DESIGN OF A TWO STOREYED CLASSROOM BLOCK FOR ARIPEA SECONDARY SCHOOL

A FINAL YEAR PROJECT REPORT SUBMITTED TO KAMPALA INTERNATIONAL UNIVERSITY IN PARTIAL FULFILLMENT OF THE AWARD OF BACHELOR OF SCIENCE IN CIVIL ENGINEERING

BY

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DECLARATION

I, Embatiru Flavia, do declare that this final year project of the design of a two storeyed classroom block for Aripea secondary school is my original work and has never been submitted in any form in any university or institution of higher learning for the award of a Bachelor's degree in civil engineering.

Date signature

………………… …………………………….

APPROVAL

This final year project, design of a two storeyed classroom block has been done under the supervision of;

MR BUKENYA STANLEY

Sign………………………………………………. Date …………………………………………….

DEDICATION

This project report is dedicated to all who are aspiring to be women engineers, my parents, and siblings, friends who greatly contributed in terms of knowledge, skills, finance and many other things.

ACKNOWLEDGEMENT

I thank the almighty God for the good health and care given to me ever since first year up to this level amidst the challenges.

I would like to express my sincere gratitude to all those who gave me the possibility to carry out this project report .I deeply appreciate my supervisor who tirelessly sacrificed to enhance me with knowledge to come up with this report, not forgetting to give special thanks to my parents and guardians for the financial support, care and love which has enabled me to complete this project report. GOD reward abundantly on my behalf.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

This is my final year project report for the award of a Bachelor's degree in civil engineering at Kampala International University that presents the structural Design of a two storied classroom block for Aripea secondary school.

This report comprises of three chapters, highlighting the order in which the design was carried out.

A detailed introduction of the project that was considered for the design; which includes the background and introduction of both the project and the site.

Problem statement, objectives i.e. main and specific objectives, the scope of the project, project significance and justification.

The Literature Review presenting clear information in details concerning the type of structure, the design philosophy, and safety measures that were ensured for successful design of the project.

The Methodology presents the materials and methods that were undertaken to achieve the desired aims and objectives of the project. It covers the Loading computations, Choice of materials, Structural analysis and design analysis.

CHAPTER ONE: INTRODUCTION

1.0 Introduction

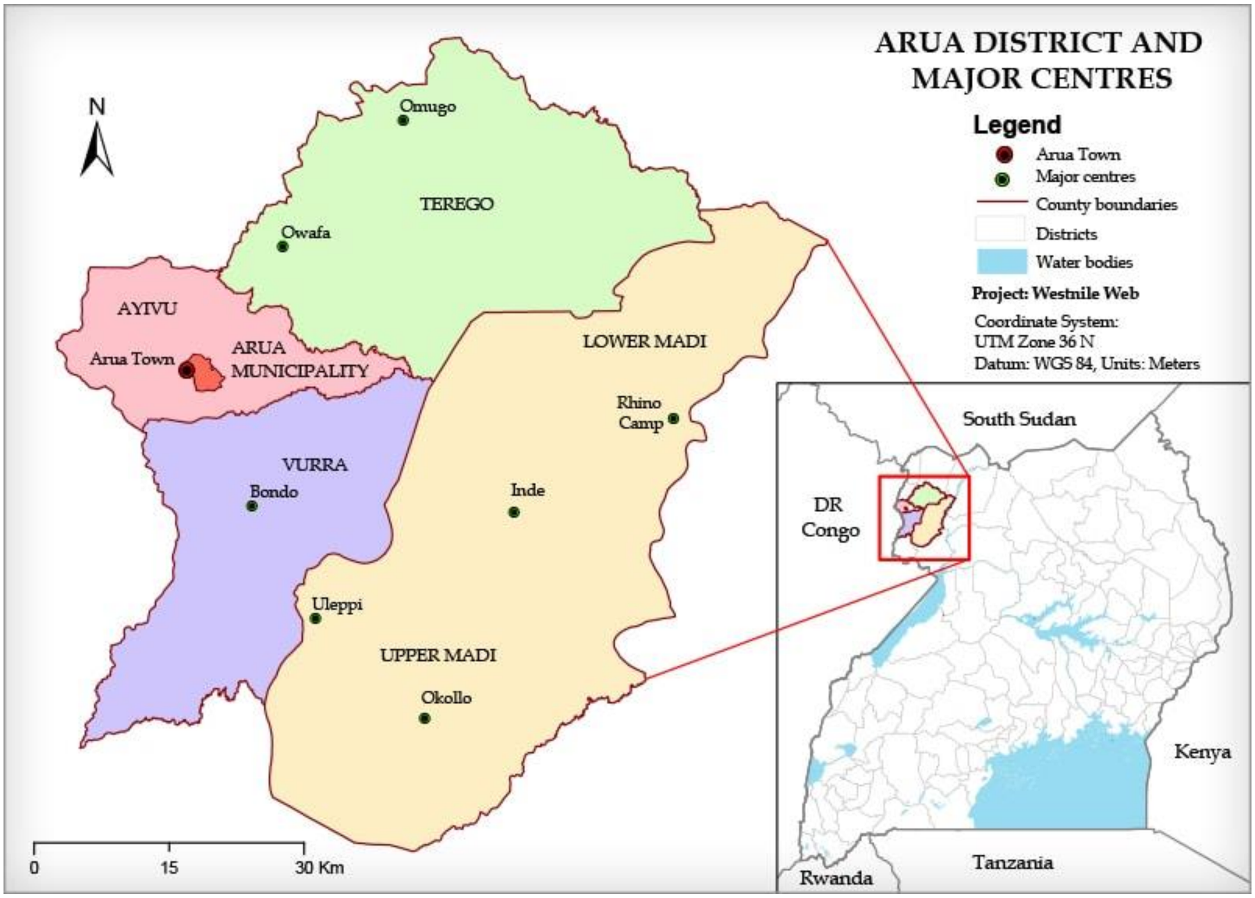
This chapter presents the definition of terms, back ground of the project, the problem statement, project objectives (purpose of study), research questions, scope of the project all applied to suit the context of the report .

1.1 Background

Arua district is located strategically at the vanguard of two countries namely; south Sudan and democratic republic of Congo. It lies in the north western corner of Uganda between latitude 20 30'N and 30 50'N and longitude 300 30'E and 310 30'E. it is bordered by Maracha district to the north west, Yumbe district to the north east, democratic republic of Congo to the west, Nebbi district to the south and Gulu district to the east. Arua is 1211 meters above sea level. According to Uganda bureau of statistics figures the district had a total population of 782,077 in 2014 with a projected population of 862,700 in 2018 of which 52.1% are female.

The administrative and commercial headquarters of the district is 520 kilometers away from Kampala, Uganda's capital city. The district covers a total Area of 4,274.12km^2

Aripea Secondary school is a government aided school found in Aripea town, Aii-vu Sub-County, Arua District. It was established in 1990 under the pioneer-ship and ownership of Fr. Tonino Cosmas. Aripea town has a population of about 65000.



Geographical location of Arua district.

1.2 Problem statement

The increasing population of Aripea village with Aripea S.S being the only secondary school in the area led to over population in the school.

The school was initially built to accommodate about 450 students but now it has over 1000 students where by the students are over congested in every classroom, this has led to inadequate facilities in the school such as classrooms, toilets, etc leading to poor health and poor education services since teacher to student ratio is very small. Hence Design of a two storey classroom block for Aripea secondary school

1.3 objectives of the study

1.3.1 Main objectives

To Design a two story classroom block for Aripea secondary school.

1.3.2 Specific objectives

- To produce architectural drawing of Aripea S.S classroom block
- To analyze the structure.
- To design the structure
- To produce structural drawings

1.4 The significance of the project

- The project bridged the gap between theoretical skills studied in class and the practical skills that are always needed in day today practice in the field of Civil Engineering.
- The project enhanced the academic experience through carrying out research, engineering practice to come up with the architectural and structural drawings, as well as design of a two story classroom building.
- The project also expanded my creativity while seizing the professional ethical values as a basis to venture into the professional career in future. This has build confidence and skills to be competitive in the current job market.

1.5 Scope of the project.

This project involved generation of architectural drawings, analysis of the members, designing and producing structural drawings.

1.5.1 Time scope

The project started January 2019 and ended in September 2019.

1.5.2 Geographical scope

The project is located in Aripea secondary school, Aii-vu sub-county, Terego County, Arua district

1.6 Justification

This project is intended to provide enough space and accommodation for the increasing number of students in the school.

It also includes the various facilities required in a school such as the library, offices, science laboratory, and sick bay. All this provides conducive environment for learning and reduces unnecessary movements on the compound hence increasing the rate of concentration of students improving their performance.

CHAPTER TWO : LITERATURE REVIEW

2.0 Introduction

The primary aim of a structural design is to ensure that the structure performs satisfactorily throughout its design lifespan.

Checks were made to ensure that the structure is capable of carrying the loads safely and that it does not deform excessively due to applied loads. Realistic estimates of the strengths of the material composing the structure and the loading to which it may be subject during its design lifespan was made, therefore, a basic understanding of structural behavior is needed.

The task of the Structural Engineer is to design a structure, which satisfies the needs of both the client and the intended user.

Specifically, the structure must be safe, economical to build and maintain, and aesthetically pleasing.

For the structural engineer the major difference between low and tall buildings is the influence of the wind and earthquake forces on the behavior of the structural elements. Generally, it can be stated that a tall building structure is one in which the horizontal loads are an important factor in the structural design. In terms of lateral deflections a tall building is one in which the structure, sized for gravity loads only, will exceed the allowable sway due to additionally applied lateral loads. This allowable drift is set by the code of practice. If the combined horizontal and vertical loads cause excessive bending moments and shear forces the structural system must be augmented by additional bracing elements. These could take several forms. Cross-sections of existing beams and columns can be enlarged. The analysis of tall structures pertains to the determination of the influence of applied loads on forces and deformations in the individual structural elements such as beams, columns and walls.

The design deals with the proportioning of these members for reinforced concrete structures this includes sizing the concrete as well as the steel in an element. Structural analyses are commonly based on established energy principles and the theories developed from these principles assume linear elastic behaviour of the structural elements. Non-linear behaviour of the structure makes the problem extremely complex. It is very difficult to formulate, with reasonable accuracy the problems involving inelastic responses of building materials. At present the forces in structural components and the lateral drift of tall structures can be determined by means of an elastic method of analysis regardless of the method of design.

The starting point for the designer is usually a conceptual brief from the client, who may be a private developer or perhaps a government body. The conceptual brief may simply consist of some sketches prepared by the client or perhaps a detailed set of the Architect's drawings.

The engineering design process can often be divided into two stages:

1. A feasibility study involving comparison of the alternative forms of structures and selection of the most suitable type.

2. A detailed design of the chosen structure.

The success of stage 1, the conceptual design, relies to a large extent on engineering judgment and instinct, both of which are the outcome of many years' experience in designing structures. Stage 2, the detailed design, requires these attributes but is usually more dependent upon a thorough understanding of the codes of practice for structural design, e.g. BS8110 and BS5950. These documents are based on the past experience of many generations of Engineers, and the results of research. They help to ensure safety and economy of construction, and that mistakes are not repeated.

2.1 Design

The design of reinforced concrete elements BS 8110 is based on the limit state method.

2.1.1 Design method

The design method outlined in this team project report is the limit state design. In addition, considerations were given to the requirement for durability and fire resistance.

Equally important are the consideration of suitable materials, workmanship and quality control.

2.1.2 Design working life

The design working life from code of practice assumes a design working life of 50 years, which is deemed appropriate for general buildings and other common structures (T.J McGinley, 1981).

2.1.3 Durability, workmanship and materials

The structure should be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance. In order to achieve an adequately durable structure, the following should be taken into account; The intended or foreseeable use of the structure, the required design criteria, the shape of members and the structural detailing, the quality of workmanship and the level of control (T.J McGinley, 1981).

2.1.4 Limit States Design

In general, structural design requirements relate to corresponding limit states, so that the design of a structure that satisfies all the appropriate requirements is termed a limit states design. Structural design criteria may be determined by the designer, or may use those stated or implied in design codes. The stiffness design criteria are usually related to the serviceability limit state. These may include excessive deflections, vibration, noise transmission, member distortion, etc. Strength limit states pertain to possible methods of failure or overload and include yielding, buckling, and brittle fracture. (T.J McGinley, 1981).

The two principal categories of limit states normally considered in design are:

(i) Ultimate limit state

(ii) Serviceability limit state. (Chanakya, 2009)

2.1.4.1 Ultimate state design

Ultimate limit state (ULS) is related to the safety of people and the safety of the structure. The ultimate limit state is concerned with the strength, stability, collapse, overturning, and buckling of the structure. The ultimate limit state models the behavior of the element at failure due to a variety of mechanisms including excessive bending, shear and compression or tension. In reinforced concrete structures, the ultimate limit states of bending and shear are used to determine the size of the beam. The design is then checked for the remaining limit states, e.g. deflection and cracking. (Chanakya, 2009)

2.1.4.2 Serviceability limit state

The serviceability limit state models the behavior of the structural members at working loads and in the context of reinforced concrete; design is principally concerned with the limit states of deflection and cracking. (Chanakya, 2009) The serviceability limit states are discussed in BS8110: Part 1, section 2.2.3. The code states that account is to be taken on temperature, creep, shrinkage, sway and settlement. The main serviceability limit states and code provisions are as follows.

- **Deflection**

The deformation of the structure should not adversely affect its efficiency or appearance. Deflections may be calculated, but in normal cases span-to-effective depth ratios can be used to check compliance with requirements (T.J McGinley, 1981).

- **Loads**

The loads acting on a structure are divided into three basic types namely dead, imposed and wind. Associated with each type of loading there are characteristic and design values which must be assessed before the individual elements of the structure can be designed (T.J McGinley, 1981).

2.1.4.3 Characteristic design loads

The characteristic or service loads are the actual loads that the structure is designed to carry. These are normally thought of as the maximum loads which will not be exceeded during the life of the structure. In statistical terms, the characteristic loads have 95% probability of not being exceeded. The characteristic loads used in design are defined in BS8110: Part 1, clause 2.4.1, are as follow:

- The characteristic dead load Gk is the self-weight of the structure and the weight of finishes, ceilings, services and partitions;
- The characteristic imposed load Qk is load imposed movements of people, furniture and equipment. On floors and snow on roofs. Imposed loads for various types of buildings are given in BS 6399: Part 1

2.2 Materials, properties and design strengths

The characteristic strengths or grades of materials are as follows; Concrete, fcu is the 28 days cube strength in Newton per square millimeter Reinforcement, Fy is the yield or proof stress in Newton per square millimeter. The minimum grades for reinforced concrete are given in Table 3.4, BS 5338 in the code. These grades are; 25, 30, 35, 40, 45 and 50 in Newton per square millimeter. The specified characteristic strengths of reinforcement given in the table below

Table 2.1: Strength of Reinforcement

Designation	Characteristic specified strength fy(N/mm^2)
High yield steel, hot rolled or cold worked	460
Hot rolled mild steel	250

(Source BS 8110-1:1997)

Table 2.2: Values of fy for various load combinations

Load combination	Load type				
	Dead, G_k		Imposed, Q_k		Wind, W_k
	Adverse	Beneficial	Adverse	Beneficial	
Dead and impose	1.4	1.0	1.6	0	-
Dead and wind	1.4	1.0	-	-	1.4
Dead and wind and imposed	1.2	1.2	1.2	1.2	1.2

(Source, BS 8110-1:1997)

2.3 Detailing

The general arrangements of drawings give the overall layout and principal dimensions of the structure. The structural requirements for the individual elements are presented in the detail drawings. The output of the design calculations are sketches giving sizes of members, arrangement, and spacing and cut-off points for reinforcing bars at various sections of the structure. Detailing translates this information into a suitable pattern of reinforcement for the structure as a whole.

2.4 Design Process

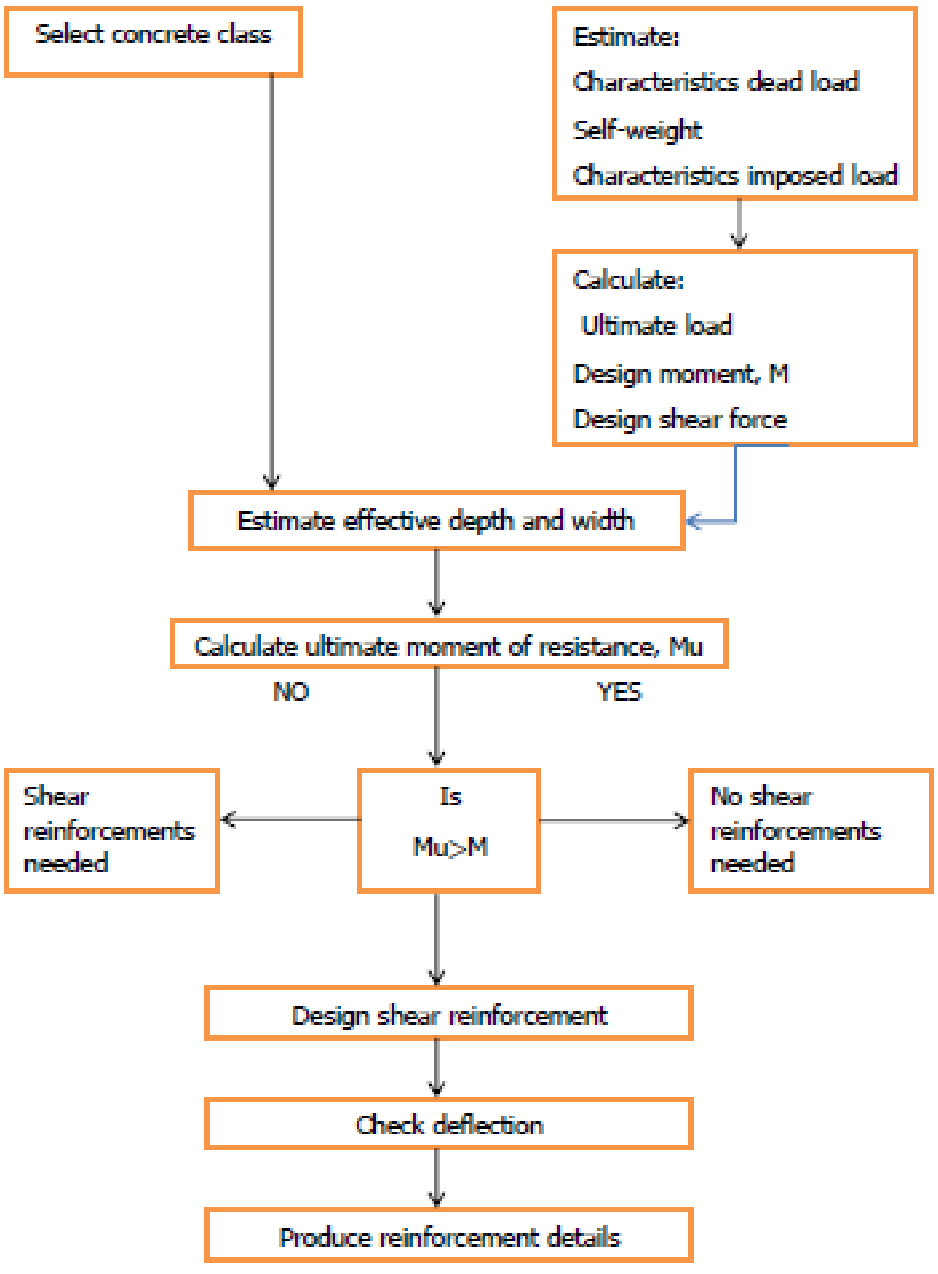


Figure 2.1: Design process (Chanakya A., 2009)

2.5 Structural Elements

The complete building structure can be broken down into the following elements: Beams, slabs, columns, walls, bases and foundations (pads or strips).

2.5.1 Beams

Beams are usually straight horizontal members used primarily to carry lateral loads. Quite often they are classified according to the way they are supported (R.C.Hibbeler, 2012). Beams carry lateral loads in roofs, floors, etc. and resist the loading in bending, shear etc. The design must comply with the ultimate and serviceability limit states.

- **Simply supported beams**; simply supported beams do not occur as frequently as continuous beams in in-situ concrete construction. They are an important element in precast concrete construction. The effective span of a simply supported beam is defined in BS 8110: Part 1.

- **Continuous beams**; Continuous beams are common elements in cast-in-situ construction. A reinforced concrete floor in a multistory building (T.J McGinley 1981).

2.5.1.1 Curtailment and anchorage of bars in beams.

General and simplified rules for curtailment of bars in beams are set out in BS 8110: Part 1, section 3.12.9. The same section also sets out requirements for anchorage of bars at a simply supported end of a beam. These provisions are set out as; Clause 3.12.9.1 of the code states that except at end supports every bar should extend beyond the point at which it is no longer required to resist moment by a distance equal to the greater of;

- Twelve times the bar.
- The effective depth of the beam.

2.5.2 Slabs

Slabs are plate elements forming floors and roofs in buildings which normally carry uniformly distributed loads. Slabs may be simply supported or continuous over two or more supports and are classified according ribbed and precast. These can be classified according to BS 8110-1:1997 as both two way ($l_y/l_x \leq 2$) and one way ($l_y/l_x>2$)

spanning slabs. In practice, the choice of slab for a particular structure largely depends upon economy, build-ability, the loading conditions and the length of the span.

Slabs can be classified as one way slab, two way slab, flat slab, ribbed slab with definitions in BS 8110 Cl. 5.2.1.1 of BS Code.

One way slab is defined by the BS 8110 as one subjected predominantly to uniformly distributed load (U.D.L). Either it possesses two free and parallel edge or it is the central part of a rectangular slab supported on four edges with a ratio of the longer to the shorter span greater than 2.

Two way slab is a square or rectangular one supported on four sides with length to breadth ratio smaller than 2 (T.J McGinley, 1981)

2.5.3 Columns

Columns are structural members in buildings carrying roof and floor loads to the foundations. They are classified as short when Lex/h and ley/b are less than15 (braced) and 10 (unbraced) otherwise slender. The primary purpose of the column is to transfer loads in the vertical direction to the foundation. In braced frames, i.e. those in which the lateral loading is transferred by structural elements such as shear walls, core or bracing, the columns are subjected to axial loading in addition to moments due induced by the dead and imposed loads only (BS 8110-1:1997)

The recommendations of 3.8 of BS 8110: part 1:1997 clause 3.8.1.3 defines the column as short or slender. Slender columns are subjected to moments due deflection of the columns which must be added to those calculated for the loading and sway effects. The definition of short and slender is dependent on Lex/h and ley/b ratios of the column. Slenderness ratios definitions are given in BS 8110: part 1:1997 clause 3.8.1.3 and summarized in the table 2.3 below:

Table 2.3: Defining slenderness ratios for short columns

Defining slenderness ratios for short columns		
	Lex/h	**ley/b**
Braced column	15	15
Un-braced column	10	10

Source BS 8110: part 1:1997 Clause 3.8.1.3

The columns that have values greater than in the table above are considered to be slender. The effective height (Lex) can be evaluated using the le=βlo where β is a coefficient which is dependent on the end condition of the column and lo is the clear height between the end restraints.

These conditions are defined in BS 8110: part 1:1997 clause 3.8.1.6.2 of the code as shown in table below

Table2.4 constants for moment distribution

Values of β for braced columns			
End condition at the top	End condition at the bottom		
1	1	2	3
2	0.75	0.80	0.90
3	0.80	0.85	0.95
4	0.90	0.95	1.00
Values of β for un-braced columns			
End condition at the top	End condition at the top		
	1	2	3
1	1.2	1.3	1.6
2	1.3	1.5	1.8
3	1.6	1.8	-
4	2.2	-	-

2.5.4 Foundations

A foundation is a constructed unit that transfers the load from a superstructure to the ground. With regard to vertical loads, most foundations receive a more or less concentrated load from the structure and transfer this load to the soil underneath the foundation, distributing the load as a stress over a certain area. Part of the soil structure interaction is then the condition that the stress must not give rise to a deformation of the soil in excess of what the superstructure can tolerate. (Bengt H Fellenius, 1992) Foundations transfer loads from the building or individual columns to the earth. Types of foundations are;

- Isolated basis for individual columns
- Combined bases for several columns
- Rafts for whole buildings which may incorporate basements.

Foundation failure can produce catastrophic effects on the overall stability of structure so that it may slide or even overturn. The size of a foundation bearing directly on the ground depends on the safe bearing pressure of the soil, which is taken to mean the bearing pressure that can be imposed without causing excessive settlement (Bengt H Fellenius, 1992). Values for various soil types and conditions are given in BS 8004

2.5.4.1 Axially loaded pad bases

For axially loaded pad footing the following symbols are used:

Gk = characteristic dead load from the column (KN)

Qk = characteristic imposed load from the column (KN)

W = weight of the base (KN)

Lx, ly = base length and breadth (m)

Pb= safe bearing pressure (kN/m^2)

The area required is obtained from the characteristic loads including the weight of the base:

Area= $Gk+Qk+wPb=lxly$ (m^2)

The design of the base is made for the ultimate load delivered to the base by the column shaft, i.e. the design load is 1.4Gk+1.6Qk.

There are many types of foundations which are commonly used, namely strip, pad and raft. The foundations may be directly on the ground or be supported on piles. The choice of foundation type will largely depend upon;

- Ground conditions (strength and type of soil)
- Type of structure (layout and level of loading)

2.5.4.2 Footings

Footings are structural members used to support columns and walls and to transmit and distribute their loads to the soil in such a way that;

- The load bearing capacity of the soil is not exceeded
- Excessive settlement, differential settlement, and rotations are prevented
- Adequate safety against overturning or sliding is maintained

When a column load is transmitted to the soil by the footing, the soil becomes compressed. The amount of settlement depends on many factors, such as the type of soil, the load intensity, the depth below ground level, and the type of footing. (Breen, J. E. 1991)

2.5.4.3 Types of Footings

Different types of footings may be used to support building columns or walls. For walls, a spread footing is a slab wider than the wall and extending the length of the wall. Square or rectangular slabs are used under single columns when two columns are so close that their footings would merge or nearly touch a combined footing. When a column footing cannot project in one direction, perhaps because of the proximity of a property line, an adjacent footing with more space may be incorporated; either a combined footing or a strap (cantilever) footing may be used under the two. For structures with heavy loads relative to soil capacity, a mat or raft foundation may prove economical. A simple form is a thick, two-way-reinforced-concrete slab extending under the entire structure. In effect, it enables the structure to float on the soil, and because of its rigidity, it permits negligible differential settlement (Breen, J. E. 1991).

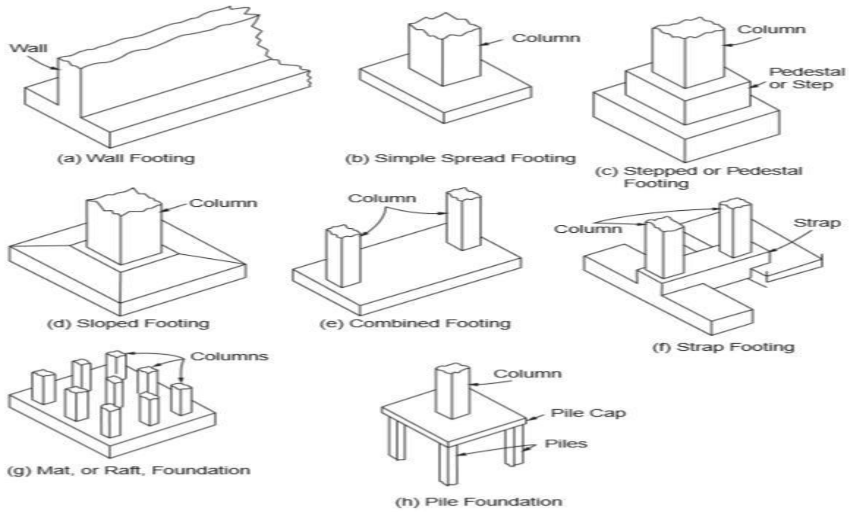


Figure 2.2: Types of footing (Breen, J. E. 1991)

Pad footings (Simple Spread Footing) are usually square or rectangular slabs and used to support a single column. The pad may be constructed using mass concrete or reinforced concrete depending on the relative size of the loading. (Breen, J. E. 1991). For this case, isolated pads were designed.

2.5.5 Walls

Walls are vertical plate elements resisting vertical and lateral loads. As a general rule, the exterior walls of a reinforced concrete building are supported at each floor by the skeleton framework, their only function being to enclose the building. Such walls are called panel walls. They may be made of concrete (often precast), concrete block, brick, tile blocks, or insulated metal panels. The thickness of each of these types of panel walls vary according to the material, type of construction, climatological conditions, and the building requirements governing the particular locality in which the construction takes place. The pressure of the wind is usually the only load that is considered in determining the structural thickness of a wall panel, although in some cases exterior walls are used as diaphragms to transmit forces caused by horizontal loads down to the building foundations. (Breen, J. E. 1991)

2.5.6 Frames

A structural frame is a three-dimensional structural system consisting of straight members that are built monolithically and have rigid joints. The frame may be one bay long and one story high such as portal frames and gable frames or it may consist of multiple bays and stories. All members of the frame are considered continuous in three directions, and the columns participate with the beams in resisting external loads. Consideration of the behavior of reinforced concrete frames at and near the ultimate load is necessary to determine the possible distributions of bending moment, shear force, and axial force that could be used in design. It is possible to use a distribution of moments and forces different from that given by linear elastic structural analysis if the critical sections have sufficient ductility to allow redistribution of actions to occur as the ultimate load is approached. (Breen, J. E. 1991)

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CHAPTER THREE: METHODOLOGY

3.0 Introduction

This chapter involves the methods that were undertaken, tools and procedures used to come up with a two storey classroom block for Aripea S.S

3.1 Reconnaissance study

Assessment study was carried out to know whether the proposed project would fit in the proposed area. This was done by simple observation using the naked eye and measurements using a tape measure were done to confirm the observation.

3.2 Preparation of the Architectural drawing

Architectural drawings were prepared using Arch CAD 18, a platform from which the plans, elevations, and pictorial views were prepared (this objective was already achieved in the proposal).

3.3 Structural layout.

The structural layout were prepared using Auto CAD where the structural elements (beams, slab, columns and foundation) were allocated in different positions on the structural plan. The reinforcement bars and the cross sections of the structural elements were also shown on this layout

3.4 Design loads.

Two types of design loads were considered as discussed below;

3.4.1 Imposed loads

The imposed loads adopted in the design were in accordance with BS 6399 depending on the functionality of the areas being designed. The details were obtained from table 1 of BS 6399 for floor and beam loads. Columns and the foundation were derived from the summation of load from roof to where the structural elements were.

3.4.2 Dead loads

Dead loads for structural members were assessed basing on the forces due to: weight of the member itself, weight of all materials of construction incorporated into the building to be supported permanently by the member, weight of permanent partitions and weight of fixed service equipment. Dead loads were calculated from the unit weights given in BS6399 or from the actual known weights of the materials used. Dead loads of beams, columns and bases were calculated depending on their sectional areas.

Analysis of the structure.

The structure was analyzed using the force method of truss analysis and moment distribution method.

Design of the structure.

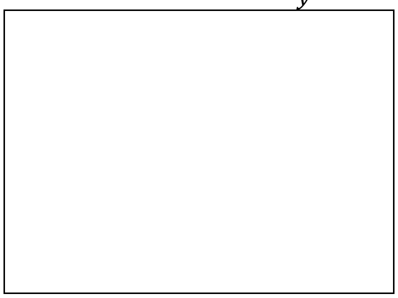
The structure was designed using BS8110, Bs 648, and Bs 6399 codes, design of structural elements by chanakya.

3.5.5 Preparation of structural drawings

Structural drawings were prepared using AutoCAD software.

CHAPTER FOUR:ANALYSIS AND DESIGN OF STRUCTURAL ELEMENTS

REFERENCE	CALCULATION	OUTPUT
	<u>SLAB DESIGN</u> **Specifications**	
B s 8500-1 2006 Table A.1	Exposure condition; exposure class Xc_1	
B s 8500-1 2006 Table A.3	Concrete grade C 20/25 $fcu = 25N/mm2$	$fcu = 25N/mm2$
	concrete cover= $15 + \Delta c$ =15+10 =25	$c = 25mm$
Bs8110-1 Table 3.9	Depth of the slab $\frac{span}{depth} = 26$ $\frac{4000}{26} = depth$ Depth =153.8	Depth =154mm
	Thickness of slab , h $\emptyset bar = 12mm$ $h = d + c + \emptyset \frac{bar}{2}$ h=154+25+6 =185mm	h=185mm
	LOADINGS; Imposed load Purpose of the structure; school Imposed load$\theta_n = 3.0KN/M^2$	
B s 6399-1 Table 1	Dead load Unit weight of concrete =24KN/m3 Finishes =20kn/m3 Screed =50mm	
B s 648	Weight of finishes =0.05x20 1.0kn/m2 Self-weight of slab= slab thickness x unit of concrete = 0.185x24 =4.44kn Dead load G k=weight of finishes + self- weight of	G k=5.4kn/m2

	slab =1.0+4.44 Design load $n = 1.4G_k + 1.6Q_k$ =1.4*5.4+1.6*3.0 =12.36kn/m2	n=12.36kn/m2
	Analysis $l_y = 5m$  $\frac{l_y}{l_x}$ =5/4 ≤2.0 =1.3	
B s 8110 Cl 3.5.3.4	Moments $$M_{sx} = \beta_{sx} \times n \times l_x^2$$ $$M_{sy} = \beta_{sy} \times n \times l_x^2$$ One long edge-discontinuous	
Table 3.14	see table below	

	β_{sx}	β_{sy}
-v e moment	0.062	0.037
+v e moment	0.047	0.028

	<u>Moments at the mid span</u> $$M_{sx} = \beta_{sx} \times n \times l_x^2$$ =0.047*12.36*4^2 = 9.29	$M_{sx} = 9.29KNm$
	$$M_{sy} = \beta_{sy} \times n \times l_x^2$$ =0.028*12.36*4^2 =5.54KNM	$M_{sy} = 5.54KNm$
	<u>Reinforcement at the mid span</u> **in x-direction** $$M_{sx} = 9.29KNm$$ Effective depth d=154mm	

B s 8110 part 1 Cl 3.4.4.4	Breadth b=1000mm $K = Mu/f_{cu} \times b \times d^2$ =9.29*10^6/25*1000*154^2 =0.01567 K<K' single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 154(0.5 + \sqrt{(0.25 - \frac{0.0157}{0.9}})) \leq 0.95 * 154$ 151.23>146.3	K=0.01567 Compression reinforcement not required Z=146.3
Table 3.22 chanakya	$A_s = \frac{Mu}{0.95 f_y Z}$ =9.29*10^6/0.95*460*146.3 =145.31mm2 **In Y-direction** $M_{sy} = 5.54KNm$ $K = Mu/f_{cu} \times b \times d^2$ =5.54*10^6/25*1000*154^2 =0.00934	$A_s = 145.31mm$ Approve=377mm2 T12@300mm cc K=0.00934
B s 8110 part 1 Cl 3.4.4.4	K<K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 154(0.5 + \sqrt{(0.25 - \frac{0.00934}{0.9}})) \leq 0.95 * 154$ 152.44>146.3	Compression reinforcement not required Z=146.3
Table 3.22 chanakya	$A_s = \frac{Mu}{0.95 f_y Z}$ =5.54*10^6/0.95*460*146.3 =86.65mm2 **<u>Moment at the support</u>** $M_{sx} = \beta_{sx} \times n \times l_x^2$ =0.062*12.36*4^2	$A_s = 86.65mm$ Approve=377mm2 T12@300mm cc

	= 12.26 $$M_{sy} = \beta_{sy} \times n \times l_x^2$$ =0.037*12.36*4^2 =7.32 KNM **<u>Reinforcement at the support</u>** **in x-direction** $$M_{sx} = 12.26KNm$$ Effective depth d=154mm Breadth b=1000mm $$K = Mu/f_{cu} \times b \times d^2$$ =12.26*10^6/25*1000*154^2 =0.02	$M_{sx} = 12.26KNm$ $M_{sy} = 7.32KNm$ K=0.02
B s 8110 part 1 Cl 3.4.4.4	K<K'=0.156 single reinforcement $$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$$ $$Z = 154(0.5 + \sqrt{(0.25 - \frac{0.02}{0.9}})) \leq 0.95 * 154$$ 150.53>146.3	Compression reinforcement not required Z=146.3
Table 3.22 chanakya	$$A_s = \frac{Mu}{0.95 f_y Z}$$ =12.26*10^6/0.95*460*146.3 =192mm2 **In Y-direction** $$M_{sy} = 7.32KNm$$ $$K = Mu/f_{cu} \times b \times d^2$$ =7.32*10^6/25*1000*154^2 =0.0123	$A_s = 192mm2$ Approve=377mm2 T12@300mm cc K=0.0123
B s 8110 part 1 Cl 3.4.4.4	K<K'=0.156 single reinforcement $$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$$ $$Z = 154(0.5 + \sqrt{(0.25 - \frac{0.0123}{0.9}})) \leq 0.95 * 154$$	Compression reinforcement not required Z=146.3

Table 3.22 chanakya	151.86>146.3 $$A_s = \frac{Mu}{0.95f_yZ}$$ =7.32*10^6/0.95*460*146.3 =114mm2 **Deflection at mid span** Basic l/d ratio=26 $$MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$$ $$fsd = 2/3fy \times As/Aprov \times 1/\beta_b$$ =2/3*460*145.3/377*1 =283.8N/mm Mu/bd^2=9.29*10^6/1000*154^2 =0.39 MFT=0.55+(477-283.8)/120(0.9+0.39) =1.8<2 Allowable l/d ratio=Basic l/d ratio*MFT =26*1.8 =46.8 Actual l/d=4000/154=26 Allowable l/d >actual l/d	$A_s = 114mm2$ Approve=377mm2 T12@300mm cc Deflection is satisfactory
Table 3;15 Table 3.8	**Shear check** Design shear stress $V = \frac{v}{bd} \quad \leq 0.8\sqrt{fcu}$ V= $V_{sx} = \beta_{vx} \times n \times l_x^2$ B v x=0.31 B v y=0 V s x=0.31*12.36*4^2 V=61.31*1000/1000*154 V=0.398N/mm $$\frac{100As}{bd} = \frac{100 * 377}{1000 * 154}$$ =0.245 V c = 0.51 > v = 0.398	V=0.398N/mm Vc=0.51

Bs 8110-1 Bs6399 Uganda clays Bs8110-1 Table 3.17 Fig 3.2	**Maxspan slab** Imposed load $\theta_k = 3.0KN/m^2$ Dead load 6" maxspan. 300*300*150 8 maxpans/m2 Weight of maxspan =9.0Kg/unit Weight of maxspan in 1m2 =9.0*8*10/1000 =0.72KN/m2 Topping to maxspan Topping 50mm Rib width 150mm Volume of concrete =volume of 1m2-volume of maxspans =0.2*1*1-0.3*0.3*0.15*8 =0.092 Weight of concrete in 1m2 =0.092*24 =2.208KN/m2 Total self-weight of slab =weight of maxspan in1m2+weight of concrete in 1m2 =2.928Kn/m2 G k=2.928+1.0 =3.93 Design load W = (1.4Gk+1.6Qk)*0.45 =(1.4*3.93+1.6*3)*0.45	No shear reinforcements required G k=3.93KN/m2 W=4.81KN/m

B s 8110 part 1 Cl 3.4.4.4 Table 3.22 chanakya	**Reinforcement at the ribs** At mid span Moment at mid span=6.63KNm $K = Mu/f_{cu} \times b \times d^2$ =6.63*10^6/25*150*169^2 =0.06 K<K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 169(0.5 + \sqrt{(0.25 - \frac{0.06}{0.9}})) \leq 0.95 * 169$ 156.86>160.55 $A_s = \frac{Mu}{0.95 f_y Z}$ =6.63*10^6/0.95*460*56.86 =266.82mm2	K=0.06 Compression reinforcement not required Z=156.86 $A_s = 266.82mm2$ Approve=339mm2 3T12
B s 8110 part 1 Cl 3.4.4.4 Table 3.22 chanakya	**At the support** $K = Mu/f_{cu} \times b \times d^2$ =6.50*10^6/25*150*169^2 =0.061 K<K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 169(0.5 + \sqrt{(0.25 - \frac{0.061}{0.9}})) \leq 0.95 * 169$ 156.64>160.55 $A_s = \frac{Mu}{0.95 f_y Z}$ =6.50*10^6/0.95*460*156.64 =94.957mm2	K=0.061 Compression reinforcement not required Z=156.64 $A_s = 94.957mm2$ Approve=226mm2

	Deflection at mid span Basic l/d ratio=26 $MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$ $fsd = 2/3fy \times As/Aprov \times 1/\beta_b$ $= 2/3 * 460 * 266.82/339 * 1$ $= 241.37N/mm$ Mu/bd^2=$6.63 * 10^6/1000 * 169^2$ $= 0.232$ MFT=$0.55 + (477 - 241.37)/120(0.9 + 0.232)$ $= 1.738 < 2$ Allowable l/d ratio=Basic l/d ratio*MFT $=26 * 1.738$ $= 45.188$ Actual l/d=$4000/169 = 23.6686$ Allowable l/d >actual l/d **Shear check** Design shear stress $V = \frac{v}{bd}$ $\leq 0.8\sqrt{fcu}$ V=6.50*1000/150*169 V=0.256N/mm $\frac{100As}{bd} = \frac{100 * 226}{150 * 169}$ =0.892 V c = 0.78 > v = 0.256 **Design of beam on grid H-H** Depth of the beam $\frac{span}{depth} = 26$ $\frac{4000}{26} = depth$ Depth =153.8 Thickness h=350mm b=300mm	2T12 Deflection is satisfactory V=0.256N/mm Vc=0.78 No shear reinforcements required Depth=154mm
B s 8110-1		

Table 3.9 B s 648 Table 3.12	$\theta k = 3.0KN/m$ **Loading** Self- weight of the beam (KN/m) $=0.35 * 0.3 * 24$ $= 2.52KN/m$ Weight of walls Hollow blocks / light weight aggregate $25.4mm \quad = \quad 25.9Kg/m2$ $300mm \quad = \quad (25.9/25.4) * 300 \; * 10/1000$ $= 3.06\ KN/m$ Weight $= 3.06 * 3 \quad = 9.18KN/m$ $G\ k = \ 9.18 + 2.52 \quad = 11.7KN/m$ Design load $w \ = 1.4\ G\ k \ + \ 1.6\ Q\ k$ $= 1.4 * 11.7 + 1.6 * 3.0$ $= 16.38 + 4.8 \qquad = 21.18kN/m$ $F \ = \ wl \qquad = 21.18 * 4 \qquad = 84.72KN$ **<u>Moment and shear on beam H-H</u>** Mid span moment $M = 0.09Fl \quad = 0.09 * 84.72 \quad = 7.6248KNm$ Moment at the support $M = 0.11\ F\ l \quad = 0.11 * 84.72 \quad = 9.32KNm$ Shear $V = 0.6 * 84.72 \qquad = 50.832KN$ **Reinforcement at mid span** Effective depth d=154mm Thickness=350mm breadth b=300mm $K = Mu/f_{cu} \times b \times d^2$ $=7.6248 * 10\text{^}6/25 * 300 * 1542$	 G k=11.7KN/m W=21.18KN/m

B s 8110 part 1 Cl 3.4.4.4	$= 0.0429$ $K < K' = 0.156$ single reinforcement $$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$$ $$Z = 154(0.5 + \sqrt{(0.25} - \frac{0.0429}{0.9})) \leq 0.95 * 154$$ $108.16 < 146.3$	K=0.0429 Compression reinforcement not required Z=108.16
Table 3.10 chanakya	$$A_s = \frac{Mu}{0.95 f_y Z}$$ $= 7.6248 * 10^6/0.95 * 460 * 108.16$ $= 161.32mm2$	$A_s = 161.32mm2$ Approve=402mm2 2T16
B s 8110 part 1 Cl 3.4.4.4	**<u>Reinforcement at the support</u>** $K = Mu/f_{cu} \times b \times d^2$ $=9.32 * 10^6/25 * 300 * 1542$ $= 0.052$ $K < K' = 0.156$ single reinforcement $$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$$ $$Z = 154(0.5 + \sqrt{(0.25} - \frac{0.052}{0.9})) \leq 0.95 * 154$$ 106.6< 146.3	K=0.052 Compression reinforcement not required Z=106.6
	$$A_s = \frac{Mu}{0.95 f_y Z}$$ $=9.32 * 10^6/0.95 * 460 * 106.6$ $= 200.06mm2$	$A_s = 200.06mm2$ Approve=402mm^2 2T16
Table 3.7 Bs 8110-1 1997 Table 3.13	**Shear check** Design shear stress $V = \frac{v}{bd}$ $\leq 0.8\sqrt{fcu}$ Shear $V = 0.6 * 84.72$ $= 50.832KN$ $V = 50.832 * 1000/300 * 154$ $V = 1.10N/mm$	

Chanakya	$$\frac{100As}{bd}=\frac{100*402}{300*154}$$ $= 0.87$ $Vc = 0.78$ $V\ c\ =\ 0.78\ <\ v\ =\ 1.0898$	Provide shear reinforcement
	$$A_{sv}=\frac{0.4xbvxSv}{0.95fyxv}$$ $Asv = 0.4*300*275.25/0.95*460*1.02$ $Asv = 74.10mm2$ $Asv/Sv = 74.10/275.25$ $= 0.269$	Approv=0.335m2 R8@300mm
	<u>Deflection at the mid span</u> Basic l/d ratio=26 $MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$ $fsd = 2/3fy \times As/Aprov \times 1/\beta_b$ $=2/3*460*637.564/646*1$ $= 302.66N/mm$ Mu/bd^2=$84.348*10^6/300*344^2$ $= 0.2376$ $MFT = 0.55 + (477 - 302.66)/120(0.9 + 0.2376)$ $= 1.28 < 2$ Allowable l/d ratio=Basic l/d ratio*MFT $= 26*1.28$ $= 33.28$ Actual l$/d = 5000/344$ $= 14.53$ Allowable l/d >actual l/d	Deflection is satisfactory
	<u>Beam on grid 8-8</u> **Loading** Weight of walls =$9.81KN/m$ Self-weight of walls =$0.4*0.3*24$ $= 2.88KN$	

	$G\ k = 9.18 + 2.88KN$ $= 12.06KN/m$ Design load from slab $= 1.4Gk + 1.6Qk$ $= 1.4 * 3.93 + 1.6 * 3.0$ $= 10.302KN/m2$ Loading from slab (2*1) $= 2 * 10.302$ $= 20.604KN/m$ $W = 1.4Gk + 20.604$ $= 1.4 * 12.06 + 20.604$ $= 37.488KN/m$ $F = w\ l \quad = 37.488 * 5$ $= 187.44KN$ **<u>Moment and shear on beam 8-8</u>** Mid span moment M= $0.09Fl$ $= 0.09 * 187.44 * 5 \qquad = 84.348KNm$ Support moment $M = 0.11\ F\ l$ $M = 0.11 * 187.44 * 5 \qquad = 103.092KNm$ $Shear \quad V = 0.6 * 187.44 \qquad = 112.464KN$ **Reinforcement at the mid span** Breadth $b = 300mm$ Overall depth $h = 400mm$ h= d + diameter of principal bar + diameter of link + cover	G k=12.06KN/m

	$d = h - C - Q/2$ $d = 400 - 25 - 8$ $d = 367mm$ Effective depth $d = 344mm$ $K = Mu/f_{cu} \times b \times d^2$ $=84.348 * 10^{\wedge}6/25 * 300 * 3672$ $= 0.083$ K>K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 367(0.5 + \sqrt{(0.25} - \frac{0.083}{0.9})) \leq 0.95 * 367$ 329.27< 348.65 $A_s = \frac{Mu}{0.95 f_y Z}$ $=84.348 * 10^{\wedge}6/0.95 * 460 * 329.27$ $= 586.19mm2$ **Reinforcement at the support** $K = Mu/f_{cu} \times b \times d^2$ $=103.092 * 10^{\wedge}6/25 * 300 * 3672$ $= 0.102$ K<K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 367(0.5 + \sqrt{(0.25} - \frac{0.102}{0.9})) \leq 0.95 * 367$ $319.17 < 348.65$ $A_s = \frac{Mu}{0.95 f_y Z}$ $= 103.092 * 10^{\wedge}6/0.95 * 460 * 367$ $= 642.802mm2$	 K=0.083 Compression reinforcement not required Z=329.27 $A_s = 586.19mm2$ Approve=603mm^2 3T16 K=0.102 Compression reinforcement not required Z=319.17 $A_s = 642.802mm2$ Approve=804mm2

	Shear check Design shear stress $V = \frac{v}{bd} \quad \leq 0.8\sqrt{fcu}$ Shear $V = 0.6 * 187.44 \quad = 112.464KN$ $V = 112.464 * 1000/300 * 367$ $V = 1.02N/mm$ $\frac{100As}{bd} = \frac{100 * 804}{300 * 367}$ $= 0.73 \quad Vc = 0.62$ $V\ c\ =\ 0.62\ <\ v\ =\ 1.02$ $A_{sv} = \frac{0.4xbvxSv}{0.95fyxv}$ $Asv = 0.4 * 300 * 275.25/0.95 * 460 * 1.02$ $Asv = 74.10mm2$ $Asv/Sv = 74.10/275.25 \quad = 0.269$ **Deflection at the mid span** Basic l/d ratio=26 $MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$ $fsd = 2/3fy \times As/Aprov \times 1/\beta_b$ $=2/3 * 460 * 586.19/804 * 1$ $= 223.588N/mm$ Mu/bd^2= $84.348 * 10\text{^}6/300 * 367\text{^}2 \quad = 2.087$ $MFT = 0.55 + (477 - 298.12)/120(0.9 + 2.087)$ $= 1.257 < 2$ Allowable l/d ratio=Basic l/d ratio*MFT	4T16 V=1.02N/m Vc=0.6 Provide shear reinforcement Asv=74.10mm^2 Approv=0.402m2 R8@250mm

$=26 * 1.257$

$$= 32.682$$

Actual l/d $= 5000/367$ $= 13.624$

Allowable l/d > actual l/d

Deflection is satisfactory

Column
<u>loadings</u>

Roof loads column	2.57 6.48	15.19
Second floor Walls Beams column	137.72 31.95 10.02 6.48	75.95
First floor Walls beams columns	137.72 31.95 10.02 6.48	75.95
Total	381.39	167.09

Design axial load

$Ned = 1.4Gk + 1.6Qk$

$$= 1.4 * 381.39 + 1.6 * 167.09$$

$$= 801.29\ KN$$

Bs 8110
Part 1
Cl 3.8.1.3

Short column

Cl 3.8.1.6.1

Effective height of column

$Le = \beta Lo$

$$Le = 1.95$$

Cl 3.8.1.3

$$Ratio\ le/h \quad = \ 1950/300 \quad = 6.5 < 15$$

Therefore it's a short column.

Bs 8110
part 1
Cl 3.8.4.4

$N = 0.35 fcuxAc + 0.7Ascxfy$

$$Ac = 0.3\text{^}2 \ - \ Asc$$

$801.29 * 10\text{^}3$

$$= \ 0.35 * 25 * (300\text{^}2 - Asc) + 0.7 * Asc * 460$$

$$801.29 * 10\text{^}3 \ = 787500 - 8.75Asc + 322Asc$$

$$801.29 * 10\text{^}3 \ = 787500 + 313.25Asc$$

$$Asc = 44.02\ mm2$$

$$Minimum\ Area$$

$$100Asc/Acc \ = 0.4\%$$

$$Ascmin = 0.4bh/100 \ = 0.004bh$$

$$= 0.004 * 300 * 300$$

$$= 360mm2 \ > Ascreq$$

As prov =804mm^2
Provide 4T16
As prov =452mm^2
Provide =4T12

Cl 3.4.5.12

Crack control in columns

$$0.2fcu * Ac \ = 0.2 * 25 * 300 * 300 * 10^{\wedge} - 3$$

$$= 450KN$$

Cl 3.8.6

$$Applied\ load\ N = 801.29KN \ > 450$$

No check required

Links
Maximum spacing = $12 * \varphi = 192mm\ \ or\ \ 300$
$spacing = 175$mm

ROOF DESIGN
Result from analysis

members	forces
$FAJ = FIJ$	-24.725KN
$FJK = FPO$	27.165KN
$FKL = FON$	23.188 KN
$FLM = FNM$	18.244 KN
$FAC = FIG$	22.03 KN
$FCD = FGF$	20.92 KN
$FDE = FFE$	19.115 KN

TRUTS	VALUES	TIES	VALUES
JC	-2.4395	CK	2.174
KD	-3.976	DL	3.543
LE	-4.945	ME	4.406

Ec 3 part 1-1 Cl 6.2.3	STRUTS AND TIES $$\frac{Ned}{Nc,Rd} \leq 1.0$$	
Table 9.4 chanakya	Trial section $150\ x\ 90\ x\ 24$ $S275$ Strength classification $tf = 12.0mm$ since t_f <16 fy=275N/mm^2 $$\varepsilon = \frac{\sqrt{235}}{fy}$$ $=(235/275)\text{^}1/2$	
Bs4 2004 page 10	Section classification Flange check in bending $$\frac{C}{tf} \leq 9\varepsilon$$ $R = 12.0mm \quad tw = 6.5mm \quad b = 90mm$ $C = \frac{b-tw-2r}{2} \qquad = \frac{90-6.5-24}{2} \quad = 29.75$ $\frac{29.75}{12.0} \leq 9 \times 0.92 \qquad =2.479 \leq 8.28$	Class 1
Table 9.5 Chanakya	Web classification $\frac{C^*}{tw} \leq 33\varepsilon \qquad but\ C^* = d = 102.0$ $\frac{102}{6.5} \leq 33 \times 0.92 \qquad 15.69 \leq 30.36$	Class 1
	Web at bending(neutral axis $\frac{C^*}{tw} \leq 72\varepsilon \qquad = \frac{102.0}{6.5} \leq 72 \times 0.92$ $= 15.69 \leq 66.24$	
EC 3 part 1.1	**compression** Ned is the design value of the compression force	

Cl 6.2.4	$\frac{Ned}{Nc,Rd} \leq 1.0$ $Nc,Rd = \frac{A \times fy}{\gamma_{mo}} = 30.4 \times 10^2 \times 275/1.0$ $= 836KN$ $\frac{Ned}{Nc,Rd} = \frac{4.945}{836} = 0.0059 < 1$	Class 1
Cl 6.3.1.1 Cl 6.3.1.3	**Buckling resistance of members** $\frac{Ned}{Nb,Rd} \leq 1.0 \qquad Nb,Rd = X\frac{Afy}{\gamma_{m1}}$ $\lambda 1 = 93.9\varepsilon \quad = 93.9 \times 0.92 \quad = 86.388$	
Table 6.2 Bs4 2004	$\lambda = \frac{lcr}{i} \times \frac{1}{\lambda 1} \quad = \frac{2500}{28.9} \times \frac{1}{86.388} \quad = 1.0$ Buckling curve C	
Table 6.1 Cl 6.3.1.2	$\alpha = 0.49$ $\phi = 0.5[1 + \alpha(\lambda - 0.2) + 1.0^2]$ $= 0.5[1 + 0.49(1.0 - 0.2) + 1.0^2]$ $= 0.5[2.392] \quad = 1.19$ $\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} \leq 1.0$ $= \frac{1}{1.19 + \sqrt{1.19^2 - 1.0^2}}$ 0.54<1.0 $Nb,Rd = X\frac{Afy}{\gamma_{m1}} \quad = 0.54 \times 30.4 \times 10^2 \times \frac{275}{1.10}$ $= 410.4KN$ $\frac{Ned}{Nb,Rd} \leq 1.0 \quad = \frac{4.945}{410.4} \leq 1.0$	

	$$0.012 < 1.0$$	The member is safe to buckling
EC 3 part 1.1 Cl 6.2.4	RAFTERS **compression** Ned is the design value of the compression force $$\frac{Ned}{Nc,Rd} = \frac{27.165}{836} \qquad = 0.032 < 1$$	Section is safe in compression
EC 3 part 1.1 Cl 6.3.1.1	**Buckling resistance of members** $$\frac{Ned}{Nb,Rd} \leq 1.0 \qquad = \frac{27.165}{410.4} \leq 1.0$$ $$0.0662 < 1.0$$	The member is safe to buckling
EC 3 part 1.1 Cl 6.2.4	TIE BEAM **compression** Ned is the design value of the compression force $$\frac{Ned}{Nc,Rd} = \frac{22.03}{836} \qquad = 0.026 < 1$$	Section is safe in compression
EC 3 part 1.1 Cl 6.3.1.1	**Buckling resistance of members** $$\frac{Ned}{Nb,Rd} \leq 1.0 \qquad = \frac{22.03}{410.4} \leq 1.0$$ $$0.054 < 1.0$$	The member is safe to buckling
	FOUNDATION DESIGN	
B s 8500-1 2006 Table A.1	**Specifications** Exposure condition; exposure class Xc2	
B s 8500-1 2006 Table A.3	Concrete grade C 25/30 $$fcu = 30N/mm2$$	$fcu = 30N/mm2$

	concrete cover=50mm $fy = 460N/mm^2$ from the column , Gk=381.39KN Qk=167.09 The safe bearing capacity of the soil is 200KN/m^{-2} **Sizing the pad foundation** Plan area of the base **Loading** Design axial load $(N) = 1.0Gk + 1.0Qk$ $= 1.0 * 318.39 + 1.0 * 167.09 \quad = 548.48KN$ Plan area=N/bearing capacity of soil $= 548.48/200 \quad = 2.7424m2$ Dimension of the square base=$\sqrt{2.7424} = 1.656m$ Provide 1.8m square base Therefore plan area $A = 1.8^2 = 3.24$ Assume the overall depth of footing $(h) = 350mm$ Bending reinforcement Design moment Total ultimate load(w) = $1.4Gk + 1.6Qk$ $= 1.4 * 381.39 + 1.6 * 167.09 \quad w = 801.29KN$ Earth pressure Ps=w/plan area of the base $= 801.29/3.24 \quad Ps = 247.31KN/m2$ Maximum design moment occurs at face of column $\frac{(Ps \times l^2)}{2} = \frac{247.31 \times 0.75}{2}$ M =69.56KN L=(1.8-0.3)/2 =0.75mm Ultimate moment $Mu = 0.156 * fcu * b * d2$ $Effective\ depth\ d = h - c - \emptyset \quad = 350 - 50 - 12$ =288mm Mu=0.156*30*1000*288^2 Mu=388.178KNm Mu=388.178KNm > M=69.56KN Main steel $K = Mu/f_{cu} \times b \times d^2$ $= 69.56 * 10^6/30 * 1000 * 2882$ $= 0.028$ K<K'=0.156 single reinforcement	$c = 50mm$ N=548.48KN Plan area A=3.24m^2 w = 801.29KN Ps =247.31KN/m^2 M =69.56KN No compression reinforcement required

	$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$	
	$Z = 288(0.5 + \sqrt{(0.25} - \frac{0.028}{0.9})) \leq 0.95 * 288$	
	278.7 < 273.6	
	$A_s = \frac{Mu}{0.95 f_y Z}$	Z=273.6
Table 3.22 chanakya	$= 69.56 * 10^\wedge 6/0.95 * 460 * 273.6$ $= 581.78 mm2$	As=581.78mm2 Aprov=646mm2 T12@175mm
	$\frac{0.13 * 1000 * 350}{100} < 581.78$ $455 < 581.78$	
	Critical shear stresses Critical perimeter, Pcrit=column perimeter+8*1.5d = $4 * 300 + 8 * 1.5 * 288 \quad 4656mm$	Ok
	Area within perimeter =(300+3d)^2 (300+3*288)^2	
	Ultimate punching force V=load on shear area =$Ps * (A - Aper)$ $= 247.31 * (3.24 - 1.355)$ =466.18KN	Area within perimeter = 1.355m^2
	Design punching shear stress $V = V/(Pcrit\ d)$ $=466.18 * 10^\wedge 3/4656 * 288$ $= 0.347N/mm2$	Punching failure is unlikely and 350mm depth of slab is acceptable.
Table 3.11	$\frac{100As}{b\ d} = \frac{100 * 646}{1000 * 288} \quad V = 0.224$ Design concrete shear stress , $Vc = 0.43$ $Since\ Vc = 0.43 > V = 0.47$	No shear reinforcement required
	<u>Face shear</u> Maximum shear stress (Vmax) occurs at face of column Hence $Vmax = w/(column\ perimeter * d$ $= \frac{801.29 * 10^3}{4 * 300 * 288} \quad = 2.32$	
	<u>Transverse shear.</u> Ultimate shear force (V)= load on shaded area $= Ps * area \quad = 2447.31 * (0.468 * 1.8)$ $= 208.33KN$ $V = v/(b\ d) \quad = (208.33 * 10^3)/(1.8 * 10^3 * 288)$ $v = 0.40 < Vc = 0.43$	

	<u>DESIGN OF THE STAIRCASE</u> **Specifications** Dead load $Gk = 0.225 * 24 \quad = 5.4KN/m2$ $Imposed\ load\ Qk = 4.0KN$ Design load= $1.4Gk + 1.6Qk \ = 1.4 * 5.4 + 1.6 * 4.0$ $= 13.96KN/M2$ $Total\ load \ = 13.96 * 1.2 * 3.35$ $= 56.12KN$ $M = Fl/8 \quad = 56.12 * 3.35/8 \quad = 23.50KNm$ **Reinforcement at the staircase** $K = Mu/f_{cu} \times b \times d^2$ $= 23.50 * 10\text{^}6/25 * 1000 * 1242$ $= 0.06$ K<K'=0.156 single reinforcement $Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$ $Z = 124(0.5 + \sqrt{(0.25} - \frac{0.06}{0.9})) \leq 0.95 * 124$ 115< 117.8 $A_s = \frac{Mu}{0.95 f_y Z}$ $= 23.50 * 10\text{^}6/0.95 * 460 * 115$ $= 467.6mm2$ $Minimum\ area =$ $\frac{0.13 * 1000 * 124}{100} < 467.6$ $161.2 < 467.6$	No shear reinforcement required K=0.06 Compression reinforcement not required Z=115 As=467.6mm2 Approv=566mm2 5T12@200mm cc Ok

	Shear check Design shear stress $V = \frac{v}{bd} \quad \leq 0.8\sqrt{fcu}$ $$V = 28.06 * 1000/1000 * 124$$ $$V = 0.226N/mm$$ $$\frac{100As}{bd} = \frac{100 * 566}{1000 * 124}$$ $$= 0.456 \qquad Vc = 0.67$$ $$V\ c\ =\ 0.67\ >\ v\ =\ 0.456$$	No shear reinforcements required
	Deflection at the mid span Basic l/d ratio=26 $$MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$$ $$fsd = 2/3fy \times As/Aprov \times 1/\beta_b$$ $$= 2/3 * 460 * 467.6/566 * 1$$ $$= 253.35N/mm$$ Mu/bd^2=$23.50 * 10^\wedge 6/1000 * 124^\wedge 2 \quad = 1.528$ Fy=460 from table 3.10 MFT=1.14 <2 Allowable l/d ratio=Basic l/d ratio*MFT =26*1.14 =29.64 Actual l/d 3350/124 =27.016 Allowable l/d > actual l/d	Deflection is satisfactory
	RAMP DESIGN **Specifications**	

	Dead load $Gk = 0.150 * 24 \quad = 3.6KN/m2$	
	$Imposed\ load\ Qk = 4.0KN$	
	Total load $F = 1.4Gk + 1.6Qk \quad = 1.4 * 3.6 + 1.6 * 4.0$	
	$= 11.44 * 1.2 * 3.35$	
	$= 45.99KN$	
	$M = Fl/8 \quad = 45.99 * 3.35/8 \quad = 19.26KNm$	
	Reinforcements at the ramp	
	$K = Mu/f_{cu} \times b \times d^2$	
	$= 19.26 * 10^6/25 * 1000 * 124^2$	
	$= 0.05$	
	K<K'=0.156 single reinforcement	
	$Z = d\left(0.5 + \sqrt{0.25 - \frac{k}{0.9}}\right) \leq 0.95d$	
	$Z = 124(0.5 + \sqrt{(0.25 - \frac{0.05}{0.9}})) \leq 0.95 * 124$	
	116.68< 117.8	Z=116.68
	$A_s = \frac{Mu}{0.95 f_y Z}$	
	$= 19.26 * 10^6/0.95 * 460 * 116.68$	Approv=452mm2
	$= 377.73mm2$	4T12@250mm
	$Minimum\ area = \frac{0.13 * 1000 * 124}{100} < 467.6$	
	$161.2 < 377.73$	
	<u>Shear check</u>	
	Design shear stress $V = \frac{v}{bd} \quad \leq 0.8\sqrt{fcu}$	
	$V = 19.26 * 1000/1000 * 124$	
	$V = 0.1553N/mm$	

	$$\frac{100As}{bd} = \frac{100 * 377.73}{1000 * 124}$$ $$= 0.305 \qquad Vc = 0.67$$ $$V\,c = 0.67 > v = 0.1553$$	no shear reinforcement required
	Deflection at the mid span Basic l/d ratio=26 $$MFT = 0.55 + (477 - fsd)/120(0.9 + \frac{Mu}{bd^2})$$ $$fsd = 2/3fy \times As/Aprov \times 1/\beta_b$$ $$= 2/3 * 460 * 377.73/452 * 1$$ $$= 256.28N/mm$$ Mu/bd^2=19.26*10^6/1000*124^2 =1.253 MFT=1.14<2 Allowable l/d ratio=Basic l/d ratio*MFT =26*1.14 =29.64 Actual l/d 3350/124 =27.016 Allowable l/d > actual l/d	Deflection is satisfactory

CHAPTER FIVE : CONCLUSIONS AND RECOMMANDATIONS

5.1 Conclusions

To provide adequate classrooms and other academic facilities, a 2 storeyed structure was designed by:-

- Establishing the design loads of the proposed structure which included both live and dead loads. The soil bearing capacity was assumed to aid the design. The dead loads were calculated basing on BS8110; Structural use of concrete while the live/imposed loads were obtained from BS6399 part 1 & 3.
- Analyzing structural members such as beams, columns, slabs and footings. This was done by manual calculations using BS 8110-1:1997
- Design of structural members. The obtained moments and shear forces from the analysis were used for structural design where both steel and concrete are the main components of these members. A concrete grade C25 and C30 were used in the design while steel of tensile strength 460 was used.
- Carrying out Structural detailing. After obtaining the required steel in the structural components, the next step was to draft the structural details of the respective members. This was also accompanied with the foundation layout to ease execution of setting out on site.
- And finally documentation to enable implementation of the project.
- Wind loads were not considered because its crucial for high rise buildings and since Arua is not in the seismic region, earthquake designs were ignored.

4.2 Recommendations

Prior to the implementation of the project, structural drawings shall be followed accordingly and no change shall be allowed without the designers consent. The sizes of structural members and reinforcement areas must be provided accordingly as shown in the structural details.

However because of time accompanied with the complexity of the project, not all the structural members were designed. A representative sample of one column that carried much loads were designed and also one base and at this point we recommend a further study to whoever is interested to design the rest of the members. Slabs for all the floors were typical (similar) so detailing was the same.

References

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3. BS 6399: *Design loading for buildings*; Part 1: Code of practice for dead and imposed loads, 1996; Part 2: *Code of practice for wind loads*, 1997; Part 3*: Code of practice for imposed roof loads*, 1988. London.
4. Bengt H Fellenius, (1992), *Design of reinforced concrete structure*, Volume 2.
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6. Chanakya (2009**).** *Design of structural elements*. USA: Spon press.
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APPENDICES
APPENDIX I: BUDGET FOR THE PROJECT

ITEM	COST
COMMUNICATION	20,000
INTERNET	50,000
PRINTING	80,000
LAPTOP MAINTAINANCE	50,000
TRANSPORT	50,000
STATIONARY	1,000
Others	60,000
TOTAL	311,000

APPENDIX II: SCHEDULE OF THE WORK.

The project was executed following the time schedule below

	MONTHS	NOV	DEC	JAN	JUNE	JULY	AUGUST
ACTVITY	Reconnaissance Survey						
	Proposal Writing.						
	Structure Analysis						
	Structure Design						
	Preparing structural drawings						
	Report Writing						

APPENDIX III: ANALYSIS FOR THE SLAB RIB

joint		B		C		
members	AB	BA	BC	CB	CD	DC
loadings	4.8		4.8		4.8	
length	4		2.96		2.6	
stiffness	0.33		0.34		0.51	
distribution factors		0.50	0.50	0.40	0.60	
fixed en moments	0	-9.60	3.50	-3.50	2.70	0
distribution		3.03	3.07	0.32	0.48	
carry over			0.16	1.53		
distribution		-0.08	-0.08	-0.61	-0.92	
carry over			-0.30	-0.04		
distribution		0.15	0.15	0.02	0.02	
carry over			0.01	0.08		
distribution		0.00	0.00	-0.03	-0.05	
carry over			-0.02	0.00		
distribution		0.01	0.01	0.00	0.00	
carry over			0.00	0.00		
distribution		0.00	0.00	0.00	0.00	
Total	0.00	-6.50	6.50	-2.24	2.24	0.00

Reactions, maximum moment, where maximum moment occurs

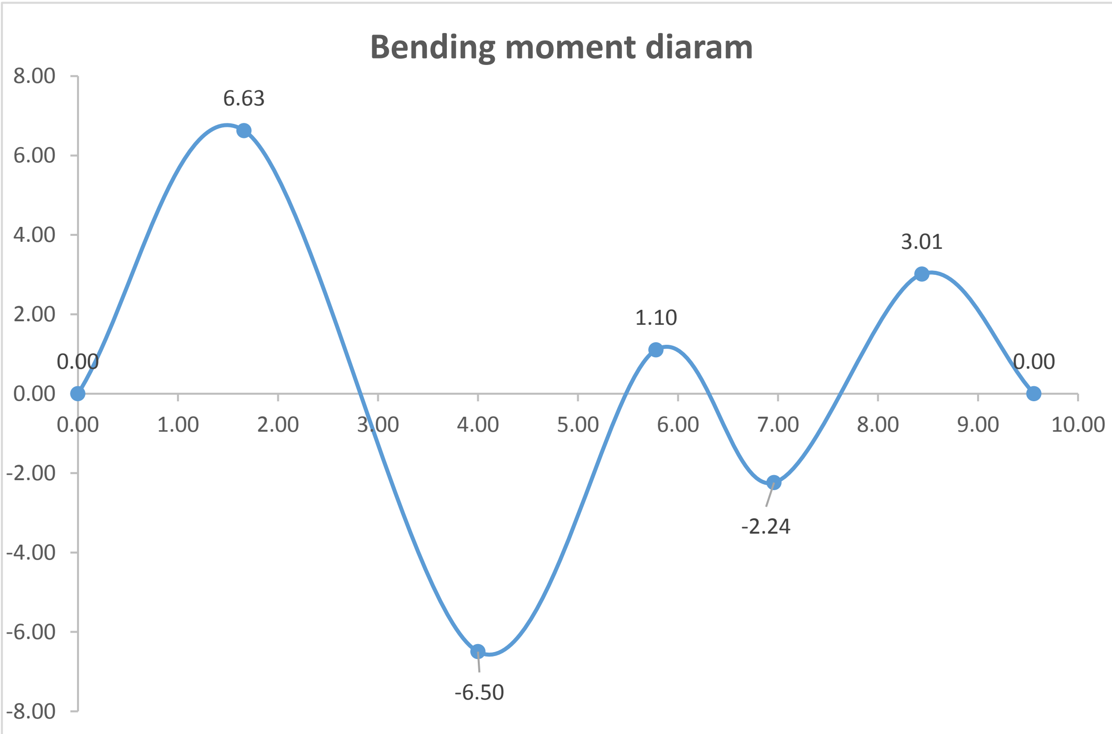
reactions	7.98	11.22	8.54	5.67	7.10	5.38
where max moment occurs	1.66		1.78		1.48	
max moment	6.63		1.10		3.01	

moments against distance

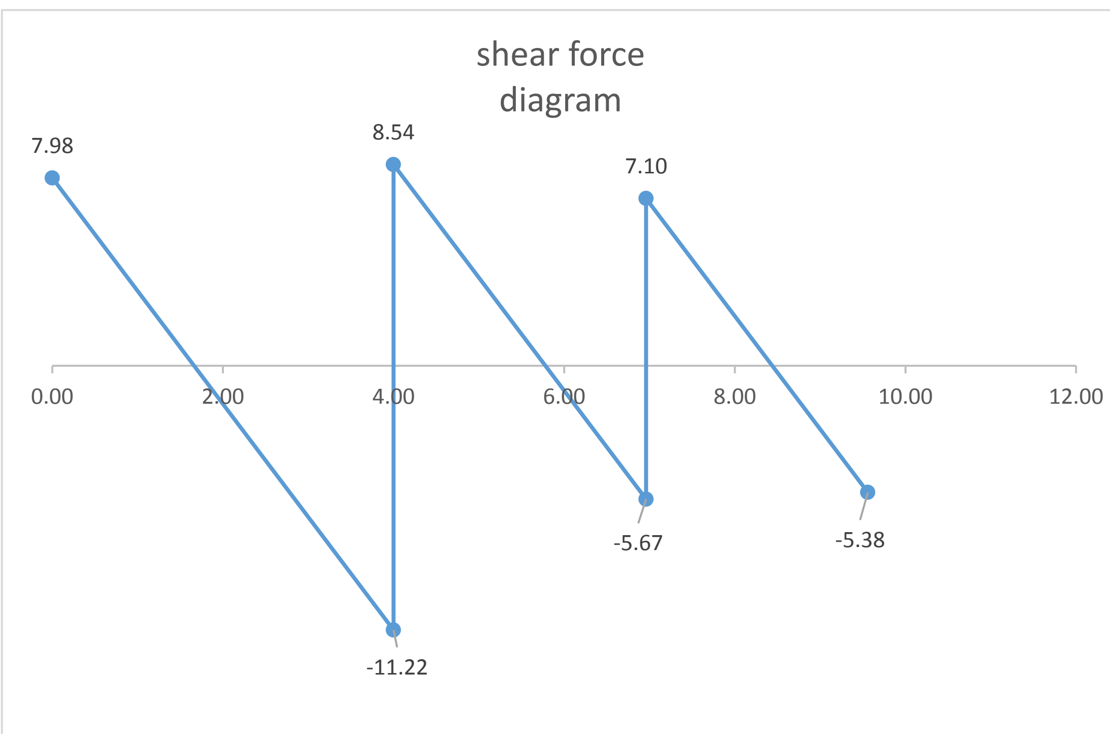
x	0.00	1.66	4	5.78	6.96	8.44	9.56
m	0.00	6.63	-6.50	1.10	-2.24	3.01	0.00

shear against distance

x	0.00	4.00	4.00	6.96	6.96	9.56
v	7.98	-11.22	8.54	-5.67	7.10	-5.38



Bending moment diaram
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
-8.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
6.63
0.00
-6.50
1.10
-2.24
3.01
0.00



shear force diagram
7.98
8.54
7.10
0.00
2.00
4.00
6.00
8.00
10.00
12.00
-5.67
-5.38
-11.22

Analysis for the beam H-H

joint		B		C		
members	AB	BA	BC	CB	CD	DC
loadings	16.38		16.38		16.38	
length	4		2.96		2.6	
stiffness	0.33		0.34		0.51	
distribution factors		0.50	0.50	0.40	0.60	
fixed en moments	0	-32.76	11.96	-11.96	9.23	0
distribution		10.33	10.47	1.09	1.65	
carry over			0.54	5.24		
distribution		-0.27	-0.27	-2.08	-3.16	
carry over			-1.04	-0.14		
distribution		0.52	0.52	0.05	0.08	
carry over			0.03	0.26		
distribution		-0.01	-0.01	-0.10	-0.16	
carry over			-0.05	-0.01		
distribution		0.03	0.03	0.00	0.00	
carry over			0.00	0.01		
distribution		0.00	0.00	-0.01	-0.01	
Total	0.00	-22.17	22.17	-7.64	7.64	0.00

Reactions, maximum moment, where maximum moment occurs

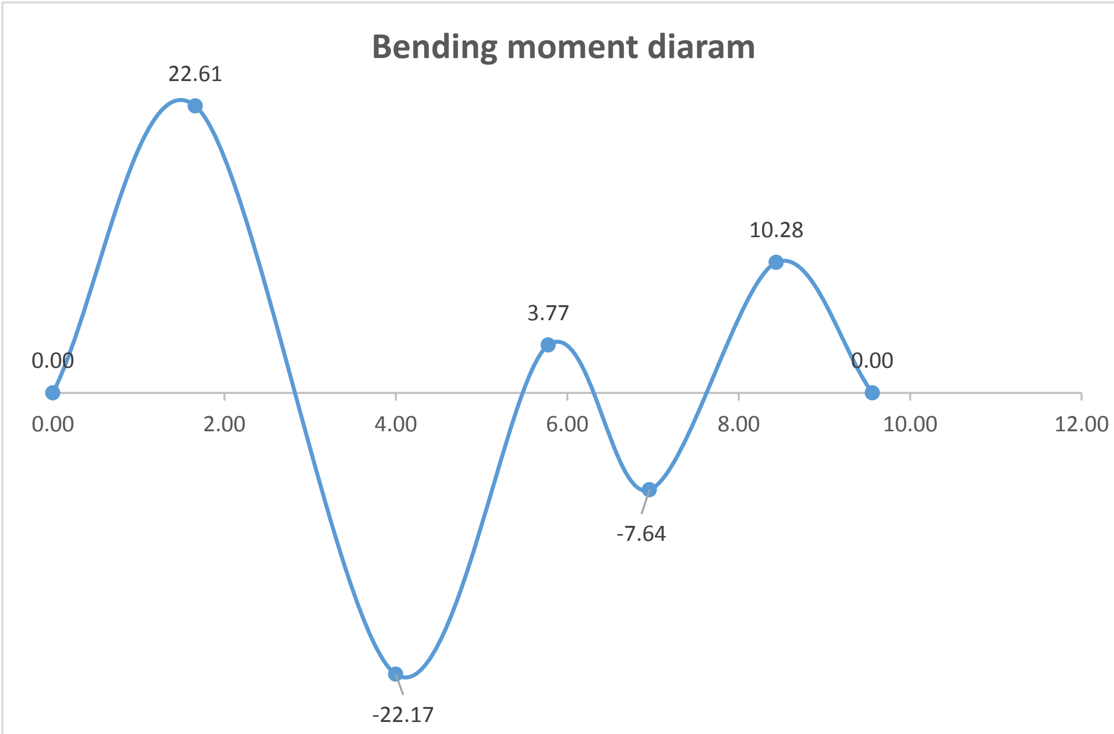
reactions	27.22	38.30	29.15	19.33	24.23	18.36
where max moment occurs	1.66		1.78		1.48	
max moment	22.61		3.77		10.28	

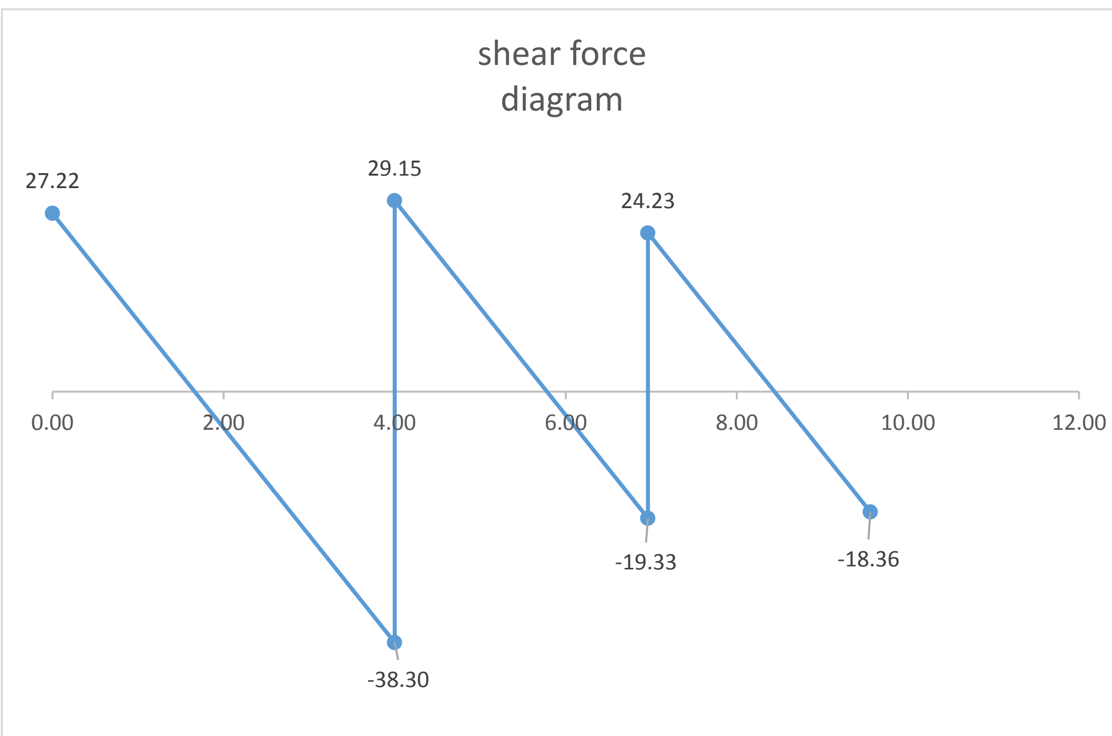
moments against distance

x	0.00	1.66	4	5.78	6.96	8.44	9.56
m	0.00	22.61	-22.17	3.77	-7.64	10.28	0.00

shear against distance

x	0.00	4.00	4.00	6.96	6.96	9.56
v	27.22	-38.30	29.15	-19.33	24.23	-18.36


Bending moment diaram
22.61
10.28
3.77
0.00
0.00
-7.64
-22.17
0.00
2.00
4.00
6.00
8.00
10.00
12.00


shear force diagram
27.22
29.15
24.23
0.00
2.00
4.00
6.00
8.00
10.00
12.00
-19.33
-18.36
-38.30