OPTIMUM DESIGN OF WATER INTAKE TOWER FOR AN EARTH DAM

BY

SALEH, Joshua Yakubu
MEng/SEET/2017/7221

DEPARTMENT OF CIVIL ENGINEERING,
FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA

AUGUST, 2021

OPTIMUM DESIGN OF WATER INTAKE TOWER FOR AN EARTH DAM

BY

SALEH, Joshua Yakubu
MEng/SEET/2017/7221

A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF ENGINEERING IN CIVIL ENGINEERING (WATER RESOURCES AND ENVIRONMENTAL ENGINEERING)

AUGUST, 2021

ABSTRACT

Water intake towers are typically tall, hollow, reinforced concrete structures and form entrance to reservoir outlet works. It is observed that most dam intakes in Nigeria are not functional due to no way to monitor assets. In this work, the best type of tower was selected and the optimum design of the water intake tower was designed using Random Finite Element Method (RFEM) to analyses according to British standard specification (BS5059, BS8110; part 1, 2 and 3 and BS5337). The materials were utilized taking into account durability factors, actions of forces on the structure were analysed taking special attention to the geometry and type of water intake tower which was selected taking into consideration the best for a particular geometry (rectangular and circular). The design contains rectangular water intake tower which has an area of 1610 mm^2, a thermal cracking of 0.039 mm < 0.2 mm, ratio of 0.0033 < 0.0053 and a minimum steel area of 500 mm^2, the dam height of 6 m has thickness of concrete wall of 0.20 m, varied of the dam height (6 m, 10 m, 15 m and 20 m) corresponding thickness of concrete wall (0.20 m, 0.35 m, 0.50 m and 0.65 m), corresponding area of steel (1085 mm^2, 1899 mm^2, 2712 mm^2 and 3525 mm^2), corresponding uplift pressure (117.6, 196, 294 and 392) and corresponding maximum horizontal pressure (176.4, 490,1102.5 and 1960) respectively.

TABLE OF CONTENTS

Content **Page**

CHAPTER TWO

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS

BS	British Standard
CP	Code of Practices
DCP	Data Collection Platform
ERL	Empty Reservoir Level
FBD	Free Body Diagram
FRL	Full Reservoir Level
GA	Genetic Algorithm
RFEM	Random Final Element Method
RCC	Reinforced Cement Concrete
SFF	Safety Factor Fluctuation
SLS	Service Limit State
ULS	Ultimate Limit State
UK	United Kingdom

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Hydrology is a subject of great importance to human and the environment, which deals with all phases of the water on earth (Chow *et al*., 1988). Hydrology has many practical uses such as in the design and operation of hydraulic structure, water supply, wastewater, irrigation, flood control, erosion and sediment control, pollution abatement, recreational and so on (McCuen, 2017). Generally, hydrology science offers guidance for planning and management of water resources and geography principles that are important for the study of hydrology (Davie, 2002). A structure placed in a water source to permit the withdrawal of water from the source and discharge it into an intake conduit through which it flows to the treatment plant is called intake. Intakes consist of two sections First, intake conduit with the screen at the inlet end and valve to control the flow of water. Second, a structure permitting the withdrawal of water from source and housing and supporting intake conduit, valves, pumps etc. The structure may be of stone masonry or brick masonry, Reinforced cement concrete, or concrete blocks, the structure is constructed watertight and is designed to resist all forces likely to come upon it including the pressures due to water, wave action, the wind, floating debris, annual rainfall, geological formations.

It acts as an entrance structure and conducts the flood into the deviation system, also it acts as intake tower when utilization by installation and construction of ducts, valves and lower discharger. Intake towers are mostly concrete and high constructions with some valves for water discharge, most of these structures have a control room to adjust and discharge the water of reservoir for public services such as drinking water, electricity power production, helping to the reservoir discharge in case of emergency and permission to the lake water level reduction for specific inspection and maintenance. Intake towers can be located inside or at contacting with concrete dams body or outside the dams (USACE, 2003).

Intake function is to provide clam and still water, free from floating matter for water supply schemes. Its main purpose is to provide clam and still water conditions so that comparatively pure water may be conveniently collected from the source. Reservoirs are readily classified in accordance with their primary purpose for example, irrigation, water supply, hydroelectric power generation, river regulation and flood control. Dams are of numerous types, and type classification is sometimes less clearly defined, an initial broad classification into two generic groups can be made in terms of the principal construction material employed (Novak *et al.*, 2007).

Intake towers are typically tall, hollow reinforced concrete structures and form entrance to reservoir outlet works. They often house equipment for regulating the release of impounded water for vital public services such as water supply or generation of electricity, aid in emptying the reservoir in an emergency condition, and permit reservoir lowering for inspections and special repairs. Intake structures can be located within or attached to concrete gravity dams or located outside the dam (Goyal and Chopra, 1989).

Water from the rivers is always drawn from the upstream side, because it is free from the contamination caused by the disposal of sewage in it. The water enters in the lower portion of the intake known as sump-well from penstocks, the penstocks are fitted with screens to check the entry of floating solids and are placed on the downstream side so that water free from most of the suspended solids may only enter the jack-well, the water from the sump-well of the intake to upper portion of the intake. Number of penstock openings is provided in the intake tower to admit water at different levels, the opening and closing of penstock valves is done with the help of wheels provided at the pump-house floor. In such cases reservoirs are constructed by constructing weirs or dams across the rivers, the water which enters the vertical pipe is taken to the other side of the dam by means of an outlet pipe. At the top of the water intake tower sluice valves are provided to control the flow of water (Novak *et al.*, 2007).

Some rivers have too much variation in their discharge of monsoon and dry season. If in dry weather the water level falls below the lowest penstock of the intake well, a weir is constructed across the width of the river to raise the water level and maintaining some storage of water for dry period. In the case of shallow and broad rivers an approach channel is constructed, so that sufficient quantity of water may reach the intake even in dry period, this is known as wet intake tower. Another typical type of intake well, which can equally be used for collecting water from river or reservoir, it is commonly known as dry intake tower. The entry of water through the ports is controlled by the cylinder gates operated from the top, by means of wheels (Goyal and Chopra, 1989).

When there is no water inside the withdrawal conduit, the dry intake will be subjected to greater buoyancy force. Hence, the structure of this intake should be more massive than the wet intake, the water from the desired depth of the river of reservoir can be collected by opening the desired port. In case of emergency and temporary works, movable intakes can be used. In this type of intake pumping plant is installed in a carriage or trolley and the suction pipe having strainer pipe at the end is lowered in the water. The water is directly pumped from the river and sent for the treatment and distribution (Novak *et al.*, 2007).

Each design is unique and may take on many forms and variations, the intake structures can be separated into two broad categories: freestanding and inclined. Selection of the appropriate type depends on a number of considerations including site conditions, economics, and effectiveness in meeting project requirements. Project requirements can include reservoir operating range, drawdown frequency, discharge range, trash conditions and required frequency of intake cleaning, reservoir ice conditions, water quality and temperature operating requirements, and environmental requirements such as fish passage (Novak *et al.*, 2007).

Models are constructed to serve as proof of an idealized logical structure and they are an important element of methodical theories (Adem, 2005). A model is an expression to show a part of the natural or human created world which can be in the form of a physical, analog or mathematical model (Dingman, 2002). As a simple definition for models, a physical model is defined as a scaled-down form of a real system (Salarpour *et al.*, 2011). The analog model is the result of a simulated process that is used to represent a natural process. Mathematical models, on the other hand, include clear chronological set of relation, numerical and logical steps that change numerical inputs into numerical outputs. Today, mathematical models are more preferred due to the rapid development of computer technology.

1.2 Statement of the Research Problem

A dam is a structure built across a stream or river to hold water back. Dams can be used to store water, control flooding, and generate electricity. Unfortunately, they also worsen the impact of climate change. They release greenhouse gases, destroy carbon sinks in wetlands and oceans, deprive ecosystems of nutrients, destroy habitats, increase sea levels, waste water and displace poor communities. Consequently, for these reasons, priority has always been given to the construction of dams throughout last decades; many communities are in need of water during the dry season and which most of our Dams are not functional due to one reason or the other. Therefore, farmers and water resources personnel need to focus attention on the management of this vital resource by making the right choice of what and when to manage the water crisis. Different types of hydraulic gates and hoists, working on different principles and mechanism are in use for controlled release of water through spillways, sluices, intakes, regulators, ducts and tunnels. It is essential to be aware of the different factors, which would largely affect the choice of gates and intake tower and would help in the selection.

The dynamic response of an intake tower may present quite complex characteristics due to many factors. The water, inside and outside of the tower, plays an important role in the modification of the response of the structure, the behavior of the structure to the loading is very important considering the various benefits of the intake towers (Goyal and Chopra, 1989). Projecting a structure, besides the usual concerns with safety and functionality, there is also the issue of durability. The structure needs to guarantee those for a determined period of time or the project would not be economic and sustainable. Therefore, there are norms that determine some parameters that should be met so the required durability is provided. Usually the design is made taking into account a 50 year minimum lifespan for the structure, but in certain cases such as hard to repair structures or important projects, the minimum lifespan should be widened for 100 years (Mago and Chamra, 2009). Modelling and analysis of intake towers requires sound understanding of the behavior due to the complexities involved. The intake tower having axis symmetric geometry and being submerged in an unbounded reservoir necessitates a 3-dimensional model for accurate analysis. It is important to incorporate fluid-structure-interaction and the effect of unbounded reservoir condition while carrying out analysis (Oogatho, 2006).

Therefore, the need for the optimum design of an intake tower to divert flow laterally in order to prevent downstream flood and minimize erosion, others have design different towers and functional ones are few, it is the main aim of this research, the tower will be check for critical situation and the parameter will be determine, the change of height and its effects using Random Finite Element Method (RFEM) structural analysis.

1.3 Aim and Objectives of the Study

The aim of this study is for the optimum design of water intake tower, for an Earth Dam.

Objectives of the study are to:

i. identify the types of water intake tower for an earth dam;

ii. design of water intake tower using a program RFEM;

iii. determine the effect of geometry on the shape of water intake tower.

1.4 Scope of the Study

This research focuses on the hydraulic design of a water intake tower. The design is based on analyzing the flood frequency in the area and using RFEM structure analysis program to design and to estimate the best discharge for an Intake Tower. The estimated upstream discharge with other known flow parameters upstream of the dam form the basis for designing the intake tower. Model will be design to give the parameters of any intake tower given the height. The seismic action using the concept of response spectrum which provides the maximum value of ground acceleration as a function of the structure vibration period is not consider.

1.5 Justification of the Study

Today's competitive world has forced engineers to realize more economical designs and designers to develop more effective optimization techniques. This thesis work is on optimum the design of water intake tower for an Earth Dam and using RFEM structure analysis as the optimization tool. Columns are primary elements in any structure and are thus very important for the stability of any structure as they play a vital role in resisting both vertical and horizontal loads. Due to inaccuracies in loading and construction, non-homogeneity of materials, imperfect placement between beams and column, and in situations where the column is a corner one, there is always some eccentricity in the member thereby subjecting it to biaxial bending.

Genetic algorithm (GA) method determines global optimum solutions as opposed to the local solutions determined by a continuous variable optimization algorithm (Arora, 2012). The characteristics of GAs such as the ability of handling both continuous and discrete variables, not needing gradient information, and their applicability to a population of candidate solutions, make GAs popular and efficient optimization techniques (Arora, 2012).

The continuous search for optimality, the promising technique of genetic algorithm in achieving optimum solutions to structural design problems, lack of adequate research specifically in the area of optimization of reinforced concrete column as well as the challenge of becoming one of the researchers in such a tasking field motivated the thesis work.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Design Philosophy and Codes of Practice

A structure is an assembly of different members each of which is subjected to either bending or to direct force (tensile or compressive) or the combination of bending and shearing force. The shearing force and tension has primary influence on the integrity of the structure, creep and shrinkage of concrete are due to temperature change. Abrasion, vibration, frost and chemical attacks possess the possibilities of causing damage to the structure. The design process involves calculating, assessing and providing resistance against, the bending moment, the shearing force and other factors in all the members. A structure which is efficiently designed will be arranged in such a way that the weight, load and force are transmitted to the foundation by cheapest means with intended use of the structure and nature of size.

Design of structure is largely controlled by regulative codes, but the designer must be able to interpret the basic requirements. In United Kingdom (U.K.), the design of reinforced concrete is based largely on the British Standard (BS). Those for loading, Code of practice (CP) 3: Chapter v: Part 2 (1972) for wind Load and BS 6399: Part1 (1996) loading for building, structural use of concrete, BS 8110:Part1, 2 and 3 (1985), CP 110:Part1, 2 and 3, (1972). The structural use of reinforced concrete in building, BS CP 114 (1965), structural use of concrete for retaining aqueous liquid, BS 5337 (1976) and steel, BS 5400 (1988) part 2 specification for loads and part4: Design of concrete bridge. Also national building regulations (Charles and James, 1988) documents are there.

The reinforced concrete design will achieve the following objectives;

i. Under the working load, deformation of the structure does not impair the appearance, durability and performance of the structure.
ii. Structural must be safe under worst condition of loading.
iii. The structure must be economical.

For good assessment of the intending loads right materials and workmanship are requirement for good design, to ensure these, component must be test as detailed in the controlling code of practice. The determination of size of the structural member and amount of reinforce required to enable them to withstand the force (Ali *et al.*, 2007).

2.2 Design of Intake

Intake should be designed on the basis of the following considerations:-

i. Sufficient factor of safety should be taken so that intake work can resist external forces caused by heavy waves and currents, impact of floating and submerged bodies and ice pressures.
ii. Intake should have sufficient self-weight, so that it may float by the up-thrust of water and washed away by the current. To prevent floating of intake structure massive masonry work should be done and broken stones should be tilled in the bottom.
iii. If intake work is constructed in navigation channels, it should be protected by clusters of piles all around from the blows of the moving ships and steamers.
iv. The foundations of intakes should be taken sufficient deep so that they may not be undermined and current may overturn the structure.

v. To avoid the entrance of large and medium objects and fishes, screen should be provided on the inlets, sides.

vi. The inlets of intakes should be of sufficient size and allow required quantity of water to enter.

vii. The positions of inlets should be such that they can admit water in all seasons near the surface of water where quality of water is good. Number of inlets should be more so that if anyone is blocked, the water can be drawn from others. The inlets should be completely submersible so that air may not enter the suction pipe (Goyal and Chopra, 1989).

2.2.1 Classification of intake

There are different classification of intake.

i. Submerged Intake: are those intakes that were constructed entirely under water and are commonly used to obtain water from lakes.

ii. Exposed Intake: are in the form of oil or tower constructed near the bank of the river, or in some cases even away from the bank of the river. It is common due to ease of its operation.

iii. Wet Intake: In wet intake the water level of intake tower is practically the same as the water level of sources of supply. It is also known as jack well or sump well.

iv. Dry Intake: In dry intake there is no water in the water tower. Water enters through the port directly into the conveying pipes. The dry tower is simply used for the operation of valves.

2.2.2 Types of intake

Intakes are used to collect water for water works from various sources. The sources may be lakes, rivers, reservoirs or canals. The intake work for each type of source is designed separately according to its requirements and situations.

(a) Reservoir Intake: There is large variation in discharge of all the rivers during monsoon and summer. The discharge of some rivers in summer remains sufficient to meet up the demand, but some rivers dry up partly or fully and cannot meet the hot weather demand. In such cases reservoirs are constructed by constructing weirs or dams across the rivers. It essentially consists of an intake tower constructed on the slope of the dam at such place from where intake can draw sufficient quantity of water even in the driest period. Intake pipes are fixed at different level, so as to draw water near the surface in all variations of water level.

i. Wet intake: A type of intake well which is generally constructed inside the river at suitable place. This is also known as wet intake and essentially consists of a concrete circular shell filled with water up to the water level inside the river. If the elevation of the water treatment plants is lower, the water will directly flow under gravitational force through withdrawal conduit. Openings for the entrance of water is provided on the outer concrete shell as well as on the inside shell. In case the elevation of the water works is more than the elevation of top of water in the river, the water is taken to the bank of river through the withdrawal conduit in the sump well, from where it is pumped to the water works.

ii. Dry intake: A type of intake well, which can equally be used for collecting water from reservoir. It is commonly known as dry intake tower. The main differences between dry and wet intakes are that, in wet intake tower the water enters first in the outer shell and then it enters in the inner shell but in case of dry intake the water directly enters the withdrawal conduit. The entry of water through the ports is controlled by the cylinder gates operated from the top, by means of wheels (Goyal and Chopra, 1989).

2.3 Gate and Valves

The main operational requirement for tower gates and valves are the control of floods, water tightness, minimum hoist capacity, convenience of installation and maintenance and above all failure free performance and avoidance of safety hazards to the operating staff and the public. Despite robust design and precautions, faults can occur and the works must be capable of tolerating these faults without unacceptable consequences.

Gates may be classified as follows:

i. Position in the dam – crest gates and high-head (submerged) gates and valves.
ii. Function – service, bulkhead (maintenance) and emergency gates.
iii. Material – gates made of steel, aluminum alloys, reinforced concrete, wood, rubber, nylon and other synthetic materials.
iv. Pressure transmission – to piers or abutments, to the gate sill, to the sill and piers.
v. Mode of operation – regulating and non-regulating gates and valves.
vi. Type of motion – translator, rotary, rolling, floating gates, gates moving along or across the flow.

vii. Moving mechanisms – gates powered electrically, mechanically, hydraulically, automatically by water pressure or by hand.

There are, however, several outstanding examples of large span or high head gates considerably exceeding these parameters particularly in modern flood control schemes and surge barriers where special types of gates are also often used, because of the multitude of their functions and sizes there is great scope for innovation in gate and valve design both in details (example, seals, trunnions), as well as in conceptual design (automatic level control by hinged flap gates, gates and valves used in water distribution systems, ‘hydrostatic’ gates) (Alembagheri, 2016).

2.4 Design Methods

There are three main methods used in reinforced concrete design:

2.4.1 Modular ratio method

In these method loads, are assessed as working load, it limiting the permissible smashes in concrete and reinforcement to fraction at their stresses in order to provide adequate factor of safety. The method is considered as an alternative method and also known as the elastic theory method (Charles and James, 1988). It is guided by CP 114 (1965).

2.4.2 Load factor method

In this method section are analyzed at failure, actual strength of section has been related to actual load that causes failure, it is latter being determined by applying a factor to design load. The ultimate strength of materials is used in calculation, that is, there is no variation in material strength taken into account. Just because of this reason it cannot be used for serviceability limit state (Charles and James, 1988).

2.4.3 Limits state method

This method of design overcome the disadvantages of the above two methods, in this method, the working loads are multiplied by partial factors of safety and the ultimate material strengths are divided by further partial factor of safety. In this method, each member must satisfy these two separate criteria.

2.4.3.1 Ultimate limit state

It requires that a structure must be able to withstand, with an adequate factor of safety against collapse, the design of load to ensure the safety of the building occupants and structure itself (Mosley *et al.*, 2007).

2.4.3.2 Serviceability limit state

The deflection of the reinforced concrete member cannot be predicted with any certainty, this is because it makes structure unfit its intended life and also truncate the aesthetic quality of the architect. It can only preview by considering the entire significant factor that effect on deflection. Efficiency or appearance of any part of the structure must not be affected by deflection or the comfort of the building users.

a. Cracking: Prevention of excessive cracking it is the second criteria for serviceability limit state as considered in BS 8110 (1985) and CP 110 (1972) with exception that, in a particularly aggressive environment when more stringent restriction are imposed, the codec specifies that the surface width of cracks should not exceed 0.3 mm. Local damage due to cracking and stalling must not affect the appearance, efficiency or durability of the structure.

b. Durability: Achieve a successfully quality design for any structure to serve as intended life time. It must be considered in terms of proposed life of the structure and its condition exposure.

c. While design the following must put into consideration.
   i. The design should be in such that surface are freely draining.
   ii. Adequate cover must provide.
   iii. Concrete also must provide the relevant quality.
   iv. Environmental condition at the design stage should be defined.

d. Excessive vibration: It may cause discomfort or alarm as well as damage.

e. Fatigue: Must be considered if the cycle loading is likely.

f. Fire resistance: This must be considered in terms of resistance to collapse, flame penetration and heat.

g. Special circumstances: Any special requirement of the structure which is not covered by any of the more common limit states, such as earthquake, resistance must be taken into account.

The relative importance of each limit state will vary according to the nature of the structure. Generally ultimate limit state is critical for reinforced concrete although subsequent serviceability check may affect some of the details of the design (Mosley *et al.*, 2007).

2.5 Water Pressure

It is the pressure of water that acts perpendicular on the upstream face of the dam. For this, there are two cases:

i. Upstream face of the dam is vertical and there is no water on the downstream side of the dam show in Figure 2.1.

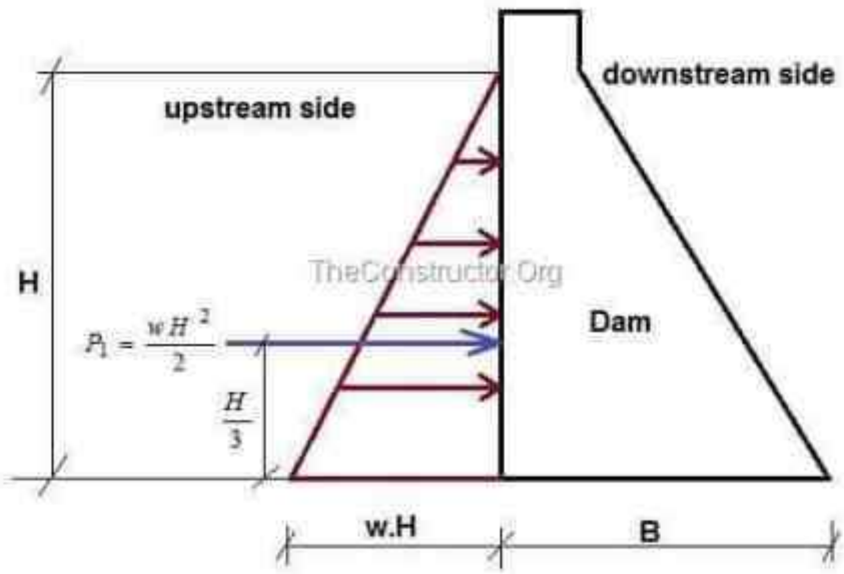


Figure 2.1: water pressure on the vertical upstream of a dam (Novak *et al.*, 2007)

Figure 2.1 show the total water pressure on the vertical upstream of a Dam is in horizontal direction and acts on the upstream face at a height H/3 from the bottom. The water pressure on the dam is computed according to equation 2.1.

$$P_1 = wH^2/2 \tag{2.1}$$

Where: w: specific weight of water. Usually it is taken as unity. H: height up to which water is stored in m.

iii. Upstream face with batter and there is no water on the downstream side as shown in Figure 2.2.

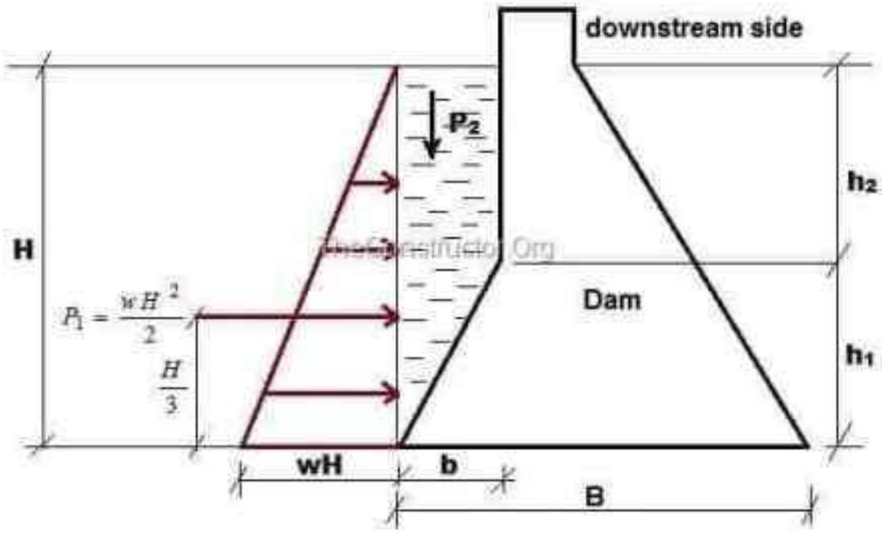


Figure 2.2: water pressure on the inclined upstream of a dam (Novak *et al.*, 2007)

Figure 2.2 shows that in addition to the horizontal water pressure of equation 2.1, there is vertical pressure of the water. It is due to the water column resting on the upstream sloping side. The vertical pressure (P_2) acts on the length (b) portion of the base. This vertical pressure is calculated as follow:

$$P_2 = (bxh_2xw) + (0.5bxh_1xw) \quad (2.2)$$

Pressure (P_2) acts through the center of gravity of the water column resting on the sloping upstream face. If there is water standing on the downstream side of the dam, water pressure will have vertical and horizontal component which can be using equation 1 except water height which is illustrated in Figure 2.3. The water pressure on the downstream face actually stabilizes the dam. Hence as an additional factor of safety, it may be neglected (Narayanan and Beedy, 2001).

2.6 Uplift Pressure or Seepage Loads

When the water is stored on the upstream side of a dam there exists a head of water equal to the height up to which the water is stored. This water enters the pores, fissures, and cracks of the foundation material under pressure, it also enters the joint between the dam and the foundation at the base and the pores of the dam itself. This water then seeps through and tries to emerge out on the downstream end, the seeping water creates hydraulic gradient between the upstream and downstream side of the dam. This hydraulic gradient causes vertical upward pressure, the upward pressure is known as uplift which is the second largest external pressure. Uplift reduces the effective weight of the structure and consequently the restoring force is reduced. Therefore, it is essential to study the nature of uplift and also some methods will have to be devised to reduce the uplift pressure value.

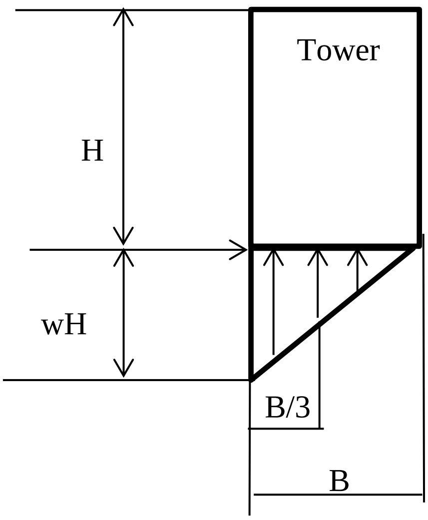


Figure 2.3: Uplift pressure (Novak *et al.*, 2007)

With reference to Figure 2.3, uplift pressure is given by

$$P_u = (wHB)/2 \tag{2.3}$$

Where P_u is the uplift pressure, B is the base width of the dam and H is the height up to which water is stored. This total uplift acts at B/3 from the heel or upstream end of the dam. Uplift is generally reduced by constructing drainage pipes between dam and its foundation, constructing cut off walls under the upstream face, holes in the dam section, or pressure grouting the Dam foundation.

2.7 Self-Weight of the Tower

The weight of dam and its foundation is a major resisting force. It can be computed using the following equation: $W = \gamma m$ Volume (2.4)

Where: γm: unit weight of dam material.

2.8 Foundation

The foundation takes the load from the column and wall, and then transfers them to the underlying soil or rock, the reason is that the soil normally much weaker than the material forming the structure, the foundation generally spread the load over a sufficient area of the soil for the stresses in the soil to be limited to levels that will not cause excessive settlement (Narayanan and Beedy, 2001).

Design load of foundation can be expressed both serviceability and ultimately limited state. Bearing capacity of the ground is expressed at serviceability state. That is, the area of foundation is required to sustain the load must be determined based on the working load. After the area and net pressure is obtained, and then expressed in the ultimate limit state. It is used for design of the foundation base (Mosley *et al.*, 2007).

2.9 Reinforce Concrete Slab

This is the thin part of a reinforced concrete floor between beams and supporting walls, the function of the slab is to transmit the loading from where it is applied to those members that are supporting slab. It is required to transfer the loads in a direction perpendicular to the direction of loading. Normally, it applied loading acting vertically because the way which gravity acts, the slabs have to transfer the load horizontally to supporting beam, wall or columns (Narayanan and Beedy, 2001).

The types of slab structure can take forms such as solid slab, ribbed floor slab, flab and waffle slab. Slab may span in one direction or two directions and it may be supported on monolithic concrete beam or steel beam or walls. The direction may depend on the ration of longer span (L_y) to the shorter span (V_x). If the ration is greater than two, then it is one-way spacing slab but if the ration is less than two, it is two-way.

For the bending moment for the slab panel are;

$$M_{sx} = \beta_{sx} N L_x^2 \quad (2.5)$$

$$M_{sy} = \beta_{sy} N L_x^2 \quad (2.6)$$

β_{sx} and β_{sy} are the bending moment co-efficient from Table 3.15 of BS 8110 part1 of (1985) for short and longer span respectively.

2.10 Roof

Roof is the topmost part of the building which protects the user from all the weather condition which is wind, rain, snow sunshine. This structural element is the most exposed element to the weather hazard compared to other structural element.

However, the capacity of the roof member to resist the unfavorable weather condition is outmost priority. Joints are assumed to be linked and external loads are applied to the joints (Panel Joints) all the members have to resist only and compression forces. In this project, the used of steel rod is considered because the durability long span, not swell and sag with, the change in the humidity and the cost of maintenance is minimized (Reynold and Steedman, 1988).

2.11 Column

Column is a structural member that post to carrying compression force in a structure. Column collects the load from the beam and slabs to transmit downward to the foundation, column is a vertical load being transferred downward (Narayanan and Beedy, 2001).

Loading and moment of column: During the analysis, it is necessary to classify column into one of the following types which are:

i. A braced column: It is where lateral loads are resisted by the shear walls or any other forms bracing.
ii. An un-braced column: The horizontal loads are resisted by the frame action of the column, beam and slabs.

For column to be short both its ration of L_{ex}/h andL_{ey}/h

a. Less than 15 for an embrace column.

b. Greater than 10 for a brace column (Chanakya, 2009).

CHAPTER THREE

3.0 RESEARCH METHODOLOGY

A water intake tower with water level of intake is practically the same as the water level of sources of supply will be investigated in this thesis. In the process of investigation, research works carried out previously and related textbooks/journals on design of wet intake tower made of reinforced concrete and other works carried out on wet intake tower generally will be reviewed. The design will be carried out to determine the effect of geometry on the design of intake towers. A computer program based on the optimization of water intake tower geometry will be done with Random Finite Element Method (RFEM) for easy practice.

3.1 Study Area

Several types of intake towers exist based on, the level of water on the tower, geometry and loading condition. This thesis will focus on the design of wet intake tower (rectangular and circular), being the cheapest and the most common in practice. Other types will not be investigated. Nigeria lies between Longitudes 2° 49'E and 14° 37'E and Latitudes 4° 16'N and 13° 52' North of the Equator. The climate is tropical, characterized by high temperatures and humidity as well as marked wet and dry seasons, though there are variations between South and North. Total rainfall decreases from the coast northwards. The South (below Latitude 8°N) has an annual rainfall ranging between 1,500 and 4,000 mm and the extreme North between 500 and 1000 mm.

The major rivers, estimated at about 10,812,400 hectares, make up about 11.5% of the total surface area of Nigeria which is estimated to be approximately 94,185,000 hectares. Thirteen lakes and reservoirs with a surface area of between 4000 ha and 550,000 ha have a total surface area of 853,600 ha and represent about one percent of the total area of Nigeria.

The water bodies are divided into saline deltas and estuaries, and freshwaters. Deltas and estuaries, with their saline wetlands have a total surface area of 858,000 ha, while freshwaters cover about 3,221,500 ha. Other water bodies, including small reservoirs, fish ponds and miscellaneous wetlands suitable for rice cultivation cover about 4,108,000 ha. Thus the total surface area of water bodies in Nigeria, excluding deltas, estuaries and miscellaneous wetlands suitable for rice cultivation-but not necessarily suitable for fish cultivation, is estimated to be about 14,991,900 ha or 149,919 km^2 and constitutes about 15.9% of the total area of Nigeria (Ali *et al.*, 2007). The Kwadna Earth dam of Federal University of Technology Minna, Niger state is where the height of the Dam is considered for this study.

3.2 Data Sources

This thesis is based majorly on secondary data as it is an analytical type of research. Textbooks, the internet, journals and contributions from supervisors are the major sources of the secondary data.

3.3 Processing of Data

The data processing stage followed the following steps: selection of the design variables, formulation of the objective function, adopting a manual approach in solving the problem, developing a computer program based on manual calculation to solve the problem and finally, comparing results obtained to previous studies.

3.3.1 Formulation of the objective function

The main goal of this dissertation is to evaluate the global and internal stability of a water intake tower in concrete. Firstly, the geometrical definition of the structure is done, not only for better comprehension of its geometry, but also for the obtainment of crucial data required for the following analysis.

Then the global safety of the structure is verified, where verifications for the fluctuation, sliding and tensions in the foundation are carefully studied, the third step consists of the verifications for the internal stability, for the evaluation of the ultimate and service limit states, in this point two procedures are presented. The first one is done using simplified analytical models which are later compared with the results of a three dimensional finite element model, which is the second procedure.

3.4 Determining the Geometric Design of the Structure

With the relevant drawings, a three dimensional model will be made in the program. Interpretations: it is important to define the main structural elements that will be subjected to an analysis, in the base of the structure there will be a footing from where the tower rises. This tower will be formed by four concrete walls, the wall with the floodgates is the front wall, which is connected to two side walls. Finally in the back there is the back wall. Inside these four walls there is the withdrawer conduit. The program will provide the volume and position of the Centre of gravity of the structure.

3.5 Materials and Durability

The definition of the materials has a straight correlation with the durability of the structure. Based on these, the materials to be selected will be M25/30 concrete for the structure, concrete M16/20 for regularization purposes and Y460 for the rebar. Figure 3.1 below shows the cross Sectional view of a water intake tower.

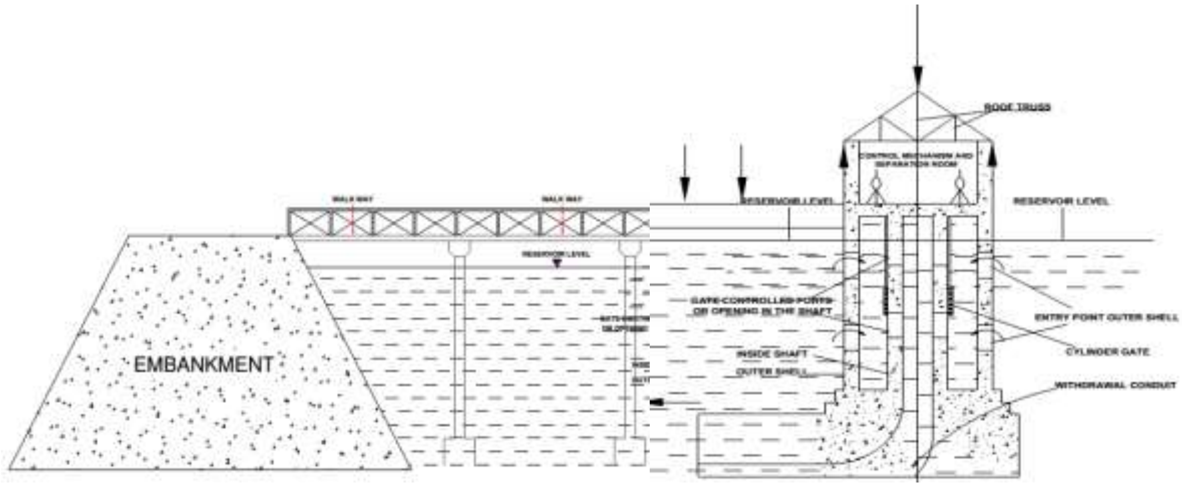


Figure 3.1: Cross sectional view of a water intake tower

3.5.1 Design scenarios and load combinations

Design Scenarios The verifications of safety are done for five different scenarios. Scenario 1 (S1) – Conditioning situation of the constructive phase correspondent to the positioning of the totality of the embankment on the back of the complete structure along with an overload on the top of the embankment.

Scenario 2 (S2) – Action of the embankment in the back and of the water at the full level of storage (both in the back and front). There is also the presence of the overload on top of the embankment and water inside the chamber.

Scenario 3 (S3) – Situation of reduction of 5.0 [m] of the water level in the front of the structure. Remainder actions according to scenario 2.

Scenario 4 (S4) – Maintenance situation, with the conditions of the scenario 2 excluding the water inside the chamber. The scenarios 1, 2, 3 and 4 are the static scenarios.

3.6 Global Stability

The evaluation of the safety factors for the fluctuation (SFF) and sliding (SFS) is performed according to a global analysis (Table 1).

Scenario SFF SFS 1, 2, 3 and 4 1, 1, 4 3 1,1 1,2

The procedure for the verification of safety consists in the calculation of coefficients which are later compared with the correspondent safety factors. For the tensions in the foundation, the following conditions have to be satisfied for the static scenarios:

$$\sigma max < \sigma adm \tag{3.1}$$

$$L\ Lcomp = 100\% \tag{3.2}$$

$$\sigma max < \frac{4}{3}\,\sigma adm \tag{3.3}$$

$$L\ Lcomp \geq 33.3\% \tag{3.4}$$

Where σmax is the calculated tension, σadm is the admissible tension (10 [MPa]), L is the length of the footing and Lcomp is the length of the footing under compression.

3.7 Internal Stability

The combination for the verification of the ultimate limit states is the following:

$$A_{d,ELU} = \Upsilon G G + \sum \Upsilon Q,\ ^{(i>1)} + \sum \Upsilon S,\ ^{(i>1)} \tag{3.5}$$

Where $A_{d,ELU}$ is the design value for the ultimate limit actions, G is the value for the permanent actions, Qi is the value for the variable actions, Si is the value for the seismic actions and ϒi is the partial coefficient relative to each action. In the ultimate limit state (ULS) it is verified the bi-axial bending with axial load and shear strength of the walls and footing. The tensions in the concrete are also limited to 0,85 fcd. The combination for the verification of the service limit states (SLS) is as follows:

$$Ad, = \Upsilon G G + \sum \Upsilon Q, i > 1 \qquad (3.6)$$

Where Ad,ELS is the design value for the service limit actions, the verifications performed are the calculation of the crack width and the tensions in the concrete (Alembagheri, 2016).

3.8 Determination of Concrete Parameter

The concrete parameter of the intake tower walls is determine from Figure 3.2

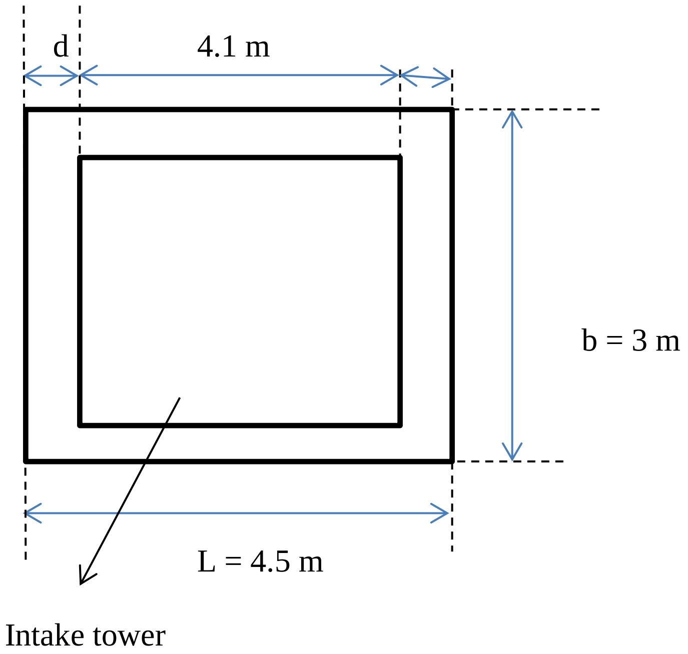


Figure 3.2: Plan view of a wet intake tower structure (Rectangular)

The pressure diagram of Figure 3.2 is show below with all the forces and Reaction on the intake tower.

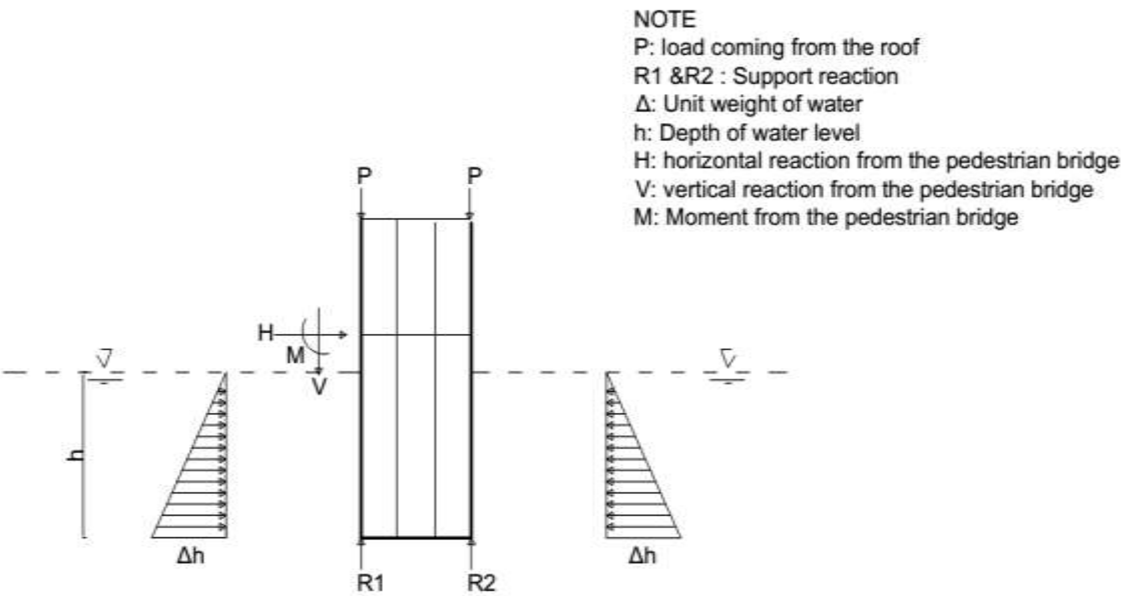


Figure 3.3: Pressure diagram on the intake tower walls

The following members have been analyzed and designed in this chapter using RFEM Structural analysis

- ✓ Intake Tower walls

The parameter details of the tower to be determine is thickness of concrete, area of steel.

3.8.1 Design data

Some of the relevant codes used in this study include: BS 5337 (1976), BS 6399: part 1 (1996) and BS 8110: part 1 (1985).

Exposure; cover; slab = 20 mm, column = 25 mm > and wall = 50 mm.

General loading condition from Figure 3.3

Live load (slab) = 5.0 kN/m^2

Wall = 3.47 kN/m^2

Roof = 1.5 kN/m^2

$K_u = 0.156$

$As = \frac{M}{0.95\, fyZ}$ $\quad$ Z = La d

$K = \frac{M}{0.95bd^2}$, $\quad$ $K \leq K_u$ otherwise

$As = \frac{(K - K_u)\, F_{cu}\, bd^2}{0.95\, fy\, (d-d')}$

$As = \frac{K_u\, F_{cu}\, bd^2}{0.95\, fy\, Z_u}$

$Z = d\,\{0.5 + \sqrt{(0.25 - K/0.9)}\} \leq 0.95$

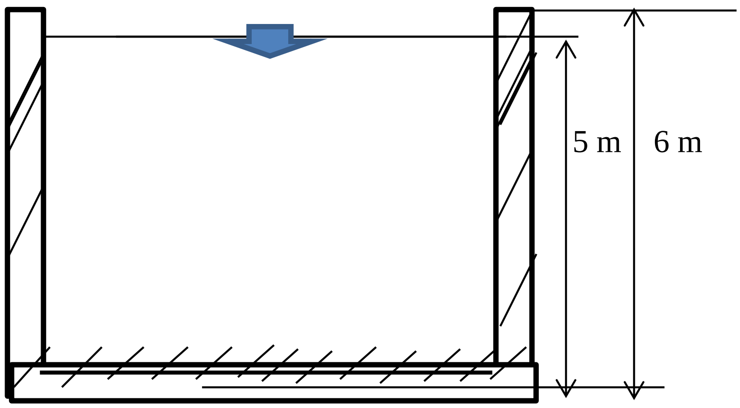
$Z_u = d\,\{0.5 + \sqrt{(0.25 - K_u/0.9)}\}$

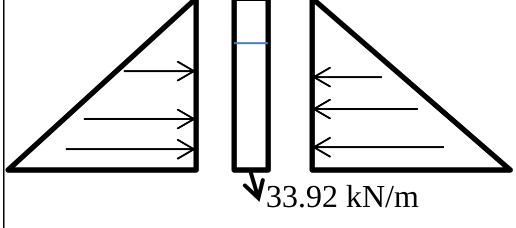
Design stresses

Concrete, $f_{cu} = 30$ N/mm^2

Steel, $f_y = 410$ N/mm^2

3.8.2 Design of wet rectangular intake tower walls

Reference	Calculation	output
	 **Figure 3.4 :** Cross sectional view of the rectangular walls From Figure 3.4, Treating the walls as cantilever walls spanning in two directions since	
BS 5337 (1976)	$\frac{x}{y}$ is small $\frac{4.5}{3} = 1.5$ Adopt wall thickness of 200 mm using wall height of 6 m: Height of pure water pressure = 6 - 0.5 = 5.5 m Height of retained water = 6 m -1 m = 5 m **Slab load** $\frac{y}{x} = \frac{4.5}{3} = 1.5$ Slab load = 0.2 x 24 = 4.80 kN/m^2	4.80 kN/m^2

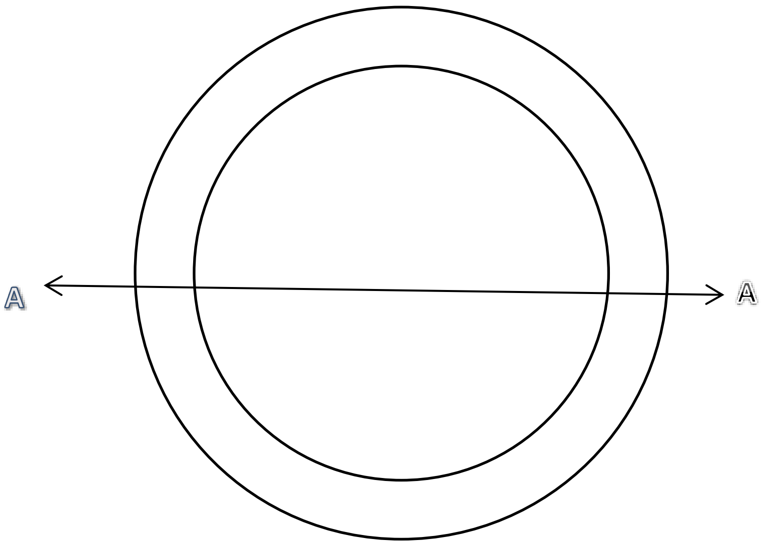
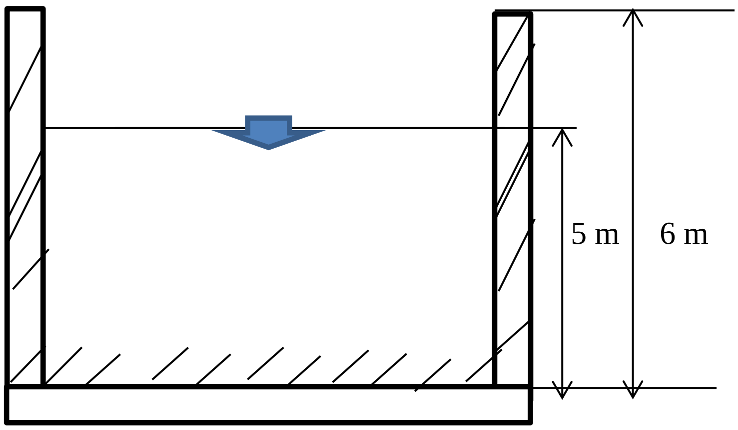
Reference	Calculation	output
	Additional load = $\frac{2.00\text{kN/m2}}{6.80\text{kN/m2}}$ Live load = 5.0 kN/m^2 Ultimate load, N = (1.4 x 6.8 + 1.6 x 5) = 17.52 kN/m^2 **Load distribution** Slab load on 4m = 0.5 x 17.52 x 3x (1-0.2) = 21.02 kN/m Slab load on 3m = 0.33 x 3 x 17.52 = 17.34 kN/m Wall load = 1.4 x 3.47 x 2.1 = 10.20 kN/m Where wall load = 3.47 kN/m^2 Wall height = 2.1 m **Roof load** Roof load = 1.5 x 1.5 kN/m^2 = 2.25 kN/m^2 = 0.5 x 2.25 x 3 x (1-0.2) = 2.7 kN/m load on wall (4 m) = 21.02 + 10.20 + 2.7 = 33.92 kN/m  **Figure 3.5:** Active Water Pressure	

Reference	Calculation	output
	Active water pressure, $p_w = 10 \times 5 = 50$ kN/m^2	
	Horizontal load = 50 x 5 x 0.2 = 50 kN/m	
	Taking moment about the Centre line of wall.	
	M = (33.92 x 4 x 0.1) + 2 [50 x (5/3 – 0.1)]	
	Note: $\frac{\text{wall thickness}}{2} = \frac{200\text{ mm}}{2} = 100$ mm	
	M = 33.92 + 2 (177.5) = 388.92 kN/m	
	h = 200 mm d = 440 mm	
BS 8110: part 1 (1985)	$k = \frac{M}{Fcubd2}$	
	Where $F_{cu} = 30$ N/mm^2	
	$K = \frac{170.17 \times 10^6}{30 \times 1000 \times 440^2} = 0.03$	
	La = 0.915	
	$As = \frac{M}{0.95 f_y Z}$	
	$As = \frac{170.17 \times 10^6}{0.95 \times 410 \times 0.915 \times 440} = 1085$ mm^2	
	$\frac{100A_s}{bd} = \frac{1210 \times 100}{1000 \times 440} = 0.28$ %	
	Check for fs to meet 'deemed to satisfy'.	
	Design concrete strength = $\frac{fw}{1.3}$	

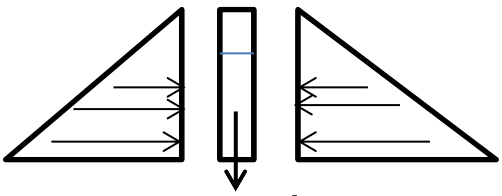
Reference	Calculation	output
	$= \frac{30}{1.3} = 23.08\ N/mm^2$	
	α_e, (modular ratio)	
	$\alpha_e, = \frac{2E}{Ec}$	
	For grade 30 concrete, $Ec = 26\ kN/mm^2$	
	$E = 200{,}000\ N/mm^2$	
	$\alpha\ e, = \frac{2 \times 200{,}000}{26 \times 1000} = 15.4$	
BS 8110 (1985)	$\frac{A_s\ \alpha_e}{bd} = \frac{2510 \times 15.4}{1000 \times 440} = 0.088$	
	$x = 0.34d$	
	$x = 149.6$ mm	
	$f_s =$ steel service stress	
	$f_s = \frac{M}{A_s\ (d - \chi/3)}$	
	$fs = \frac{170.17 \times 10^6}{\left[1210\ \left(440 - \frac{149.6}{3}\right)\right]} = 364.49\ N/mm^2$	
	Check for Thermal Cracking	
BS 6399: part 1 (1996)	$\int = \frac{A_s}{bd} = \frac{1610}{1000 \times 440} = 0.0037$	

Reference	Calculation	output
	$\int_{max} = \dfrac{f_{ct}}{f_b} \times \dfrac{\phi}{2p}$ for deformed bars $\dfrac{f_{ct}}{f_b} = 0.8$ $\int_{max} = 0.8 \times \dfrac{20}{2 \times 0.006} = 2702$	Provide Y16 (a) 100 mm c/c (1610 mm^2)
	Maximum crack width, $W_{max} = \int_{max}\ T\ \dfrac{\alpha c}{2}$ Temperature T, = 30 ^{o}c $\alpha c = 10 \times 10^{-6}$ $W_{max} = \dfrac{2702 \times 30 \times 10 \times 10^{-6}}{2} = 0.041 < 0.2$ mm	< 0.2 mm, Satisfactory
BS 6399: part 1 (1996)	**Check for Ratio** Critical ratio, $l_{crit} = \dfrac{f_{ct}}{f_y} = \dfrac{1.3}{410} = 0.003$ $p = 0.003 < 0.006$ (Satisfactory)	
	Minimum steel 0.25% bh $= \dfrac{0.25}{100} \times 1000 \times 200 = 500$ mm^2	Provide Y16 @100 mm c/c (1610 mm^2)

3.8.3 Design of wet circular intake tower walls

Reference	Calculation	output
	 **Figure 3.6**: Plan and cross sectional view of the circular walls Diameter of wall = 4 m Section A –A  **Figure 3.7:** Cross sectional view of the rectangular walls the circular walls will be treated as a cantilever wall, which is spanning in one direction. Wall thickness of 600 mm = 0.6 m	

Reference	Calculation	output
	Maximum wall height of 6 m. Height of pore water pressure = 6 – 0.5 = 5.5 m Height of pore retained water = 6 – 1 = 5 m **Slab load** Ly/Lx = 4/4 = 1 Adopt slab depth of 200 mm = 0.2 m Slab own load = 0.2 x 24 x 1.4 = 4.80 kN/m^2 Additional Load = 2.0 kN/m^2 Total load = 6.80 kN/m^2 Live load = 5.0 kN/m^2 Ultimate load, N = (1.4 x 6.8) + (1.6 x 5) = 17.52 kN/m^2 Load distribution by diameter width of 4 m Slab load = 0.38 x 4 x 17.52 = 23.36 kN/m **Wall load** Wall height = 2.1 m Wall load = 3.47 kN/m^2 Hence, wall load = 1.4 x 3.47 x 2.1 = 10.20 kN/m	Slab load = 23.36 kN/m Wall load = 10.20 kN/m

Reference	Calculation	output
	Roof load Roof load (live + Dead) load = 1.5 x 1.5 kN/m^2 = 2.25 kN/m^2 Hence, roof load = 0.36 x 4 x 2.25 = 2.97 kN/m Thus, total load on vertical axis = 23.36 + 10.20 + 2.97 = 36.52 kN/m Considering the face of wall as rectangle  **Figure 3.8:** Active Water Pressure P_w= 10 x 5 = 50 kN/m^2 Horizontal load = load of water x Area of surface cored = 10 x Л x 5^2 x 0.1 = 78.54 kN/m Taking moment about center line of wall M = (36.52 x 4 x 0.1) + 2(196.35 x 5/3 – 0.25) = 36.52 + 556.325 M = 260.70 kNm $K = \frac{M}{F_{cu}bd^2}$	

Reference	Calculation	output
BS 8110 (1985) BS 6399: part 1 (1996)	$A_s = \frac{M}{0.95\ fyZ} =$ $A_s = 1792\ mm^2$ $\frac{100\ A_s}{bh} = \frac{100 \times 1792}{1000 \times 440} = 0.41$ checking for f_c to meet deemed to satisfy Design concrete strength = f_{cu} / 1.3 = 23.08 N/mm^2 $\alpha_e = \frac{2E}{EC} = \frac{2 \times 200{,}000}{26{,}000} = 15.4$ Modular ratio, $\alpha_e = 15.4$ Note: for grade 30 concrete, Ec = 26KN/mm^2, E = 200,000 N/mm^2 $\frac{\alpha_e A_s}{bh} = \frac{15.4 \times 3140}{1000 \times 590} = 0.082$ $\chi = 0.35d = 0.35 \times 590 = 206.5$ mm $fs = \frac{M}{As\ (d - \chi/3)}$ $fs = \frac{260.7 \times 10^6}{3140\ [\,590 - \frac{206.5}{3}\,]} = 159.3\ N/mm^2$ $F_s = 169.3\ N/mm^2$ **Check for Thermal Cracking** $\int = \frac{A_s}{bd} = \frac{1810}{1000 \times 550} = 0.0033$	Provide Y16 mm @100 mm spacing c/c (1810 mm^2) Fs = 169.3 N/mm

Reference	Calculation	output
	$\int_{max,}$ maximum spacing for thermal cracking $\int_{max} = \frac{f_{ct}}{f_b} \times \frac{\phi}{2p}$ $= \frac{0.8 \times 20}{2 \times 0.0033} = 2424$ and Maximum crack width, $W_{max} = \int_{max} T \frac{\alpha c}{2}$ $W_{max} = \frac{2424 \times 30 \times 8 \times 10^{-6}}{2} = 0.18 < 0.2$ mm	
BS 6399: part 1 (1996)	**Check for Ratio** Critical ratio, $l_{crit} = \frac{f_{ct}}{f_y} = \frac{1.3}{410} = 0.003$ $p = 0.0033 < 0.0053$ (Satisfactory)	$p = 0.0033 < 0.0053$ (Satisfactory)
	Minimum steel 0.25% bh $= \frac{0.25}{100} \times 1000 \times 200 = 500$ mm^2 Since provide As = 1310 mm^2 > 500 mm^2 Thus, provide Y16 @ 100 mm c/c 1610 mm^2 at F.F and N.F	Provide As = 1310 mm^2 > 500 mm^2 Minimum steel adequate

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Pressure Distributions in Tower

(a) Full Reservoir level (FRL); It is the level corresponding to the storage which includes both inactive and active storages and also the flood storage, if provided for. In fact, this is the highest reservoir level that can be maintained without spillway discharge or without passing water downstream through sluice ways (Appendix A). The height of dam is 6 m and varied (10 m, 15 m, 20 m) having corresponding thickness of wall, area of steel, uplift pressure and maximum horizontal height, seen in Figure 4.1.

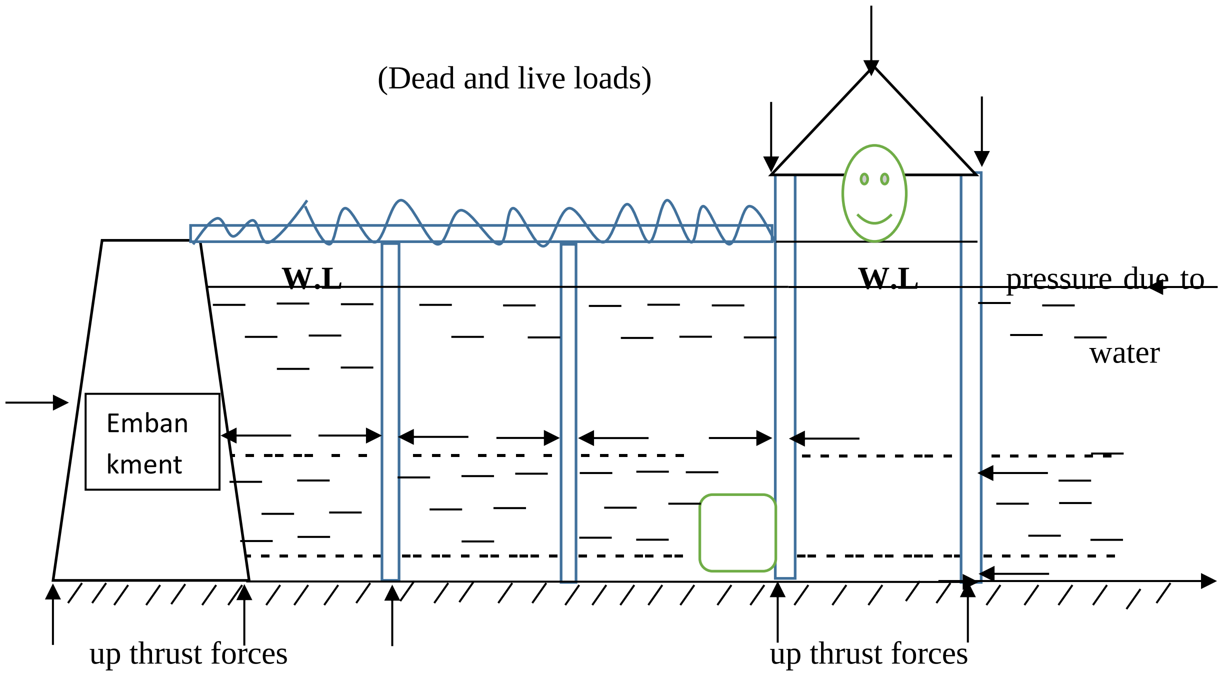


Figure 4.1: Cross section of an earth dam and the forces acting on the full reservoir

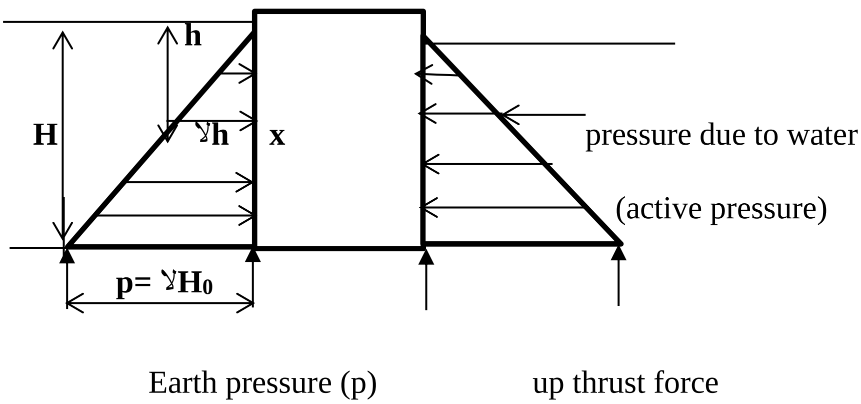


Figure 4.2: Free body diagram of pressure distribution in full reservoir tower gate

From Figure 4.2 the triangular part of the Reservoir show the pressure distribution on the tower due to the height of water, the terms g denotes the unit weight of water, x is the depth to the natural axis and P denotes the pressure on the tower gate.

(b) Empty Reservoir Level (ERL): It is the level corresponding to the flow which makes it inactive and also no water storage, the water enter directly into the withdrawer conduit causing the structure to be under greater forces.

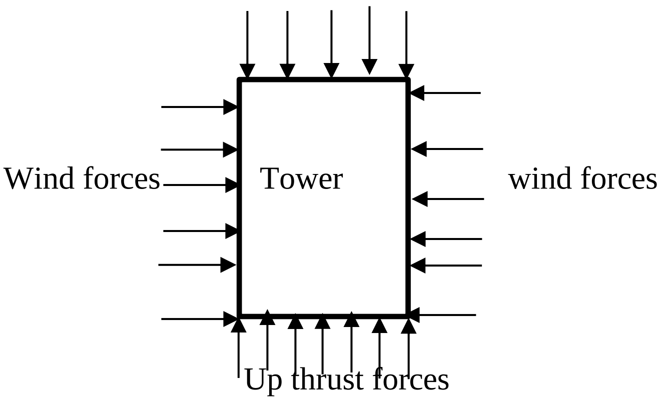
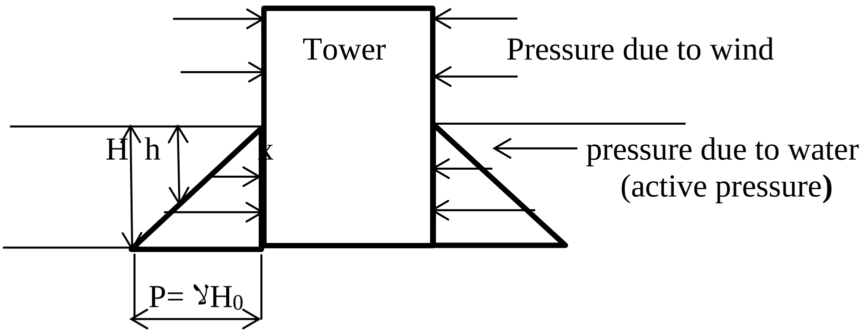


Figure 4.3: Free body diagram of pressure distribution in an empty reservoir

Figure 4.3 is subjected to greater buoyancy force, it is more critical; the structure should be more massive than the full reservoir.

(c) Mid - level Reservoir (FBD)



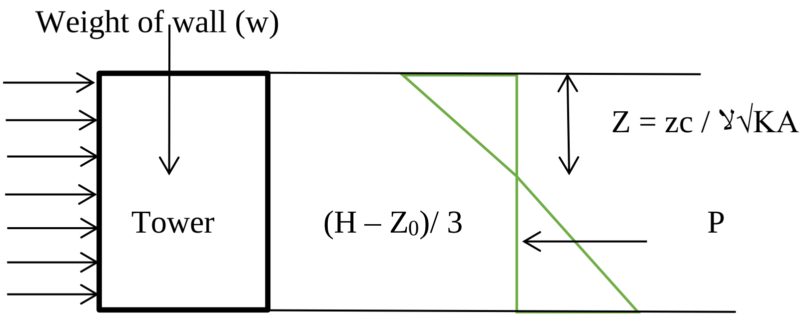


Figure 4.4: Free body diagram of pressure distribution in mid-level reservoir

From Figure 4.4 the triangular part of the Reservoir show the pressure distribution on the tower due to the height of water, the terms g denotes the unit weight of water, x is the depth to the natural axis and P denotes the pressure on the tower gate. If the wall is rigid and does not move with the pressure exerted on the wall, the water behind the wall will be in a state of elastic equilibrium.

4.2 Pressure Distribution Relationship

From Figure 3.4 the designs of rectangular intake tower has a calculated area of 1610mm^2 , a thermal cracking of 0.039mm < 0.2mm, Ratio (ק) of 0.0033 < 0.0053 and a min2imum steel area of 500mm^2. The calculated area of the column is 1625mm^2. This is satisfactory following the BS codes. This can be seen in appendix B to I.

Table 4.1: Combined result and discussion

	Intake tower wall		**Volume of concrete**	**Remarks**
Rectangular intake tower	Area(A_S)mm^2 provided	1610		
	Thermal cracking (mm)	0.1993< 0.2		
	Ratio(Ꝑ)	0.003< 0.006	81.9m^3	Satisfactory
	Minimum steel (mm^2)	500		
	Output	Provide Y16@ 125mm c/c		
Circular intake tower	Area(A_S)mm^2 provided	1810		
	Thermal cracking (mm)	0.18< 0.2		
	Ratio(Ꝑ)	0.003< 0.0053	87.91m^3	Satisfactory
	Minimum steel (mm^2)	713		
	Output	provide Y16@ 100mm c/c		

From Table 4.1, due to the hardness and thickness of dam and the enclosure of the tower with the reservoir, the rectangular tower responses are far less than the circular tower response. Also, changes of the tower responses are more obvious on rectangular than circular tower. It is concluded from above design that stresses increase in rectangular and circular tower in reservoirs with more stiff sediments. The reflective waves are more and they influence dam in one face but sediment type has fewer effects on tower responses specially on displacements it can be because tower is embedded in reservoir. From the results it is known that the rectangular intake tower has less volume of concrete and steel rod to be use than the circular intake tower, which will make it less cost in construction.

Table 4.2: Effect of dam height on thickness of concrete

S/NO	Dam Height (m)	Thickness of concrete wall (m)	Area of Steel (mm^2)	Uplift pressure (P_u)	Max Horizontal pressure (P_h)
1	6	0.20	1085	117.6	176.4
2	10	0.35	1899	196	490
3	15	0.50	2712	294	1102.5
4	20	0.65	3525	392	1960

4.3 Relationship Effect of Dam Height

Table 4.2 shows that uplift pressure, base width of the dam and the height up to water level, tower responses in the above design show that the shape of the tower can disorder the ascending procedure of maximum dynamic responses on the tower. This means that the shape can reduce the effects of dynamic responses (water movement on the tower).

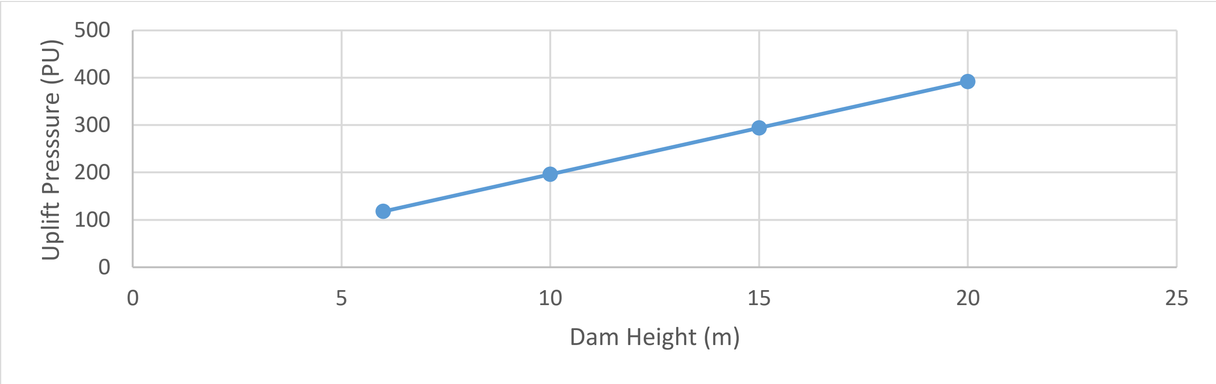


Figure 4.5: Uplift pressure against dam height

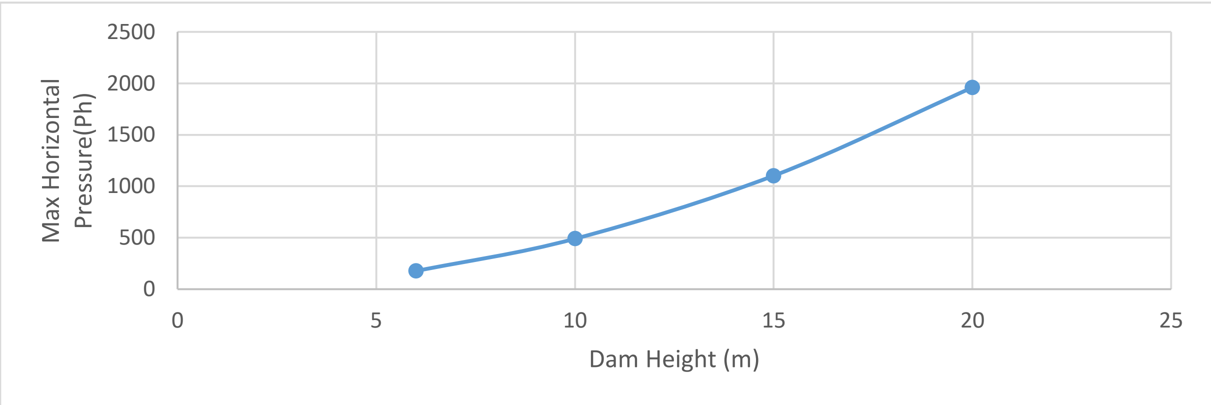


Figure 4.6: Max horizontal pressure against dam height

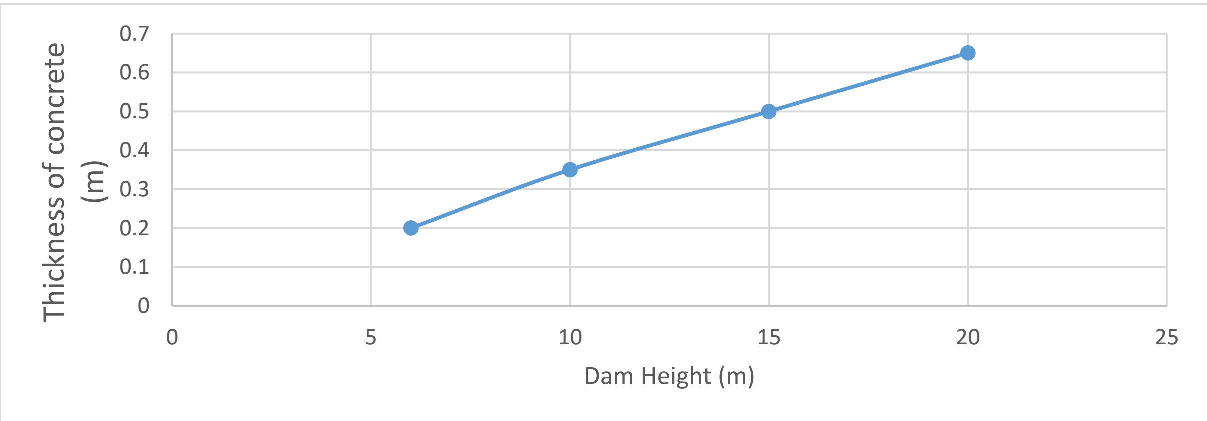


Figure 4.7: Thickness of concrete wall against dam height

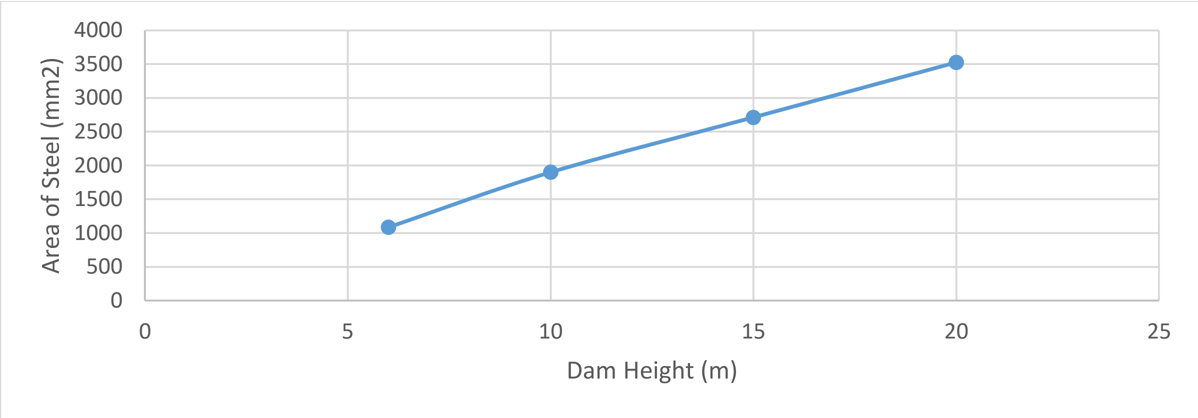


Figure 4.8: Area of steel against dam height

4.4 Discussion of Results

From Figure 4.5 shows that the uplift pressure increases as dam height also increases. The max horizontal pressure increase as the dam height increase, from figure 4.6 which can be seen at Appendix H and I. Figure 4.7 shows that as the dam height increases the thickness of the wall also increases and figure 4.8 shows that as the dam height increases the area of steel also increases (Appendix G), due to that having equilibrium balances of the tower from all forces.

The dam height of 6 m has thickness of concrete wall of 0.20 m, varied of the dam height (6 m, 10 m, 15 m and 20 m) corresponding thickness of concrete wall (0.20 m, 0.35 m, 0.50 m and 0.65 m), corresponding area of steel (1085 mm^2, 1899 mm^2, 2712 mm^2 and 3525 mm^2), corresponding uplift pressure (117.6, 196, 294 and 392) and corresponding maximum horizontal pressure (176.4, 490, 1102.5 and 1960) respectively.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Dam-reservoir-intake tower systems with interior water of tower are analyzed considering displacements of dam and tower crest, pressure and principal stresses of the tower have been extracted and results define in the following.

i. Intake tower system with interior water of tower is less critical and economical in construction (wet intake tower).
ii. The Random Finite Element Method (RFEM) structural analysis gives a great analysis on the geometry for the Water intake tower.
iii. The pressure on the geometry of rectangular tower gives it more stability and is economical in construction.

5.2 Recommendations

Based on this study the following recommendations were made:

i. It would be beneficial if more is study on the seismic effects on towers with different geometries.
ii. It would be interesting if more studies were made on the dynamic behaviour of reinforced concrete towers.

5.3 Contribution to Knowledge

Wet Intake tower is better than dry intake tower, the geometry of the tower shape, height of (6 m,10 m, 15 m and 20 m) corresponding area of steel (1085 mm2 ,1899 mm2, 2712 mm2 and 3525 mm2) and corresponding uplift pressure (117.6, 196, 294 and 392) respectively.

REFERENCES

Adem, G. (2005). *Modeling groundwater-surface water interaction by coupling MODFLOW with Wetspa*. M.Sc. Thesis, vrije university brussel, Belgium.

Alembagheri, M. (2016). Dynamics of Submerged Intake Towers Including Interaction with Dam and Foundation, *Soil Dynamics and Earthquake Engineering*, 84: 108–119.

Ali, J. D., Navayi, N. B. and Vaseghi, A. J. (2007). Dynamic Analysis of Concrete Gravity Dams with Modeling Reservoir by Lagrangian and Eulerian Methods, *Modares Civil Engineering Journal*, 41 (6): 709–724.

Arora, J. S. (2012). *Structural dynamic systems: Computational techniques and optimization* .Optimization of structures subjected to dynamic loads. The University of Lowa, lowa city, lowa 52242, USA. Gordon and breach science publishers.

British Standards Code of Practice, BS CP 114 (1965). *Structural use of reinforced concrete in Buildings*. British Standards Institution, London, UK.

British Standards Institution, BS 5337 (1976). *Steel, concrete and composite bridges specification for loads*. British Standards Institution, London, UK.

British Standards Institution, BS 5400 - 2 (2006). *Structural use of concrete*. British Standards Institution, London, UK.

British Standards Institution, BS 6399 -1 (1996). *Loading for building*. British Standards Institution, London, UK.

British Standards Institution, BS 8110 (1985). *Structural use of concrete*. British Standards Institution, London, UK.

Code of Practice, CP 110 -3 (1972). *Structural use of concrete.* Design charts for circular columns and prestressed beams. British Standards Institution, London, UK.

Chanakya, A. (2009). *Design of structural elements* (3rd ed.). Published by Taylor and Francis, 207 Madison Avenue, New York, 10016, USA.

Charles, E. R. and James, C. S. (1988). *Reinforced Concrete Designer Handbook* (10th ed.). Published by E & FN Spon.

Chow, V. T., Maidment D. R. and Mays, L. W. (1988). *Applied Hydrology*. By McGraw-Hill publisher, USA.

Davie, T. (2002). *Fundamentals of Hydrology* (1st ed.). Published by Routledge, London and New York.

Dingman, S. L. (2002). Physical Hydrology (2nd ed.). Prentice hall, New Jersey, USA.EPRI (1993). Guidelines for determining design basis ground motions, *Electric Power Research Institute*, (5): EPRI TR-102293.

Goyal, A. and Chopra, A. K. (1989). Hydrodynamic and foundation interaction effects in dynamics of intake towers. *Journal of structural engineering, 115 (6); 452-465.*

Mago, P. J. and Chamra, L. M. (2009). Analysis and optimization of CCHP systems based on energy, economical, and environmental considerations. *Energy Build*, *41*(10): 1099-106.

McCuen, R. H. (2017). *Hydrologic Analysis and design* (4th ed). Published by Pearson, University of Maryland.

Mosley, W. H. and Bungey, J. H. (2007). *Reinforce concrete design* (6th ed.). Published by Pearson, University of Maryland.

Narayanan, R. S. and Beedy, A.W. (2001). *Introduction to design for civil Engineers*. Published by Taylor and Francis, 207 Madison Avenue, New York, 10016, USA.

Novak, P., Moffat, A. I. B., Nalluri, C. and Narayanan, R. (2007). *Hydraulic structure*. Published by Taylor and Francis, 207 Madison Avenue, New York, 10016, USA.

Oogathol, S. (2006). *Runoff simulation in the canagagigue creek watershed using the mike shemodel*. M.Sc. Thesis, Mcgill University, Canada.

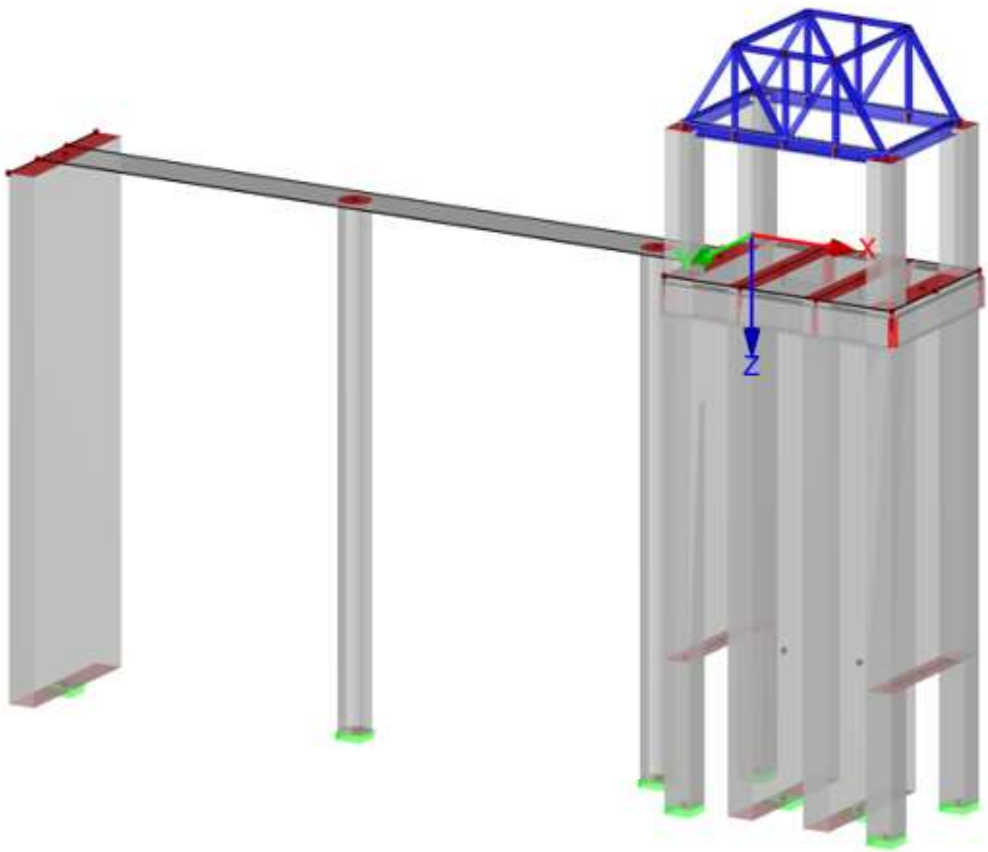
Reynold, E. C. and Steedman, C. J. (1988). *Reinforced concrete hand book* (10th ed.). Published by Van Nostrand Reinhold, 115 Fifth Avenue, New York 10003, USA.

Salarpour, M., Rahman, A. N. and Yusop, Z. (2011). Simulation of flood extent mapping by info works RS-case study for *tropical catchment. Journal software Engineering,* 5: 127-135
.

USACE (2003). United State Army Corps of Engineers. *Structural Design and Evaluation of Outlet Works*. United State, US.

APPENDICES

Appendix A: RFEM STRUCTURAL ANALYSIS SOFTWARE



Appendix B: MODEL - GENERAL DATA

General	Model name	FINALTOWER
	Project name	TOWER INTAKE
	Project description	TOWER
	Type of model	3D
	Position direction of global axis Z	Download
	Classification of load cases and	According to Standard: BS 5950
	Combinations	National Annex None
	Automatically create combinations	Load combination
Optional	RF-FORM-FINDING- Find initial equilibrium shapes of membrane and cable structures	
	RF-CUTTING-PATTERN	
	Piping analysis	
	Use COC Rule	
	Enable CAD/BIM model	
	Standard Gravity (g)	:10.00 m/s2

Appendix C: FE MESH SETTINGS

General	Target length of finite elements	I_{fe}	: 0.500 m
	Maximum distance between a node and a line to integrate it into the line		: 0.001 m
	Maximum number of mesh nodes (in thousand)		: 500
Members	Number of divisions of members with cable, elastic foundation, taper, or plastic characteristic		: 10
	Activate member divisions for lager deformation or post-critical analysis		
	Use division for members with node lying on them		
Surfaces	Maximum ratio of FE rectangle diagonals		: 1.800
	Maximum out-of-plane indination of two finite elements		: 0.50
	Shape direction of finite elements		: Triangles and quadrangles same squares where possible

Appendix D: NODES

Node No	Node Type	Reference Node	Coordinate System	Node Coordinates X (m)	Y (m)	Z (m)	Comment
1	Standard		Cartesian	0	0	0	
2	Standard		Cartesian	0	3	0	
3	Standard		Cartesian	4.5	3	0	
4	Standard		Cartesian	4.5	0	0	
5	Standard		Cartesian	4.5	3	3	
6	Standard		Cartesian	4.5	0	3	
7	Standard		Cartesian	3	3	0	
8	Standard		Cartesian	3	3	3	
9	Standard		Cartesian	1.5	3	0	
10	Standard		Cartesian	1.5	3	3	
11	Standard		Cartesian	0	3	3	
12	Standard		Cartesian	0	0	3	
13	Standard		Cartesian	1.5	0	0	
14	Standard		Cartesian	1.5	0	0	
15	Standard		Cartesian	3	0	0	
16	Standard		Cartesian	3	0	0	
17	Standard		Cartesian	0	0	3	
18	Standard		Cartesian	0	3	3	
19	Standard		Cartesian	4.5	3	0	
20	Standard		Cartesian	4.5	0	0	
21	Standard		Cartesian	3	3	3	
22	Standard		Cartesian	1.5	3	3	
23	Standard		Cartesian	1.5	0	0	
24	Standard		Cartesian	3	0	0	
25	Standard		Cartesian	0	0	3	
26	Standard		Cartesian	4.5	3	3	
27	Standard		Cartesian	4.5	3	3	
28	Standard		Cartesian	3	0	0	
29	Standard		Cartesian	1.5	0	0	
30	Standard		Cartesian	1.5	3	3	
31	Standard		Cartesian	3	3	0	
32	Standard		Cartesian	0	3	3	
33	Standard		Cartesian	0	0	3	
34	Standard		Cartesian	4.5	0	0	
35	Standard		Cartesian	4.5	3	3	
36	Standard		Cartesian	3	0	0	

Appendix E: MATERIALS

Materials No	Modulus E [kN/cm2]	Modulus G [kN/cm2]	Poisson's Ratio v	Speciment Weight ɣ [kN/m3]	Coefficient of the Expansion α[1/oC]	Partial Factor ɣM	Material Model
1	Concrete fc = 4000 psi [AC] 318-14						isotropic Linear
	2485.56	1035.65	0.2	22.62	9.90E-06	1	Elastic
2	Steel A992 [ANSI/AISC 360-16:2016						isotropic Linear
	19994.8	7722.13	0.295	78.49	1.20 E-5	1	Elastic
3	Concrete C30/37/BS EN 1992-1-1/NA 2005-12						isotropic Linear
	3300	1375	0.2	25	1.00 E-5	1	Elastic

Appendix F: SURFACES

Surface No	Surface Type Geometry	Stiffness	Boundary Lines No	Material No	Thickness Type	d (mm)	Area A (m2)	Weight W (kg)
1	Plane	Standard	1,2,10,8,3,4,16,14	3	Constant	200	13.5	6750
2	Plane	Standard	19,20,28,26,21,22,34	3	Constant	200	13.5	6750
3	Plane	Standard	37,38,46,44,39,40,52	3	Constant	200	13.5	6750
4	Plane	Standard	67-70	3	Constant	200	13.5	6750

Appendix G: CROSS-SECTIONS

Material No	J (cm^4) A (cm^2)	I_y (cm^4) A_y (cm^2)	I_z (cm^4) A_z (cm^2)	Principal Axes α	Rotation	Overall Dimensions (mm) Width (b)	Height (h)
Rectangle	300/600						
3	370777.5	540000	135000	0	0	300	600
	1800	1500	1500				
Rectangle	250/500						
3	178808.6	260416.7	65104.17	0	0	250	500
	1250	1041.67	1041.67				
HEB							
2	59.28	5696	2003	0	0	200	200
	78.08	50.04	15.35				
HEB							
2	9.25	449.5	167.3	0	0	100	100
	26.04	16.71	4.97				

Appendix H: MEMBERS

		Rotation		Cross-Section		Length	
Line No	Member	Type	ß	Start	End	L (m)	
5	Beam	Angle	0	1	1	3	Z
6	Beam	Angle	0	1	1	3	Z
7	Beam	Angle	0	1	1	3	Z
9	Beam	Angle	0	1	1	3	Z
11	Beam	Angle	0	1	1	3	Z
12	Beam	Angle	0	1	1	3	Z
13	Beam	Angle	0	1	1	3	Z
15	Beam	Angle	0	1	1	3	Z
4	Rib	Angle	0	2	2	1.5	X
16	Rib	Angle	0	2	2	1.5	X
14	Rib	Angle	0	2	2	1.5	X
1	Rib	Angle	0	2	2	3	Y
18	Rib	Angle	0	2	2	3	Y
17	Rib	Angle	0	2	2	3	Y
3	Rib	Angle	0	2	2	3	Y
2	Rib	Angle	0	2	2	1.5	X
10	Rib	Angle	0	2	2	1.5	X
8	Rib	Angle	0	2	2	1.5	X
23	Beam	Angle	0	1	1	3	Z
24	Beam	Angle	0	1	1	3	Z
25	Beam	Angle	0	1	1	3	Z
27	Beam	Angle	0	1	1	3	Z
29	Beam	Angle	0	1	1	3	Z
30	Beam	Angle	0	1	1	3	Z
31	Beam	Angle	0	1	1	3	Z
33	Beam	Angle	0	1	1	3	Z
22	Rib	Angle	0	2	2	1.5	X
34	Rib	Angle	0	2	2	1.5	X
32	Rib	Angle	0	2	2	1.5	X
19	Rib	Angle	0	2	2	3	Y
36	Rib	Angle	0	2	2	3	Y

Appendix I: LOAD CASES-CALCULATION PARAMETERS

Load Case Description	Calculation Parameters	
		Members (factor for GJ, EI, EA, GA,)
	Method of analysis	Geometrically linear analysis
Imposed q121212qq1qqqqload	Method for solving system of nonlinear algebraic equations	Newton-Raphson
	Activate stiffness factors	Cross-sections (factor for J, I, A) Members (factor for GJ, EI, EA, GA)
Wind	Method of analysis	Geometrically linear analysis
	Method for solving system of nonlinear algebraic equations	Newton-Raphson
	Activate stiffness factors	Cross-sections (factor for J, I, A) Members (factor for GJ, EI, EA, GA)

Appendix J: Deflection on Members