac{1}{6}l_2^2(f_{xx}+2f_{max}-3f_{base})$	
	$= \frac{1}{6}(0.65)^2(123.325+2(234.02)-3(7.25))$	
	$= 40.1KNm$	
	$d = 600-40-6 = 554$	
	$\frac{M}{fcubd^2} = \frac{40.11\times10^6}{25\times1000\times554^2} = 0.005$	
	$As = \frac{40.11\times10^6}{0.95\times0.95\times554\times425} = 188mm^2$	
	$= 188mm^2 <$ Min. Permissible reinforcement	
	Min reinf = $0.0015 \times 10^3 \times 554$	
	$= 831mm^2$	
	Y12 – 200 ($1010mm^2$)	Provide Y12-200 in wall foundation
	$V_{xx} = l_2(f_{xx}+f_{max}-2f_{base})$	
	$=$ ½ (0.65) (123.3254 x 234.02 – 2 (7.25))	
	$= 111.42KN$	
	$v = \frac{111.98\times10^3}{1000\times554} = 0.20N/min^2$	
	$v_c = 0.36+\frac{0.03}{0.10}(0.42-0.36)$	
	$= 0.38\ N/mm^2$	
	$v = 0.20N/mm^2 < Vc = 0.38N/mm^2$	

REFERENCE	CALCULATIONS	OUTPUT
	If reservoir is empty e = $0.015 < \frac{1.2}{6} = 0.02$	
	$f_{max} \quad = \quad \frac{111.98}{1 \times 1.2}\left(1 + 6\left(\frac{0.015}{1.2}\right)\right)$	
	$= \quad 100.32KN/m^2$	
	$f_{min} \quad = \quad \frac{111.98}{1 \times 1.2}\left(1 - 6\left(\frac{0.015}{1.2}\right)\right)$	
	$= \quad 86.32KN/m^2$	
	$f_{xx} \quad = \quad 86.32 + \frac{0.65}{1.2}(100.32 - 86.32)$	
	$= \quad 93.91KN/m^2$	
Table 3.8 BS8110	$M_{xx} \quad = \quad \frac{1}{6}$ (0.65)2 (93.91 + 2 (100.32)-3 (7.25)	The reinforcement provided is okay.
	$= \quad 19.21KNm$	

REFERENCE	CALCULATIONS	OUTPUT
Mosley and Bungey eq. 9.3	**Column** i.) Loading on column head from roof W_1 = D.L = 1.4 x 7.675^2 x 7.44 = 613.56 W_2 = L.L = 1.6 x 7.675^2 x 2.50 = 235.62 849.18KN ii.) Weight of drop panel W_3 = 1.4 x 2.5 x 2.5 x 0.15 x 24 = 35.50KN iii.) Self weight of column head W_4 = $1.4 \times 24 \times \pi (0.7^2 - 0.25^2) \times 0.3$ = 13.54KN iv.) Self weight of column $W_5 = 1.4 \times 24 \times \frac{\pi}{4}(0.5)^2 \times (7.5 - 0.15) =$ 61.74 959.96KN **Column Reinforcement** N = 0.35 fcu Ac + 0.7 Asc fy. For column head N = 884.68KN $Ac = \frac{\pi}{4}(1.4)^2 = 1.54m^2$ $Asc = \frac{884.68 \times 10^3 - 0.35 \times 25 \times 1.4 \times 10^6}{0.7 \times 425}$ = -ve Provide min. Asc $= \frac{0.13}{100} \times \frac{\pi}{4}(1.4)^2 \times 10^6$ $= 2001mm^2$ $NY_{12} = \frac{2001}{113} = 17.7$	Provide 18 Y12 along column head

REFERENCE	CALCULATIONS	OUTPUT
	For the column, N = 959.9KN Ac = $\frac{\pi}{4}(0.5)^2 = 0.196$ Asc = $\frac{959.9 \times 10^3 - 0.35 \times 25 \times 0.196 \times 10^6}{0.6\ 460}$ = -ve Provide minimum reinforcement = $\frac{1}{100} \times \frac{\pi}{4}(0.5)^2 \times 10^6 = 1964mm^2$ Provide 10 Y16, Asp = $2010mm^2$ = $10.82 \times 10^{-5} \times 21 \times 103\ \frac{(350)^2}{12}$ **Wall** Minimum thickness of wall tmin	Provide 10 Y16 in circular column
Reynolds and Steedman Table 135	tmin = $\delta h \cdot \frac{1}{2} d\left(\frac{1}{fct} - \frac{\alpha - 1}{fst}\right)$ δ = $9.81 \times 10^3 N/m^3$ fct = $1.31N/mm^2$ fst = $115N/mm^2$ tmin = $9.81 \times 10^3 N \times \frac{7.25}{2000} \times 15\left(\frac{1}{1.31} + \frac{14}{115}\right)$ = 342.3mm <350mm.	Wall thickness is okay

REFERENCE	CALCULATIONS	OUTPUT
	Bending moment due t restrain at the base of slab.	
Reynolds and Steedman Table 185	$\frac{h^2}{td} = \frac{7.25^2}{0.35 \times 15.0} = 10.01$	
	K_1 = 0.012	
	K_2 = 0.364	
	K_3 = 0.62	
	N_{max} = $\frac{\delta}{2} hdK_3$	
	= 0.5 x 9810 x 7.25 x 15 x 0.62 = 330.72kw/m	
	height of wall.	
	M_{max} = $K1\gamma h^3$	
	= 0.012 x 9810 x $(7.25)^3$	
	= per metre sum of circumference position of maximum circumferential tension.	
	= K_2h = 0.364 x 7.25 = 2.64m from bottom reservoir	
	Column Foundation	
	N_{max} = 959.96KN	
	Thickeness of foundation = 0.600	
		
	Soil bearing pressure => 500KN/m^2 (rock)	
	A_{freq} = $\frac{959.96}{500} = 1.92m^2$	
	Provide square foundation – 2.5 x 2.5 = 6.25m^2	

REFERENCE	CALCULATIONS	OUTPUT

Punching shear.

$q = \dfrac{959.96}{6.25} = 153.54\,KN/m^2$

For /m

W_c = Strip = 24 x 0.60 = 14.4KN/m^2

W_w = 7.25 x 9.81 = 71.12KN/m^2

239.06KN/m^2

Length of Cantilever

$L = \dfrac{2.5 - 0.5}{2} = 1.0$

$d = 600 - 40 - 8 = 552\text{mm}$

$M = \dfrac{W_T L^2}{2}$

$= \dfrac{239.06 \times 10^6}{2} \text{ x } 1.0 = 119.53\text{KNm}$

$\dfrac{M}{fcubd^2} = \dfrac{119.53 \times 10^6}{25 \times 10^3 \times 552^2}$

$= 0.016$

$Ia = 0.95$

$As = \dfrac{119.53 \times 10^6}{25 \times 10^3 \times 552^2}$

$= 522\text{mm}^2$

Minimum reinforcement = 0.13% x 10^3 x 552

= 718mm^2

Provide Y16 – 225 Asp = 900mm^2

Output: Provide Y16-225 at the bottom of column foundation in both direction

Local Bond Stresses

$V = 239.06\text{KN/m}^2$

For 1metre width

$V = 239.06 \times 1.0 \times 1.0 = 239.06\text{KN}$

$\Sigma Us = \dfrac{1000}{225}\pi \bullet 16 = 223.402\,mm$

<table>
<tr><th>REFERENCE</th><th>CALCULATIONS</th><th>OUTPUT</th></tr>
<tr>
<td>

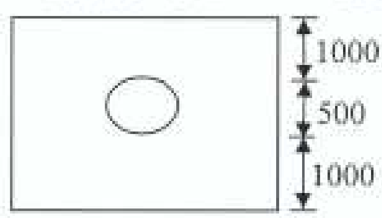
Mosley and Bungey Table 5.2

Mosley and Bungey Table 5.2</td>
<td>
Where ΣUs = The sum of the perimeters of the reinforcement bars.
Fbs = $\dfrac{V}{\sum Usd} = \dfrac{239.06\times10^3}{223.4\times552}$
= 1.94N/mm^2
fbu = $\beta\sqrt{fcu}$
= $0.4(25)^{0.5}$ = 2.0N/mm^2
fbs = $1.94\text{N/mm}^2 <$ fbu = 2.0N/mm^2

Punching Shear in foundation

d = 0.5 + 3(0.6) = 2.3

Critical perimeter π (2.3) = 7.225m

Area within critical perimeter = $\dfrac{\pi}{4}(2.3)^2 = 4.15m^2$
Net upward pressure = pressure on rock -pressure due to foundation
= 239.06 – 14.4 = 224.66KN/m^2


<u>Punching Sheer</u>

Punching Shear force V = $224.66\ (2.5^2 - 4.155)$

= <u>470.66KN</u>

Punching Shear Stress (vc) = $\dfrac{470.66\times10^3}{7225\times550}$

= 0.12N/mm^2

$\%As = \dfrac{900\times100\%}{1{,}000\times550} = 0.16\%$

$V_{cu} = (0.36 + 0.42 - 0.36)\times\dfrac{0.01}{0.10}$

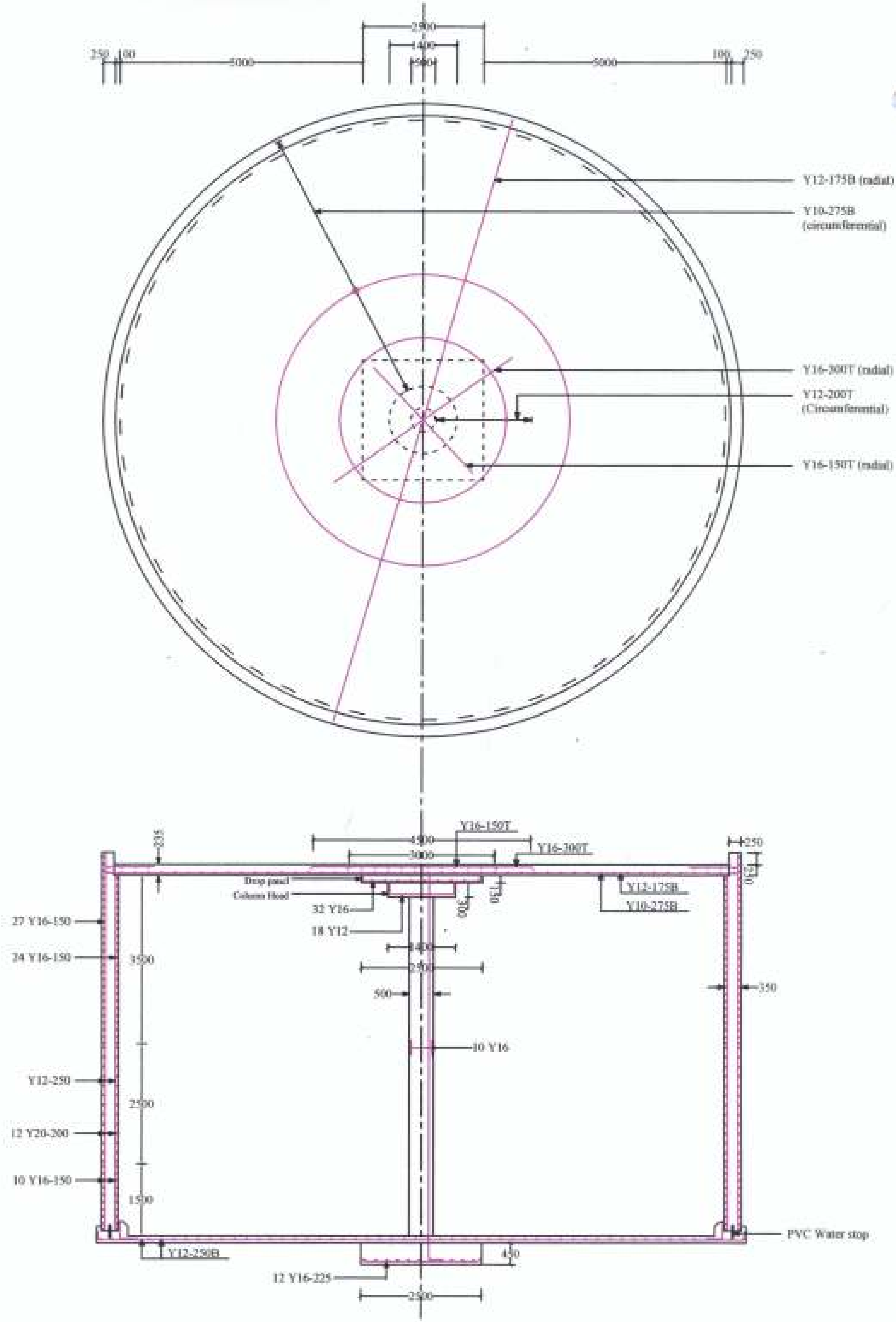
$= 0.37\text{N/mm}^2$
$V_c = 0.12\text{N/mm}^2 < V_{cu} = 0.37\text{N/mm}^2$
</td>
<td>

Bond stress is within tolerance limits

Punching Shear stress is okay</td>
</tr>
</table>

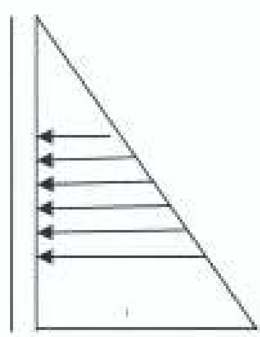
REFERENCE	CALCULATIONS	OUTPUT
	Anchorage length for the dowel bars	
	Compression anchorage length	
	$= 0.3\phi$	
	$L = \frac{0.95 f_y \phi}{4 f_{bu}}$	
	$f_{bu} = 0.63\,(f_{cu})^{0.5} = 0.63\,(25)^{0.5} = 3.15\text{N/mm}^2$	
	$L = \frac{0.95 \times 425 \times 16}{4 \times 3.15}$	anchorage length is adequate
	$= 513\text{mm}$	
	h = 60mm is ok	
	Base Slab Design	
	Permissible bearing pressure = 500KN/m^2	
	W_1 = Self weight of slab $= 1.4 \times 0.15 \times 24 = 5.04\text{KN/m}^2$	
	Load due to weight of water W2 $= 1.6 \times 7.25 \times 9.81$	
	$= 113.78\text{KN/m}^2$	
	$\underline{\underline{118.82\text{KN/m}^2}}$	
	Consider floor a 2 way slab $118.82\text{KN/m}^2 < 500\text{KN/m}^2$	
	Provide min reinforcement in $0.13\% \times 1000 \times 150 = 195\text{mm}^2$	
	Provide Y12 – 250B	Provide 12-250B along both direction base on Sob.
	$M_{sx} = M_{sy} = 0.062 \times 5.04 \times 7.675 = 18.41\text{KNm}$	
	$d = 150 - 40 - 6 = 104\text{mm}$	
A still and Martin Table 12	$\frac{M}{f_{cu} b d^2} = \frac{18.41 \times 10^{-6}}{25 \times 1000 \times 104^2} = 0.07$	

The roof plan of the reservoir and the section through the reservoir are all indicated in Appendix VB.



Appendix V (B): 1200m3 Reinforced Concrete Ground Tank.

APPENDIX V C: STRUCTURAL DESIGN OF OTHER WATER RETAINING STRUCTURES

REFERENCE	CALCULATIONS	OUTPUT
Wall Data	**Rectangular Structures** Bending Moment Crack width = 0.2, f_s = 130N/mm^2 $F = \frac{1.4\gamma H^2}{2}$ **Wall Data**  h=200mm c = 40mm Φ = 12mm fs = 130 N/mm^2 $d = h - c - \frac{\phi}{2}$ = 200 - 40 – 6 = 154 mm At 2.5 metres, $F = \frac{1.4 \times 9.81 \times 2.5^2}{2}$ = 42.92KN M = $\frac{1.4\gamma H^3}{6}$ = $\frac{1.4 \times 9.81 \times 2.5^3}{6}$ = 35.77KNM	

REFERENCE	CALCULATIONS	OUTPUT
	Bending Moment consideration	
	$K = \frac{M}{f_{cu}bd^2} = \frac{35.77\times10^6}{25\times1000\times154^2} = 0.06$	
	$l_a = 0.5 + \left(0.25 - \frac{K}{0.9}\right)^{1/2}$	
	$= 0.5 + \left(0.25 + \frac{0.06}{0.9}\right)^{1/2} = 0.928$	
Mosley and Bungey Fig. 4.29 As	$A_{st} = \frac{35.77\times10^6}{0.95\times460\times0.928\times154} = 573mm^2$	
	Provide Y 12 – 150	Provide Y12-150 at inner sides of tank walls.
	Asp = $754mm^2$	
	Min. tension Reinforcement at outer side of the tanks.	
	$As_{min} = \frac{0.15}{100}\times bd$ = $0.0015 \times 10^3 \times 154$	
	= $231mm^2$	
	Provide Y12.- 300	
	<u>Serviceability Condition</u>	
	For a modular ratio of 14.8	
	$\alpha e\frac{A_s}{bd} = \frac{14.8\times754}{1000\times154} = 0.072$	
	X = 0.3d = 0.30 x 154 =46.2mm	
	$M = 0.5bxf_{cc}\left(d - \frac{x}{3}\right) = A_sf_{st}\left(d - \frac{x}{3}\right)$	
	$f_s = \frac{M}{As\left(d - \frac{x}{3}\right)} = \frac{25.55\times10^6}{754\cdot\left(154 - \frac{46.2}{3}\right)}$	
	$f_s = 244.5N/mm^2$	
	This is greater than $130N/mm^2$ allowed in Table 11.2 of Mosley and Bungey.	

REFERENCE	CALCULATIONS	OUTPUT
	Minimum $A_s = \frac{244.5}{130} \cdot 754 = 1418mm^2$	Provide Y16-125 vertical on the inner part of tank wall.
	A_{sp} Y16 – 125	
	$A_{sp} = 1610mm^2$	
	Curtailment of vertical Reinforcement.	
	At 1.5m depth,	
	$M = \frac{9.81}{6} \times 1.5^3 = 5.52KNM$	
	Assume Y16 – 250	
	$\alpha e \ \frac{As}{bd} = \frac{14.8 \times 804}{1000 \times 154} = 0.077$	
	$X \approx 0.33d = 50.82mm$	
	$f_s = \frac{5.52 \times 10^6}{804 \cdot \left(154 - \frac{50.28}{3}\right)}$	Curtail reinforcement to Y12-250 from 1.5 depth.
	$= 50.1N/mm^2 < 130N/mm^2$	
	Distribution bars	
	Provide $As = \frac{0.15}{100} \times bd$	
	$= \frac{0.15}{100} \times 1000 \times 154 = 231/mm^2$	
	Provide Y10 – 250	Provide Y12-250 transverse reinforcement on both faces wall.
	$A_{sp} = 314mm^2$	
	Thermal and Shrinkage Cracking Control	
	$r_{cru} = \frac{f_{ct}}{f_y} = \frac{1.13}{460} = 0.00285$	
	Min $A_s = r_{crit} A_c$	
	$0.0028 \times 1000 \times 200 = 570mm^2$	
Mosley and Bungey Eq 6.13	$\frac{f_{ct}}{f_b} = 0.67$	
	$f_b = \frac{f_t}{0.67} = \frac{1.31}{0.67} = 1.96N/mm^2$	

REFERENCE	CALCULATIONS	OUTPUT
	$S_{max} = \frac{f_{ct}}{f_b} \cdot \frac{\phi}{2r}$, $f_b = 1.96N/mm^2$ $= \frac{1.31 \times 12}{1.96 \times 2 \times \frac{570}{200 \times 10{,}000}}$ $W_{max} = S_{max} \frac{\alpha c}{2} T_1$ $\frac{1407 \times 10 \times 10^6 \times 25}{2} = 0.18mm$ This satisfies the 0.2m limit.	Crack width is o.k.
	Shear Stress Considerations at H = 2.0 $F = \frac{1.4\gamma H^2}{2} = \frac{1.4 \times 9.81 \times 2^2}{2} = 27.47KN$ $V = \frac{F}{A_c} = \frac{27.47 \times 10^3}{1000 \times 154} = 0.18N/mm^2$, $\frac{100A_s}{bd} = \frac{100 \times 25}{1000 \times 154} = 0.28\%$, $Vc = 0.43 + (0.51 - 0.43) \times \frac{3}{25} = 0.54N/mm^2$ $V = 0.18N/mm^2 < V_c = 0.54\ N/mm^2$	Shear stress is okay.
	Serviceability Condition $M = \frac{\gamma H^3}{6} = \frac{9.81 \times 2^3}{6} = 13.08KNm$ $f_s = \frac{M}{A_s\left(d - \frac{x}{3}\right)}$ $f_s = \frac{13.08 \times 10^6}{425_s\left(154 - \frac{18.3}{3}\right)} = 208.01N/mm^2$ $S = \frac{0.95 \times 460 \times 425}{0.45 \times 25 \times 1000} = 16.5mm$ $X = \frac{S}{0.9} = 18.3mm$	

REFERENCE	CALCULATIONS	OUTPUT
	Min $A_s = \dfrac{208.01 \times 460}{130}$ $= 736mm^2$ Provide Y12 – 150 both sides of wall $A_{sp} = 754$. $S_{max} = \dfrac{f_{ct}}{f_b} \cdot \dfrac{\phi}{2r}$ $= \dfrac{1.31 \times 12}{1.96 \times 2 \times \dfrac{754}{1000 \times 200}} = 1064mm$ $T = 25^0C$ $\alpha = 10 \times \dfrac{10^{-6}}{{}^0C}$	
Table 11.2 Mosley and Bungey	$W_{max} = S_{max} \cdot \dfrac{\alpha_c T_1}{2}$ $\dfrac{1064 \times 10^{-5} \times 25}{2} = 0.133$	

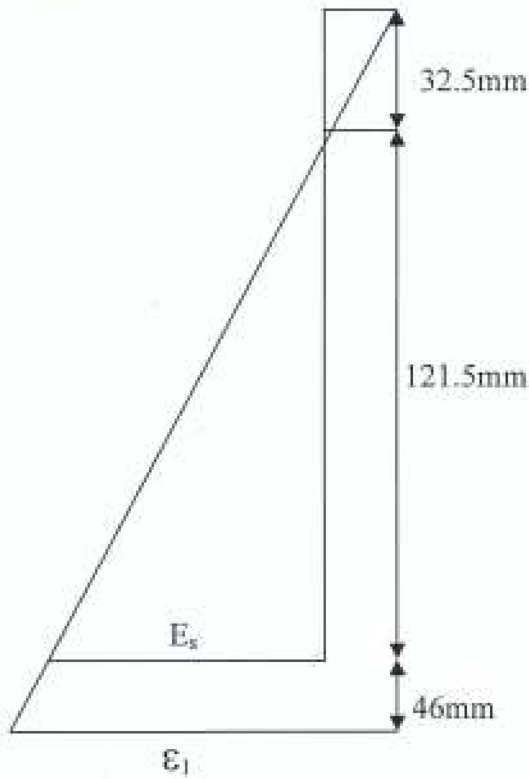
REFERENCE	CALCULATIONS	OUTPUT
	TEMPERATURE STRESSES ε_c = 0.0000 IP/^{0}C , Ec = 21000N/mm^2 ε_c E_c = 21000 x 0.00001 = 0.21 E_s = 210,000N/mm^2 K_c = 1.5 Resistance at external face a_0 = 0.09m(w)$^{-1}$ $T_0^0 = (T_0 - T_1)\frac{Kh_c}{K_c}$ $\frac{1}{k} = a_0 + \frac{h_c}{k_c}$ $T_0 = 15\left(\frac{1}{0.09 + \frac{0.20}{1.5}}\right) - \frac{0.2}{1.5} \quad = 8.96^0$ C $M_T = T^0 \varepsilon_C E_c \frac{I_c}{h_c}$ $= 8.96 \times 0.00001 \times 21000 \times \frac{200^3}{12(200)}$ = 6.3KNm/m run $\frac{M}{fcubd^2} = \frac{6.3 \times 10^6}{25 \times 1000 \times 154^2} = 0.01$ I_a = 0.95 $A_s = \frac{6.3 \times 10^6}{0.95 \times 460 \times 154 \times 0.95} = 98mm^2$ f_{ct} = $0.5T^0\varepsilon_sE_s$. = 0.5 x 8.96 x 10^{-5} x 21 x 10^3	

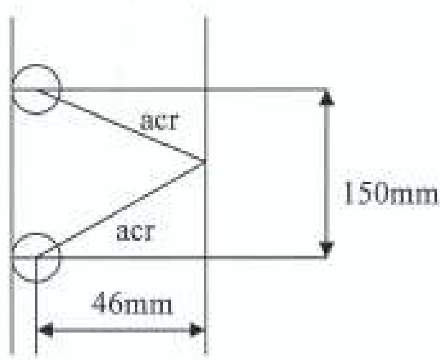
REFERENCE	CALCULATIONS	OUTPUT
	$= 0.94 N/mm^2 < V_c = 1.31 N/mm^2$	Temperature stresses o.k.
	$M_d = \frac{\gamma h^3}{6} = \frac{9.81 \times 2.5^3}{6} = 25.55 KNm$ $N_d = \frac{\gamma H^2}{2} = 9.81 \times 2.5^3 = 30.66 KN$ $\frac{M_d}{N_d} = \frac{25.55 \times 10^6}{30.66 \times 10^6} 833 mm$	
	h = 200 $d = 200 - 40 - \frac{12}{2} = 154 mm$ $d - \frac{h}{2} = 154 - \frac{200}{2} = 54 mm$	
Reynolds and Steedman Table 126	$d - \frac{h}{2} < \frac{M_d}{N_d}$ Both tension and compression occurs across the section. Modified Moment $= 25.55 \times 10^6 - 30.66(154 - 100) \times 10^6$ $= 23.89 KNM/m$	
	$h = 200$, $\Phi = 12$, $C = 40mm$ $C_w = 0.2$ $150 \longrightarrow 754$ $160 \longrightarrow x$ $\frac{150 \times 754}{160} = 707 mm^2/m$ 12mm bars at 160mm centres will suffice $\alpha_e = 14.8$	Provide Y12-160 along both faces of wall.

REFERENCE	CALCULATIONS	OUTPUT
	$\frac{\alpha_e A_s}{bd} = \frac{14.8 \times 707}{1000 \times 154} = 0.068$	
Mosley and Bungey Fig. 4.29	Assume $A_s^1 \Rightarrow Y_{12} @ 300 \Rightarrow \frac{377 \times 14.8}{154000} = 0.04$	
	$X \cong 0.28d = 0.28 \times 154 = 43.12mm$	
	Actual $f_{st} = \frac{M}{A_s\left(d - \frac{1}{3}\alpha\right)}$ f_{st} = $\underline{M_d}$	
	$= \frac{25.55 \times 10^6}{707 \times \left(15.4 - \frac{43.12}{3}\right)}$	
	Additional steel required $= \frac{30.66 \times 10^3}{0.95 \times 130 \times 154}$	
	$= 2mm^2$	
	Total area of steel reinforcement needed	
	$= 707 + 2$	
	$= 709mm^2$	
	Provide Y_{12} - 150 $A_{sp} = 754mm^2$	Provide Y12 – 150 along the faces of wall

REFERENCE	CALCULATIONS	OUTPUT

Checking crack width





$$d = 154,\ \frac{100A_s}{bd} = \frac{100 \times 754}{10^3 \times 154}$$

$$= 0.49$$

$0.45\ fcub_s = 0.95 f_y A_s$

$$S = \frac{0.95 f_y A_s}{0.45\, fcub}$$

$$S = \frac{0.95 \times 460 \times 754}{0.45 \times 25 \times 1000} \qquad = 29.3\text{mm}$$

REFERENCE	CALCULATIONS	OUTPUT
	$X = \frac{S}{0.9} \qquad = \underline{32.5\text{mm}}$ $f_s = \frac{M}{ZA_s}$ $\frac{M}{A_s\left(d - \frac{x}{3}\right)} = \frac{25.55 \times 10^6}{754\left(154 - \frac{32.5}{3}\right)}$ $= \underline{236.7\text{N/mm}^2}$ acr = $\sqrt{75^2 + 46^2 - 6^2} = 82mm$ $E_s = \frac{f_s}{E_s} = \frac{236.7}{210 \times 10^3}$ $= 0.001127$ $\varepsilon_1 = \frac{167.5}{121.5} \times 0.01127$ $= 0.00155$ $\varepsilon_1 = \varepsilon_1 - \frac{0.7bh \times 10^3}{A_s f_s}$ $= 0.00155 - \frac{0.7 \times 10^3 \times 200 \times 10^{-3}}{754 \times 236.7}$ $= 0.00078$ $W_{cr} = \frac{4.5 a_{cr} \varepsilon_m}{1 + 2.5\left(\frac{a_{cr} - c_{\min}}{h - x}\right)}$ $= \frac{4.5 \times 82 \times 0.00078}{1 + 2.5\left(\frac{82 - 40}{200 - 32.5}\right)}$ $= 0.18\text{mm} < 0.2\text{mm}$	Crack width is okay.

APENDIX VI : WATER RETICULATION DESIGN AND OUTPUT

```
Option Explicit

Const e As Double = 0.00n01, Viscocity As Double = 0.00000113  ' m^2/s
Const PI As Double = 22 / 7, Gravity As Double = 9.81, Gamma As Double = 9.81
'KN/m^3

Dim SumQ As Double, Q() As Double, NodalPress() As Double, EL As Double
Dim QP() As Double, InFlo As Double, Elevation() As Double, HGL_ As Double,
HGL() As Double
Dim HL() As Double, PressHead() As Double, Re() As Double, Const1 As Double,
Const2 As Double
Dim NN() As Integer, PipeNodes() As Integer, N As Integer, Q_ As Double, NP As
Integer
Dim NodeWithFlow() As Integer, D() As Double, F() As Double, L() As Double
Dim Counts As Integer, PipesAtNode As Integer, Skipper As Boolean, Skip As Boolean
Dim A As Double, B As Double, u As Integer, V As Integer
Dim HLp As Double, r As Double, POWER As Double
Dim Diameter As Double

Private Sub DoFileDistribution()
Open App.Path & "\Global34.rtf" For Output As #1
Open App.Path & "\Global_Input.txt" For Input As #2


Dim i, j, k, NodeLabel, No, ThisCount, PipeList(), P, PipeTreated()

Input #2, N
Input #2, NP


ReDim NodalPress(N), Q(N), PR(N, N), NN(N), PipeNodes(NP, 2), X(N, N),
NodeWithFlow(N), Elevation(N)
ReDim HGL(N), HL(N, N), PressHead(N)
ReDim D(NP), F(NP), QP(N, N), L(NP), PipeList(NP), PipeTreated(NP), Re(NP)

For i = 0 To NP - 1
Input #2, PipeNodes(i, 0), PipeNodes(i, 1), L(i)

Debug.Print PipeNodes(i, 0), PipeNodes(i, 1), L(i)

Next i


For i = 0 To N - 1
```

```
Input #2, NodeLabel, EL
Elevation(NodeLabel) = EL
HGL(NodeLabel) = 0

Debug.Print EL

Next i


Input #2, NodeLabel, HGL_
HGL(NodeLabel) = HGL_

'Debug.Print
'Debug.Print
'Debug.Print HGL_

SumQ = 0!
No = N
InFlo = 100 / 3600: InFlo = InFlo / 15387

For i = 0 To N - 1
      NodeWithFlow(i) = i + 1
      Q(NodeWithFlow(i)) = 0#

      If i = 0 Then
         Q(NodeWithFlow(i)) = 100 / 3600       '100m^3/s at the Res. Intake
      Else
         For j = 0 To NP - 1
               If PipeNodes(j, 0) = i + 1 Or PipeNodes(j, 1) = i + 1 Then
                  If j > 0 Then
                     Q(NodeWithFlow(i)) = Q(NodeWithFlow(i)) + L(j) * InFlo / 2
                  Else
                     Q(NodeWithFlow(i)) = Q(NodeWithFlow(i)) + L(j) * InFlo
                  End If
               End If
         Next j
      End If
      SumQ = SumQ + Q(NodeWithFlow(i))
Next i

ThisCount = 1
Counts = 0
For i = 0 To NP - 1: PipeTreated(i) = 1: Next i
While Counts < NP
   For i = 0 To NP - 1
      Skip = False
```

```
For j = 0 To 1
    For k = 0 To Counts - 1
        If PipeTreated(k) = i Then
            Skip = True
            Exit For
        End If
    Next k
  If Skip = False Then 'the pipe has not been treated determine the no of pipes
connected at this node
  PipesAtNode = 0
  For u = 0 To NP - 1
    If PipeNodes(u, 0) = PipeNodes(i, j) Or PipeNodes(u, 1) = PipeNodes(i, j)
Then
        PipeList(PipesAtNode) = u
        PipesAtNode = PipesAtNode + 1
    End If
  Next u
  P = PipesAtNode
  SumQ = 0
    For u = 0 To No - 1
        If PipeNodes(i, j) = NodeWithFlow(u) Then SumQ = SumQ -
Q(NodeWithFlow(u))
    Next u
  If PipesAtNode = ThisCount Then
  For u = 0 To PipesAtNode - 1
    For V = 0 To Counts
        If PipeList(u) = PipeTreated(V) Then
            For k = 0 To 1
                If PipeNodes(PipeTreated(V), k) = PipeNodes(i, j) Then
                    If k = 0 Then
                        SumQ = SumQ + QP(PipeNodes(PipeTreated(V), k),
PipeNodes(PipeTreated(V), k + 1))
                    Else
                        SumQ = SumQ + QP(PipeNodes(PipeTreated(V), k),
PipeNodes(PipeTreated(V), k - 1))
                    End If
                End If
            Next k
        P = P - 1
        End If
    Next V
  Next u

  If P = 1 Then ' only one pipe remaining at this node
  Select Case j
    Case 0
```

```
            QP(PipeNodes(i, j), PipeNodes(i, j + 1)) = -SumQ
            QP(PipeNodes(i, j + 1), PipeNodes(i, j)) = -QP(PipeNodes(i, j), PipeNodes(i, j
+ 1))
            Case 1
            QP(PipeNodes(i, j), PipeNodes(i, j - 1)) = -SumQ
            QP(PipeNodes(i, j - 1), PipeNodes(i, j)) = -QP(PipeNodes(i, j), PipeNodes(i, j -
1))

          End Select

          PipeTreated(Counts) = i
          If Counts < NP Then Counts = Counts + 1
          End If
          End If
          End If
       Next j
    Next i
    ThisCount = ThisCount + 1
    If ThisCount > 5 Then ThisCount = 1
Wend

'computed all the discharges within pipes
Close #2 'close the input file
'now Determines the diameter and the corr, fiction factor, this is a highly iterative
procedure
Design
Design2
'pressures and HGL
PressHead(NodeLabel) = HGL(NodeLabel) - Elevation(NodeLabel)
NodalPress(NodeLabel) = Gamma * PressHead(NodeLabel)
Counts = 0
While Counts < NP
For i = 0 To NP - 1
For j = 0 To 1
If HGL(PipeNodes(i, j)) > 0 Then
Select Case j
  Case 0
    HGL(PipeNodes(i, j + 1)) = HGL(PipeNodes(i, j)) - HL(PipeNodes(i, j), PipeNodes(i,
j + 1))
    PressHead(PipeNodes(i, j + 1)) = HGL(PipeNodes(i, j + 1)) - Elevation(PipeNodes(i,
j + 1))
    NodalPress(PipeNodes(i, j + 1)) = Gamma * PressHead(PipeNodes(i, j + 1))
  Case 1
    HGL(PipeNodes(i, j - 1)) = HGL(PipeNodes(i, j)) - HL(PipeNodes(i, j), PipeNodes(i,
j - 1))
```

```
      PressHead(PipeNodes(i, j - 1)) = HGL(PipeNodes(i, j - 1)) - Elevation(PipeNodes(i, j
- 1))
      NodalPress(PipeNodes(i, j - 1)) = Gamma * PressHead(PipeNodes(i, j - 1))
   End Select

   Counts = Counts + 1: Exit For
   End If
   Next j
   Next i
   Wend
'Write out Results
Print #1, "WATER DEMAND CHARACTERISTICS FOR OBA - ILE"
Print #1, "............................................"
Print #1,
Print #1, "Pipe No", "Node Fro", "Node To", "Discharge", "Diameter", "Length",
"F.Factor", "HeadLoss", "Fro Eleva", "To Eleva.", "Down Press", "Up Press"
Print #1, "    ", "    ", "    ", "Litres/s", " (mm) ", " (m) ", "  ", " (m) ", "(m) ", ", (m) ", "
(N/m^2)", "(N / m ^ 2)"
Print #1,
For i = 0 To NP - 1
Diameter = Round(D(i) * 1000, 3)
Select Case Diameter
    Case Is < 100
       Diameter = 100
    Case Is < 150
       Diameter = 150
    Case Is > 150
       Diameter = 200
    Case Is > 200
       Diameter = 250
    Case Else
       Diameter = 250
End Select

Print #1, i + 1, PipeNodes(i, 0), PipeNodes(i, 1), Round(QP(PipeNodes(i, 0), PipeNodes(i,
1)) * 1000, 2), Diameter, L(i), Round(F(i), 3), Round(HL(PipeNodes(i, 0), PipeNodes(i,
1)), 2), Elevation(PipeNodes(i, 0)), Elevation(PipeNodes(i, 1)),
Round(NodalPress(PipeNodes(i, 0)), 2), Round(NodalPress(PipeNodes(i, 1)), 2)


With RsWHassien

    .AddNew
    !pipeno = i + 1
    !Nodefrom = PipeNodes(i, 0)
    !Nodeto = PipeNodes(i, 1)
```

```
    !Discharge = Round(QP(PipeNodes(i, 0), PipeNodes(i, 1)) * 1000, 3)
    !Diameter = Diameter  'Round(D(i) * 1000, 3)
    !Length = L(i)
    !FFactor = Round(F(i), 5)
    !HeadLoss = Round(HL(PipeNodes(i, 0), PipeNodes(i, 1)), 3)
    !FromEleva = Elevation(PipeNodes(i, 0))
    !ToEleva = Elevation(PipeNodes(i, 1))
    !DownPress = Round(NodalPress(PipeNodes(i, 0)), 3)
    !UpPress = Round(NodalPress(PipeNodes(i, 1)), 3)
    .Update
End With



Next i
Close #1
End Sub

Function log10(X)
  log10 = Log(X) / Log(10)
End Function

Sub Design()
Dim M As Integer, ci4 As Double, Cons As Double, F1, D1, Sf, DW, Diff, F_, i, L_, hf

For i = 0 To NP - 1
Q_ = Abs(QP(PipeNodes(i, 0), PipeNodes(i, 1)))
L_ = L(i)
HL(PipeNodes(i, 0), PipeNodes(i, 1)) = Abs(Elevation(PipeNodes(i, 0)) -
Elevation(PipeNodes(i, 1)))
HL(PipeNodes(i, 1), PipeNodes(i, 0)) = -HL(PipeNodes(i, 0), PipeNodes(i, 1))
hf = (HL(PipeNodes(i, 0), PipeNodes(i, 1)))
If hf = 0 Then
hf = 1
End If
M = 0
ci4 = 7.3434712828 * Viscocity / Q_
Cons = 0.90031632 * Q_ * Sqr(L_ / (Gravity * hf))
DW = 0.001

Do
POWER = POW(DW, 2.5)

L2:  Sf = Cons / POWER
F_ = Sf - 1.14 + 2# * log10(e / DW + ci4 * DW * Sf)
M = M + 1
```

```
If (M Mod 2) Then
F1 = F_
D1 = DW
DW = DW + 0.0001
 GoTo L2
End If
Diff = (DW - D1) * ((F_ - F1) / F_)
DW = D1 - Diff
Loop While ((Abs(Diff) > 0.000001) And (M < 400000))

D(i) = DW
F(i) = POW(POW(DW, 2.5) / Cons, 2#)
Next i
End Sub

Function POW(X, Y) As Double
'On Error Resume Next

   POW = X ^ Y

End Function

Private Sub Command2_Click()
       DoFileDistribution
       Unload Me
       MsgBox " processing Completed "

End Sub


Public Sub Design2()
Dim i As Integer
   For i = 0 To NP - 1
      HL(PipeNodes(i, 0), PipeNodes(i, 1)) = Abs(Elevation(PipeNodes(i, 0)) -
Elevation(PipeNodes(i, 1)))
      HL(PipeNodes(i, 1), PipeNodes(i, 0)) = -HL(PipeNodes(i, 0), PipeNodes(i, 1))
      HLp = HL(PipeNodes(i, 0), PipeNodes(i, 1))

      If HLp = 0 Then
      HLp = 1
      End If
      Q_ = Abs(QP(PipeNodes(i, 0), PipeNodes(i, 1)))

      D(i) = 0.001  ' start with 1mm diameter pipe

      Const1 = 0.90031632 * Q_ * ((L(i) / (Gravity * HLp)) ^ (1 / 2))
```

```
        A = Const1 / (D(i) ^ 2.5)

        Const2 = 7.3434728 * Viscocity / Q_
        B = 1.14 - 2 * log10(e / D(i) + Const2 * D(i) * A)

        While Abs(A - B) > 0.1 And D(i) < 2
        D(i) = D(i) + 0.0001
        A = Const1 / (D(i) ^ 2.5)
        B = 1.14 - 2 * log10(e / D(i) + Const2 * D(i) * A)
        Wend

        F(i) = 1 / (A ^ 2) ' No Unit .D in Ft
    Next i

End Sub


Private Sub Command3_Click()
   Form6.Show
End Sub

Private Sub Form_Load()
Dim strSQL2 As String

WaterRetiDATA

    Set RsWHassien = New Recordset

    strSQL2 = "Delete  from whassien "
    RsWHassien.Open strSQL2, ConWater, adOpenDynamic, adLockOptimistic,
adCmdText


  RsWHassien.Open "WHassien", ConWater, adOpenDynamic, adLockOptimistic,
adCmdTable


End Sub
```

APPENDIX VI
NETWORK DESIGN COMPUTATIONS FOR OBA-ILE HOUSING ESTATE RETICULATION SYSTEM

PipeNO :	Node	Node To:	Discharge: Litres / s	Diameter: mm	Length: m	FFactor:	HeadLoss: m	FromEleva: m	ToEleva: m	DownPress: N /m^2	UpPress: N /m^2
1	1	2	27.778	100	325	0.0201	44.008	147.188	103.18	518.086	518.086
2	2	3	28.522	150	100	0.01988	11.526	103.18	91.654	518.086	518.086
3	3	4	57.262	150	65	0.01851	6.213	91.654	97.867	518.086	396.187
4	4	5	-0.099	100	110	0.04063	1.55	97.867	99.417	396.187	365.776
5	4	6	-0.248	100	275	0.03461	6.618	97.867	91.249	396.187	396.187
6	3	7	-1.182	100	75	0.02847	7.695	91.654	99.349	518.086	367.11
7	7	8	-0.429	100	225	0.032	7.481	99.349	91.868	367.11	367.11
8	8	9	-0.113	100	125	0.04157	0.214	91.868	91.654	367.11	367.11
9	7	11	-0.241	100	267	0.03522	2.818	99.349	96.531	367.11	367.11
10	2	12	-24.881	150	75	0.01927	1.092	103.18	102.088	518.086	518.086
11	12	13	-0.406	100	450	0.03218	10.5	102.088	112.588	518.086	312.076
12	12	14	-23.911	150	100	0.01945	2.127	102.088	99.961	518.086	518.086
13	14	10	-0.384	100	425	0.03268	8.093	99.961	91.868	518.086	518.086
14	14	15	-22.968	200	95	0.0194	0.061	99.961	99.9	518.086	518.086
15	15	16	-0.451	100	500	0.03173	5.384	99.9	105.284	518.086	412.452
16	15	17	-21.844	150	150	0.01966	3.506	99.9	96.394	518.086	518.086
17	17	18	-1.569	100	300	0.02682	7.635	96.394	88.759	518.086	518.086
18	18	19	-0.569	100	630	0.03092	8.144	88.759	96.903	518.086	358.3
19	18	20	-0.09	100	100	0.04284	0.201	88.759	88.558	518.086	518.086
20	17	21	-19.736	150	125	0.0198	2.247	96.394	94.147	518.086	518.086
21	21	22	-3.433	100	83	0.02419	0.746	94.147	93.401	518.086	518.086
22	22	23	-2.804	100	97	0.02484	0.868	93.401	92.533	518.086	518.086
23	23	24	-2.225	100	95	0.02566	2.093	92.533	90.44	518.086	518.086
24	24	25	-1.444	100	100	0.02725	1.882	90.44	88.558	518.086	518.086
25	21	26	-16.025	200	100	0.02001	0.267	94.147	93.88	518.086	518.086
26	26	27	-0.654	100	75	0.03099	0.117	93.88	93.763	518.086	518.086
27	27	28	-0.474	100	125	0.03166	1.321	93.763	95.084	518.086	492.168
28	28	29	-0.181	100	200	0.037	4.653	95.084	90.431	492.168	492.168
29	26	30	-15.145	100	75	0.02074	3.245	93.88	90.635	518.086	518.086
30	30	31	-0.393	100	285	0.03441	0.204	90.635	90.431	518.086	518.086
31	31	32	-0.068	100	75	0.04312	3.298	90.431	87.133	518.086	518.086
32	30	33	-14.36	150	75	0.0207	2.249	90.635	88.386	518.086	518.086

PipeNO :	Node	Node To:	Discharge:	Diameter:	Length:	FFactor:	HeadLoss:	FromEleva:	ToEleva:	DownPress:	UpPress:
			Litres / s	mm	m		m	m	m	N /m^2	N /m^2
33	22	34	-0.233	100	258	0.03515	6.046	93.401	87.355	518.086	518.086
34	23	35	-0.203	100	225	0.03558	5.939	92.533	86.594	518.086	518.086
35	24	36	-0.302	100	335	0.03368	7.314	90.44	83.126	518.086	518.086
36	25	37	-1.015	100	375	0.02845	3.643	88.558	84.915	518.086	518.086
37	37	38	-0.587	100	100	0.03068	1.059	84.915	83.856	518.086	518.086
38	38	39	-0.203	100	225	0.03655	0.995	83.856	84.851	518.086	498.564
39	38	40	-0.045	100	50	0.04645	1.638	83.856	82.218	518.086	518.086
40	33	41	-1.458	100	285	0.0272	1.248	88.386	87.138	518.086	518.086
41	41	42	-1.074	100	140	0.02819	0.721	87.138	86.417	518.086	518.086
42	42	43	-0.496	100	50	0.03206	3.698	86.417	90.115	518.086	445.531
43	43	44	-0.271	100	200	0.03475	4.227	90.115	94.342	445.531	362.597
44	44	45	-0.045	100	50	0.0457	4.254	94.342	90.088	362.597	362.597
45	42	46	-0.203	100	225	0.0358	3.671	86.417	90.088	518.086	446.061
46	33	47	-12.465	150	125	0.02065	1.031	88.386	87.355	518.086	518.086
47	47	48	-4.77	100	175	0.02335	4.229	87.355	83.126	518.086	518.086
48	48	49	-4.522	100	100	0.02328	0.648	83.126	83.774	518.086	505.372
49	49	50	-4.274	100	175	0.02349	1.556	83.774	82.218	505.372	505.372
50	50	51	-4.062	100	60	0.0239	1.503	82.218	80.715	505.372	505.372
51	51	52	-3.904	150	115	0.02454	0.058	80.715	80.773	505.372	504.234
52	52	53	-0.74	100	80	0.03002	2.998	80.773	83.771	504.234	445.413
53	53	54	-0.018	100	20	0.05427	0.892	83.771	84.663	445.413	427.912
54	53	55	-0.316	100	350	0.03391	2.539	83.771	86.31	445.413	395.598
55	52	56	-2.717	100	300	0.02494	5.537	80.773	86.31	504.234	395.598
56	56	57	-2.288	150	175	0.0265	0.082	86.31	86.392	395.598	393.989
57	57	58	-2.063	100	75	0.026	1.892	86.392	84.5	393.989	393.989
58	58	59	-0.564	100	75	0.0318	0.117	84.5	84.617	393.989	391.694
59	59	60	-0.068	100	75	0.04297	1.364	84.617	83.253	391.694	391.694
60	58	61	-1.273	100	100	0.0276	3.563	84.5	88.063	393.989	324.083
61	61	62	-0.271	100	300	0.03417	8.853	88.063	79.21	324.083	324.083
62	61	63	-0.546	100	105	0.03119	5.322	88.063	82.741	324.083	324.083
63	63	64	-0.09	100	100	0.04106	1.177	82.741	81.564	324.083	324.083
64	59	65	-0.181	100	200	0.03668	1.6	84.617	83.017	391.694	391.694
65	63	66	-0.135	100	150	0.04009	0.334	82.741	82.407	324.083	324.083
66	47	67	-7.198	150	250	0.02207	0.646	87.355	88.001	518.086	505.411
67	67	68	-0.925	100	1025	0.02997	0.824	88.001	88.825	505.411	489.244
68	67	70	-5.031	150	100	0.02317	0.183	88.001	88.184	505.411	501.821

PipeNO :	Node	Node To:	Discharge:	Diameter:	Length:	FFactor:	HeadLoss:	FromEleva:	ToEleva:	DownPress:	UpPress:
			Litres / s	mm	m		m	m	m	N /m^2	N /m^2
69	70	69	-4.106	100	125	0.024	4.178	88.184	92.362	501.821	419.848
70	69	71	-1.167	100	125	0.02805	2.846	92.362	95.208	419.848	364.01
71	71	72	-0.196	100	217	0.03643	6.756	95.208	88.452	364.01	364.01
72	71	73	-0.55	100	125	0.03102	3.275	95.208	91.933	364.01	364.01
73	73	74	-0.241	100	217	0.03534	3.22	91.933	88.713	364.01	364.01
74	74	75	-0.023	100	25	0.05459	4.253	88.713	92.966	364.01	280.566
75	70	76	-0.587	100	150	0.03096	7.024	88.184	95.208	501.821	364.01
76	76	77	-0.226	100	250	0.03513	11.842	95.208	83.366	364.01	364.01
77	69	78	-2.533	100	200	0.02529	3.814	92.362	88.548	419.848	419.848
78	78	79	-0.316	100	250	0.03338	4.22	88.548	84.328	419.848	419.848
79	79	80	-0.045	100	50	0.04581	0.962	84.328	83.366	419.848	419.848
80	78	81	-1.779	100	35	0.02623	0.333	88.548	88.881	419.848	413.315
81	81	82	-0.618	100	85	0.03067	1.524	88.881	90.405	413.315	383.414
82	82	83	-0.271	100	300	0.03436	12.397	90.405	102.802	383.414	140.185
83	81	84	-0.872	100	200	0.02913	1.498	88.881	87.383	413.315	413.315
84	84	85	-0.143	100	158	0.03827	4.565	87.383	91.948	413.315	323.75
85	84	86	-0.203	100	225	0.03627	2.128	87.383	89.511	413.315	371.564

APPENDIX VII

HIGH LIFT PUMP HEAD CALCULATION

$\mu_w = 10^{-3} Vs/m^2$

$L = 3{,}000m$

$Q = 100m^3/hr$

$Z_{res} = 147m$

$Z_0 = 81m$ (treatment area)

$Z_{res} - Z_0 = 147-81$

$= 66mm$

Roughness l $= 0.15mm$

$D = 150mm$

$$A = \frac{\pi(0.15)^2}{4}$$

$$= 0.01767m^2$$

$$V = \frac{100}{3600} \times \frac{1}{0.01767}$$

$$= 1.572m/s$$

$$R_e = \frac{VD}{V}$$

$$= \frac{1.572 \times 0.15 \times 1000}{0.001}$$

$$= 235{,}800$$

$$\frac{K}{D} = \frac{0.15}{150}$$

$$= 0.001$$

$\lambda = 0.0275$ (from Moody diagram)

$$hf = f\frac{LV^2}{D2g}$$ (neglecting all minor losses)

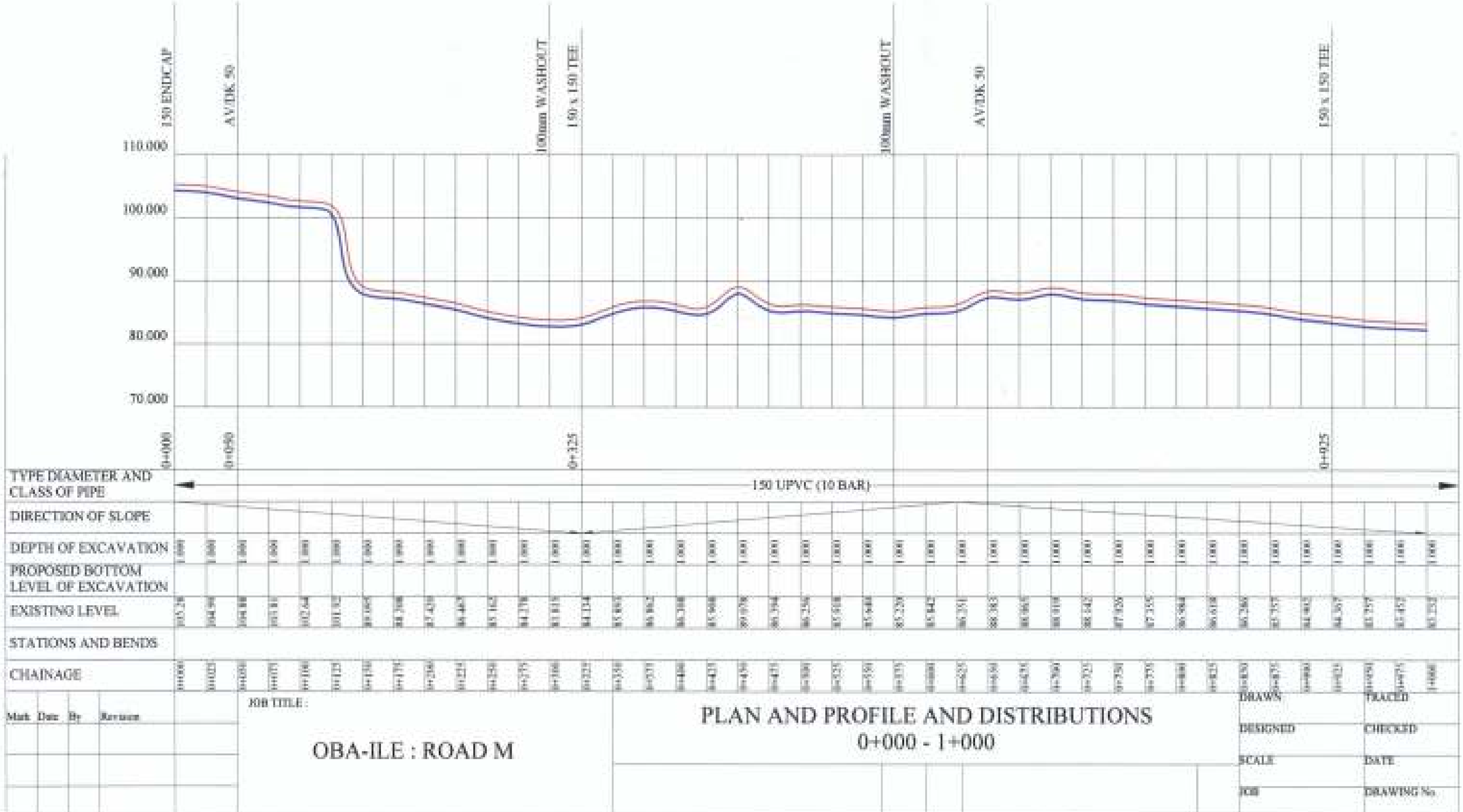
$$hf = \frac{0.0275 \times 3000 \times 1.572^2}{0.15 \times 2 \times 9.81}$$

$$= 69.27\text{m}$$

Total head required = 69.27 + 66

= 135.27m

Provide pump capable of delivering Q = 100m^3/hr and H = 150m.



Appendix VIII: Sample Pipiline Profile

APPENDIX IX: VOLUME CALCULATION FOR RAW WATER IMPOUNDMENT

T B M for project area	=	100.000m
Crest of dam	=	81.000m
Max top water level	=	80.500m
Crest of spillway	=	79.000m

At 0+000, A_1 $\frac{10}{3}$ [(0.21 + 0.27) + 2(0.31 + 0.55 + 0.75 + 0.95 + 1.25 + 1.36 + 1.38 + 1.46 + 2.226 +3.767 + 3.84 + 0.941 + 0.37 + 0.33 + 0.29) + 4(0.25 + 0.33 + 0.68 + 0.85 + 1.01 +1.34 + 1.37 + 1.40 + 1.48 + 3.132 + 4.404 + 1.949 + 0.39 + 0.35 0.31)]

$\frac{10}{3}$ [0.48 + 39.548 + 76.964]

A_1= 389.973m^2

At 0 + 025, A_2 = $\frac{10}{3}$ [(0.52 + 0.024) + 2(0.57 + 0.68 + 0.90 + 1.25 + 1.44 + 1.78 + 4.133 + 4.93 + 3.334) + 4(0.38 + 0.66 + 0.70 + 1.22 + 1.38 1.59 + 3.269 + 4.825 + 3.563 + 0.615)]

$\frac{10}{3}$ [0.544 + 38.034 + 72.808]

$\frac{10}{3}$ [111.386]

A_2=371.287m^2

At 0 + 050, A_3 = $\frac{10}{3}$ [(1.75 + 0.10) + 2(2.31+ 2.53 + 2.76+2.80+2.84 + 3.675 + 5.27 + 4.275+ 1.375) + 4(2.11 + 2.42 +2.55 + 2.78 + 2.81 + 2.965 + 4.02 + 4.93 + 3.457 + 0.20)]

$\frac{10}{3}$ [1.76 + 55.67 + 112.968]

$\frac{10}{3}$ [170.398]

A_3= 567.993m^2

At 0 + 075, A_4 = $\frac{10}{3}$ [(0.21 + 0.02) + 2 (0.24 + 0.36 + 0.68 + 0.83 + 3.139 + 3.055 + 3.386 + 0.87) + 4(0.23 + 0.25 + 0.49 + 0.81 + 1.00 + 3.0 + 3.51 + 4.126 + 1.764 + 0.10)]

$\frac{10}{3}$ [0.23 + 30.90 + 61.16]

$\frac{10}{3}$ [92.29]

$A_4 = 307.633m^2$

At 0 + 100, $A_5 = \frac{10}{3}$ [(0.42 + 0.29) + 2(0.67 + 0.82 + 1.20 + 1.46 + 1.59+2.876+3.74 + 3.147+ 2.503 + 0.70 + 0.67 0.65 + 0.49 + 0.38) + 4(0.54 + 0.71 + 0.97 + 1.32 + 1.55 + 2.76 + 2.976

2.976 + 4.347 + 2.973 + 1.035 + 0.68 + 0.66 + 0.51 + 0.41 + 0.31)]

$\frac{10}{3}$ [0.71 + 41.786 + 87.004]

$\frac{10}{3}$ [129.500]

$A_5 = 431.667m^2$

At 0 + 125 , $A_6 = \frac{10}{3}$ [(0.20 + 0.10) + 2(1.40 + 1.80 + 2.10 + 2.40 + 2.674 + 4.149 + 3.819 + 3.255 + 2.559 + 1.58 + 1.48 + 1.32 + 0.90 + 0.50) + 4(0.4 + 1.62 + 2.00 + 2.21 + 2.45 + 3.45

+ 4.134 + 4.10 + 3.214 + 1.684 + 1.56 + 1.42 + 1.10 + 0.90 + 0.22)]

$\frac{10}{3}$ [0.30 + 59.875 + 121.848]

$\frac{10}{3}$ [182.023]

$A_6 = 606.743m^3$

At 0 + 150 , $A_7 = \frac{10}{3}$ [(0.35 + 0.11) + 2(1.11 + 1.34 + 1.56 + 2.00 + 3.709 + 3.414 + 3.217 + 3.099 +

1.869 + 1.79 + 1.75 + 1.65 + 1.45) + 4(0.7 + 1.21 + 1.44 + 1.95 + 2.20 + 3.071 +

3.593 + 3.528 + 3.059 + 2.519 + 1.80 + 1.774 + 1.68 + 1.57 + 0.91)]

+ 1.50 + 1.655 + 3.081 + 3.416 + 2.895 + 3.275 + 2.541 + 1.45 + 1.22 + 1.02 + 0.61 + 0.40)]

$\frac{10}{3}$ [0.599 + 50.52 + 101.532]

$\frac{10}{3}$ [152.651]

$A_8 = 508.850m^2$

At 0 + 200, $A_9 = \frac{10}{3}$[(0.90 + 0.90) + 2(1.2 + 1.40 + 1.50 + 1.61 + 1.8 + 2.383 + 3.283+3.903+2.858 + 2.307 + 1.90 + 1.40) + 4(1.00 + 1.30 + 1.42 + 1.60 + 1.70 + 1.844 + 2.833 + 3.853 +

3.148 + 2.758 + 2.307 + 2.23 + 1.90 + 1.69 + 1.47)]

$\frac{10}{3}$ [1.8 + 51.088 + 123.932]

$\frac{10}{3}$ [176.820]

$A_9 = 589.400 m^2$

Volume of raw water when dam is full

$V = L[(A_1 + A_2)/2 + A_2 + A_3 + A_4 + A_5 + A6 + A_7 + A_8]$

$V = 25[\underline{389.973 + 589.400} + 371.287 + 567.993 + 307.663 + 431.667 + 606.743 +$
$\quad\quad 2$

601.253 + 508.850]

V = 25[489.687 + 371.287 + 567.993 + 307.633 + 431.667 + 607.734 + 601.253 +

508.850]

V = 25[3885.104]

$$V = \underline{\mathbf{97{,}127.588 m^3}}$$

APPENDIX X: EARTH DAM AND SPILLWAY DESIGN

REFERENCE	CALCULATIONS	OUTPUT
	(a) Embankment Design Freeboard = 0.5m For GC, GM, SC, SM soils as the embarkment, Front slope = 2.5 : 1 Back slope = 2.0 : 1 At 6.0m depth (overall depth up to top of riverbed) Front stretch = 2.5 x 6 = 15.0m Back stretch = 2.0 x 6 = 12.0m Clay core = 6.0m Minimum width to allow equipment = 3.6m Camber of crest = 1% A cut off trench extending to the bedrock or other impervious stratum. $W = h - d$ Where d = depth of cut off trench h = reservoir above river bed at $h = 6.0m$ $d = 1.5m$ $W = 4.5m$ **(b) Spillway Design** From the developed rating curve	

REFERENCE	CALCULATIONS	OUTPUT

$Q = 3.95\,(h - 0.2)^{1.596}$

$Q = 3.95\,(4 - 0.2)^{1.596}$

$= 33.26m^3$

$Q = \frac{2}{3}\,(2g)^{1/2}\, cd\, LH^{3/2}$

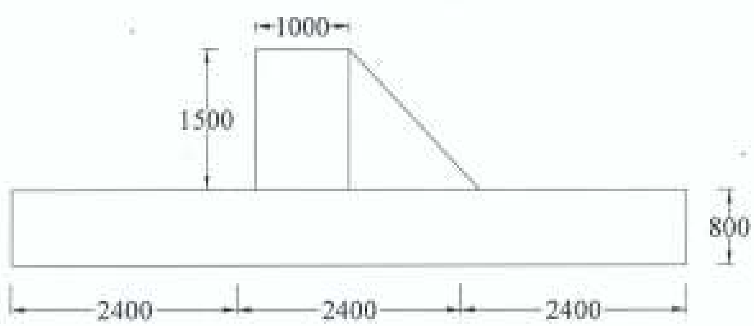
For L = 10metres

Featherstone and Nallri (1985) $H = \left(\dfrac{3Q}{2(2g)^{1/2} \cdot CdL}\right)^{2/3}$

$= \left(\dfrac{3 \times 33.26}{2(2 \times 9.81)^{1/2} \times 0.7 \times 10}\right)^{2/3}$

$= 1.37m$ Take 1.5m

(c) <u>Spill Weir</u>



$\rho = 1000Kg/m^3$

$P_{all} = 100KN/m^2$

<u>Stability</u>

$Pa = \rho gh \quad = 3.8 \times 9.8$

$= 37.27KN/m^2$

$H_f = Pah \quad = 3.8 \times 37.27$

$= 141.66KN/m$ width of wall.

REFERENCE	CALCULATIONS	OUTPUT
	Sliding: $F_s = \frac{\gamma f \lg h^2}{2}$ $= \frac{1.6 \times 9.81 \times 3.8^2}{2}$ $= 113.32kN$ **Frictional Resistance** $F_r = \mu(1.0Gk + 1.0V_k)$ $Gk_1 = 0.8 \times 7.2 \times 24 = 138.24kN$ $Gk_2 = 1.0 \times 1.5 \times 24 = 36.00kN$ $Gk_3 = \frac{1.5 \times 1.4 \times 24}{2} = 25.20kN$ $V_k = 2.4 \times 3 \times 9.81 = 70.63kN$ $F_r = 0.5(270.07) = 135.04kN/mm^2$ $F_r > F_s$	Structure is safe against sliding.
	Overturning Overturning moment $M_o = \frac{\gamma f \gamma h^3}{6}$ $\gamma f = 1.6$ $M_o = \frac{1.6 \times 9.81 \times 3.8^3}{6}$ $= \underline{143.55kNm}$ **Resisting Moment** $M_{R1} = \frac{7.2 \times 0.3 \times 24 \times 7.2}{2} = 186.62$ $M_{R2} = 1.0 \times 1.5 \times 24 \times 1.9 = 68.40$ $M_{R3} = \frac{1.4 \times 1.5 \times 24 \times 3.33}{2} = 83.92$ $M_{R4} = 2.4 \times 3.0 \times 9.81 \times 6 = \underline{423.73}$ $\underline{762.73}$	The structure is safe against overturning.

REFERENCE	CALCULATIONS	OUTPUT
	Bearing Pressures $P = \frac{N}{D} \pm \frac{6M}{D^2}$ N = 270.07kN M = $\frac{3.8 \times 2.4 \times 3 \times 9.81}{3} + 1.0 \times 1.5 \times 24$ -1.5 x 1.4 x 24 x 0.27 = $89.47 + 25.2 - 13.61$ = 101.06kNm M = 75 ± 46.78 P_{max} = 121.79 P_{min} = 28.22kN $\frac{M}{N} = \frac{101.06}{270.07} = 0.37$ $\frac{D}{6} = \frac{3.6}{6} = 0.6$ $\frac{M}{N} = \frac{D}{6}$ **Reinforcement** **Wall** M = 143.55kNm $\frac{M}{bd^2 fcu} = \frac{143.55 \times 10^6}{1000 \times 2300^2 \times 25}$ = 0.001 A_s = $\frac{143.55 \times 10^6}{0.95 \times 0.95 \times 2300 \times 410}$ = $169 mm^2/mm$	The effective eccentricity lies within the middle third.

<table>
<tr><th>REFERENCE</th><th>CALCULATIONS</th><th>OUTPUT</th></tr>
<tr><td></td><td>

Base Slab

Bearing Pressure on soil $= \dfrac{270.07}{7.2\times1.0} = 37kN/m^2$

Assume a bearing pressure of $100kN/m^2$

C = 40m, ϕ = 16mm

At the base of the wall

$M = \dfrac{100\times2.4\times1.0\times24}{2} = 288kNm$

$\dfrac{M}{fcubd^2} = \dfrac{288\times10^6}{25\times1000\times750^2}$

$= 0.02$

As $= \dfrac{288\times10^6}{0.95\times0.95\times720\times410}$

$= 1{,}081mm^2$

$A_sp = 1150mm^2$

% steel $= \dfrac{1150\times100\%}{1000\times750}$

= 0.15% Okay.

Distribution

A_s = 0.13% x bd

$= \dfrac{0.13\times1000\times750}{100}$

= 975

</td><td>

Provide
Y16-175
(Top)

Provide
Y16-175 top
and bottom
in Base
Slab.

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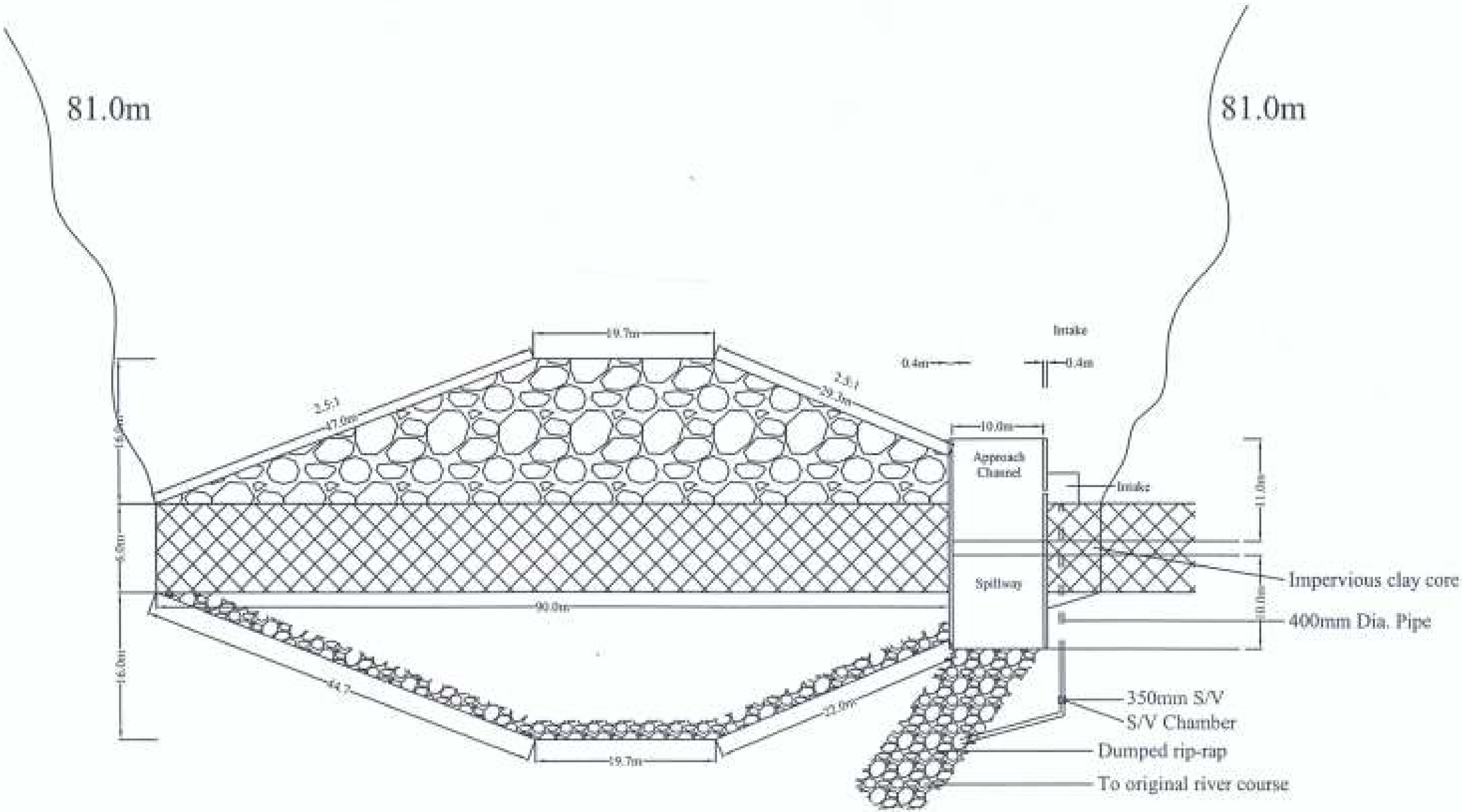
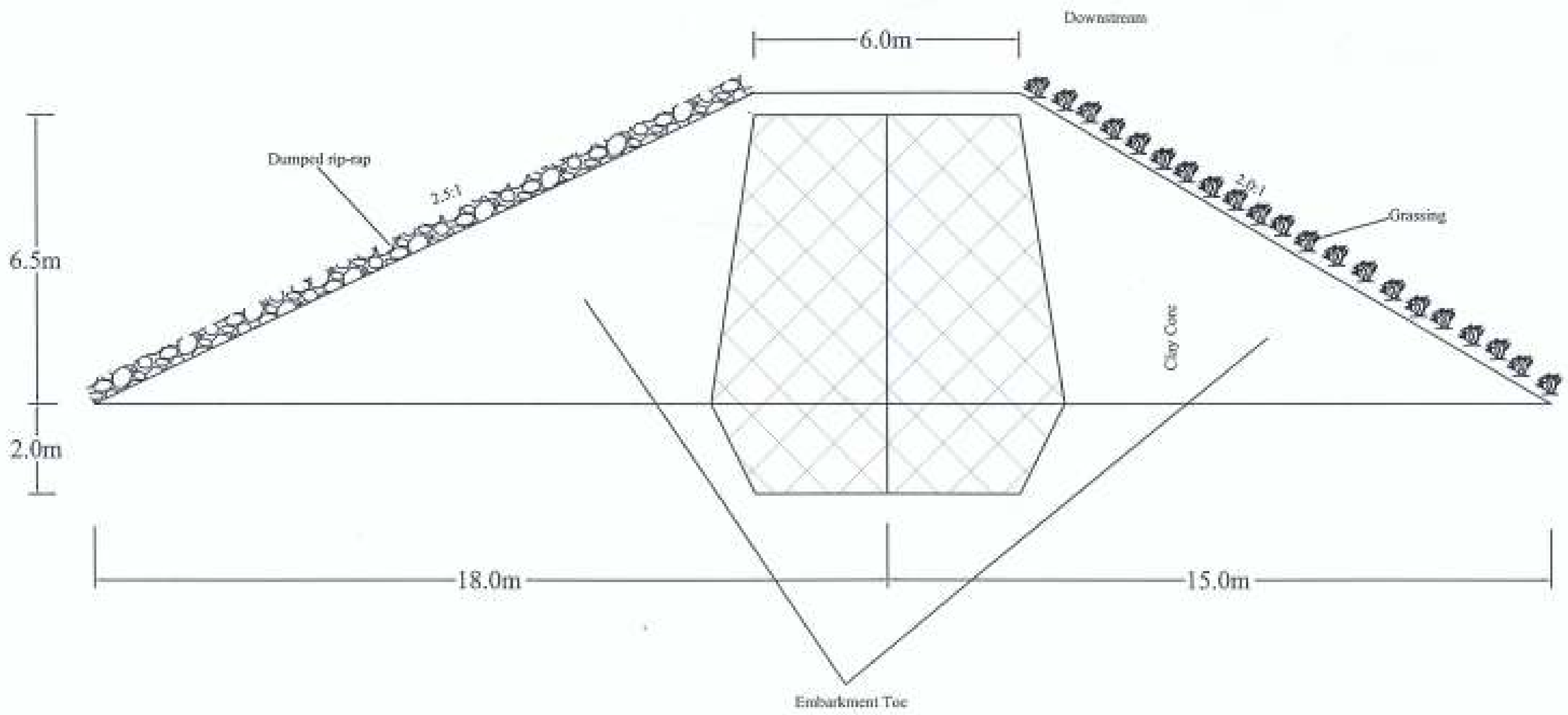
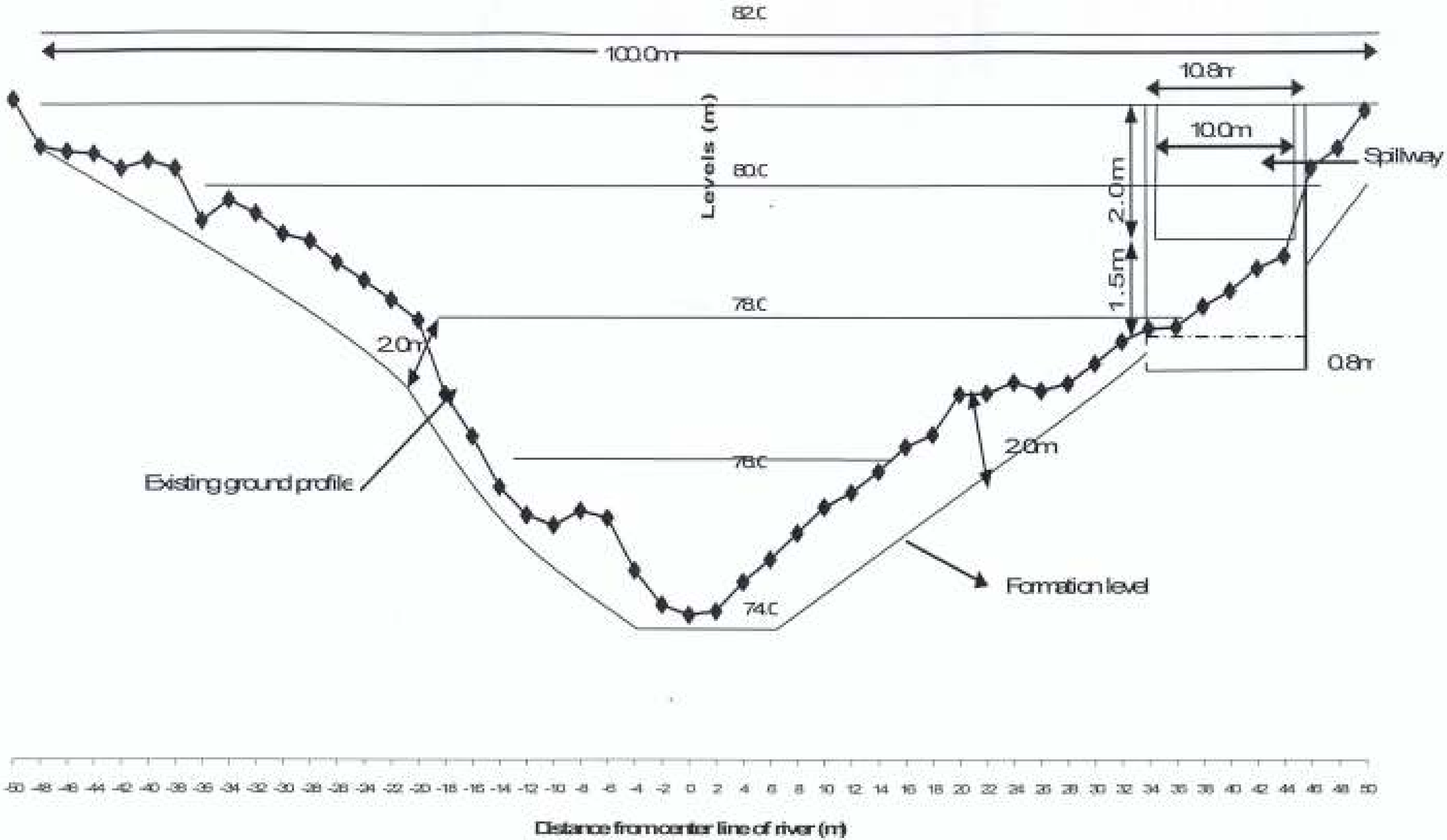


APPENDIX XIA: Layout of Earthdam



APPENDIX XI (B): LONGITUDINAL SECTION THROUGH EARTHDAM



Appendix XI (C): TRANVERSE SECTION THROUGH DAM SPILLWAY

APPENDIX XII:
BILL OF ENGINEERING MEASUREMENTS AND EVALUATION

ITEM	DESCRIPTION	UNIT	QTY	RATE N : K	AMOUNT N : K
1.0	**HEADWORKS**				
1.1	Clearing	m^2	100,000	15	1,500,000.00
1.2	Unsuitable materials	m^3	1000	600	600,000.00
1.3	Impervious clay core	m^3	1200	900	1,080,000.00
1.4	Lateritic filling in embarkment	m^3	4,500	850	3,825,000.00
1.5	Concrete grade 25	m^3	150	17000	2,550,000.00
1.6	Valves and pipes	Lump	sum		600,000.00
1.7	Dumped rip rap	m^2	450	2000	900,000.00
1.8	Grassing	m^2	450	250	112,500.00
	Headworks carried to summary.				**11,167,500.00**
2.0	**1,200m³ RESERVOIR**				
2.1	Clearing	m^2	2000	30	60,000.00
2.2	Concrete grade 25	m^3	226.94	17000	3,857,980.00
2.3	High yield reinforment	kg	19941	135	2,692,035.00
2.4	Formwork	m^2	976	600	585,600.00
	Reservoir carried to summary				**7,195,615.00**
3.0	TREATMENT UNIT				
3.1	Treatment house, treatment plant, pumping plant and generator				15,000,000.00
	Treatment unit carried to summary				**15,000,000.00**
4.0	**PIPELINES**				
4.1	Rising Mains: 150mm UPVC pipes (15 bars)	m	3000	5000	15,000,000.00
4.2	Distribution:				
4.2.1	200mm UPVC (10 bars)	m	2310	5500	12,705,000.00
4.2.2	150mm UPVC (10 bars)	m	4000	4500	18,000,000.00
4.2.3	100mm UPVC (10 bars)	m	9077	3500	31,769,500.00
	Pipelines carried to summary.				**77,474,500.00**

SUMMARY

1.0	Earth Dam	11,167,500.00
2.0	1200m^3 Ground Level Reservoir	7,195,615.00
3.0	Treatment and Pumping Plant	15,000,000.00
4.0	Pipelines	77,474,500.00
	Subtotal 1	110,837,615.00
	Add Consultancy fees (3%)	3,325,128.45
	Subtotal 3	114,162,743.45
	Add Contingencies (5%)	5,708,137.17
	Subtotal 3	119,870,880.62
	Add VAT (5%)	**5,993,544.03**
	Grand Total	**125,864,424.65**