EVALUATION OF THE RELATIVE DIFFERENCE BETWEEN THE PROVISIONS OF BS 8110 AND EUROCODES FOR THE DESIGN OF REINFORCED CONCRETE STRUCTURES IN NIGERIA

By

AMOO, Aishat Yetunde

(CVE/11/0367)

A project report submitted to the Department of Civil Engineering, Federal University Oye -Ekiti in partial fulfillment of the requirement for the award of the B. Eng. (Hons) in Civil Engineering.

Department of Civil Engineering

Faculty of Engineering

SEPTEMBER, 2016

Abstract

The conventional method of design of reinforced concrete structures commonly used in Nigeria is the BS 8110 method. This project shows a comparative study of BS 8110 and Eurocode 2 for the design of reinforced concrete elements of a building with the aim of determining which of the two codes provides the most economic design is carried out using manual calculations. A library complex of three storey building was selected and designed taking into account dead and live loads and assuming all spans to be loaded with $4KN/m^2$ as the imposed load. The result of the analysis was used to design the structural elements based on both codes using manual calculations. The percentage difference between the areas of steel required by the two codes was calculated with the BS 8110 code results as the control values and the total weight of steel required. It was found that Eurocodes are more economical than the BS codes.

ACKNOWLEDGEMENT

I am grateful to the Almighty God for his inspiration, guidance and strength throughout the course of this work.

I would like to express my profound gratitude to my supervisors in the person of Dr. (Engr.) Christopher Fapohunda and Engr. Tochukwu Onuorah, for their invaluable support, guidance and wisdom. I really and sincerely say a big thank you for your warm gesture and efforts

To my beloved parents, Alhaji O. M. Amoo and Mrs I. I. Amoo, I can't find the right words to express how sincere i am for the inestimable support and love shown to me during this period of my life. You were there for me when i needed you the most.

To my siblings: Yusuf Nafisat Olajumoke, Amoo Raliat Ajoke, Amoo Mohammed Tomiwa and Amoo Basit Kolawole, i am very proud of you all, thank you all for your support, patience, perseverance and tolerance. May god bless you all

To all my colleagues, especially Emate Victor E., Josu Gigemi M., Olojo-Kosoko Adebowale and Adigun Olumide, who in one way or the other contributed to the success of this work, I say thank you.

DEDICATION

This work is dedicated to Almighty God.

CERTIFICATION

This is to certify that AMOO Aishat Yetunde, with matric no CVE/11/0367, in the department of Civil engineering, Faculty of Engineering, Federal University Oye-Ekiti, Ekiti state carried out this project.

.... 27-9-2016
Dr. C.A. Fapohunda **Date**
Project supervisor

....
Engr. Tochukwu Onuorah **Date**
Co. Supervisor

....
Prof. J.B. Adeyeri **Date**
Head of Department

Table of contents

List of abbreviations

b	width of section
d	effective depth of the tension reinforcement
h	overall depth of section
x	depth to neutral axis
z	lever arm
l_{eff}	effective span of beams and slabs
l_n	clear distance between the faces of the supports
a_1, a_2	allowance at supports used for calculating the effective span of a member
c_{nom}	nominal cover to reinforcement
d_2	effective depth to compression reinforcement
g_k, G_k	characteristic permanent action
q_k, Q_k	characteristic variable action
w_k, W_k	characteristic wind load
f_{ck}	characteristic compressive cylinder strength of concrete at 28 day
f_{yk}	characteristic yield strength of reinforcement
f_{cd}	design compressive strength of concrete
f_{yd}	design yield strength of reinforcement
F_d	design action
F_k	characteristic action
F_{rep}	representative action
X_k	characteristic strength
X_d	design strength
γ_c	partial factor for concrete
γ_f, γ_F	partial factor for actions
$\gamma_{G,j}$	partial safety factor for permanent action j for persistent and transient design situations
γ_{GA}	partial safety factor for permanent action j for accidental design situations
$\gamma_{Q,i}$	partial safety factor for variable action i

γ_P	partial safety factor for prestressing force
γ_M	partial safety factor for the material property, including model uncertainty.
K	coefficient given by $M/f_{cu}bd^2$
K'	coefficient given by $M_u/f_{cu}bd^2 = 0.156$ when redistribution does not exceed 10 percent design ultimate moment
M	design ultimate moment
M_u	design ultimate moment of resistance
A_s	area of tension reinforcement
A'_s	area of compression reinforcement
V	design shear force due to ultimate loads
V_c	design shear stress
v	design concrete shear stress
A_{sv}	total cross-sectional area of shear reinforcement
N	design ultimate axial load
A_c	net cross-sectional area of concrete in a column
A_{sc}	area of longitudinal reinforcement

List of Plates

List of tables

CHAPTER ONE

INTRODUCTION

1.1 Background

The primary requirement of all structural design is to achieve an acceptable probability that a structure will perform satisfactorily during its design life. The structural design of most structures is based on national or international codes of practice. These codes serve as a guide to the structural engineer in the general appraisal of the overall structural scheme, detailed analysis and design. Codes are what professionals work with and are often the founding document for a profession. A code is a document covering a system of laws. Codes of practice are basically documents which serve as guides drawn up by experienced engineers and a team of professionals, and they provide a general arrangement for addressing issues of safety and serviceability in structural engineering design. Building code is a document containing standardized requirements which specify the minimum acceptable limit of safety for structures. These codes are based on the experience of engineers, experimental work and specific conditions and behaviour. Codes protect buildings against various risks like fire, structural collapse and amenity issues such as ventilation, lighting, sanitation, sound insulation and dampness. Moreover, codes are significant tools for achieving society's goals such as sustainability and energy efficiency, addressing all the aspects of construction such as: safe exits, electrical, plumbing, seismic design, structural integrity and correct use of the construction materials. Building codes classify structures by applying and using various standards, as an example, schools and office buildings are in separate occupancy categories with various performance requirements (Building Code , 2013)

1.1.1 History of Building Codes

First code was written by King Hammurabi (3000 BC) during early civilization in Babel.(Al- Ansari, Al-Taie, & Knutsson, 2014)

The Code of Hammurabi is a well-preserved ancient law code, created about 1760 BC in ancient Babylon. It was enacted by the sixth Babylonian king, Hammurabi. Only one example of the Code survives today, inscribed on a basalt stone stele. Originally, several stelae would have been displayed in temples around the empire. The text has been broken down by translators into 282 laws, but this division is arbitrary, since the original text contains no divisional markers

1. If a builder build a house for a man and complete it, (that man) shall give him two shekels of silver per SAR of house as his wage.

2. If a builder build a house for a man and do not make its construction firm, and the house which he has built collapse and cause the death of the owner of the house, that builder shall be put to death.

3. If it causes the death of a son of the owner of the house, they shall put to death a son of that builder.

4. If it causes the death of a slave of the owner of the house, he shall give to the owner of the house a slave of equal value.

5. If it destroys property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed at his own expense.

6. If a builder build a house for a man and do not make its construction meet the requirements and a wall fall in, that builder shall strengthen that wall at his own expense.

Many of the restrictions encountered in building design are imposed by legal regulations. The most important ones for structural engineers are building codes, which represent a set of regulations regarding:

1. principles of structural design
2. guidance in evaluation of loads on structures
3. specific design provisions for different type of structures (steel structures, reinforced concrete structures, foundations, etc.) and building components (electrical system, HVAC, plumbing, etc

In general, building code requirements are the minimum needed for public protection. Often, however, architects and engineers must design more conservatively, to meet the client's needs, produce a more efficient building system, or take into account conditions not covered fully by code provisions.

Emperor Nero developed master plan for idealized Roman city after burning in 64 ADs, because the constructions of Roma public buildings were of bad construction quality. In addition, the distances between buildings were cramped and they had poor sanitation. After burning, the constructions were done according to the master plan of Nero; principles regarding fire resistance and sanitation (Yeager, 2013). Fires broke out near the Tower of London's city in 1666 ADs, because the city was crowded with tightly spaced buildings and raw sewage flowing through open drains. Parliament started writing what is known as London Building Act for two years after that fire. The documents established the regulations for the buildings of London's city only (Yeager, 2013).

Chicago city faced huge fire for two days during 1871 ADs. More than a quarter of the city buildings were destroyed. For that reason, most of the insurance companies threatened to leave unless developments should be done to the regulations of buildings. In 1875, rules for regulations of buildings and fire prevention were enacted. On April 18, 1906, an earthquake hit San Francisco city and left it in ruins. Some parts of the city were destroyed by the fires that broke out after the earthquake. The scientific community was gathered to observe what had happened and they formed the building code organizations that still exist today. Organizations since then had been studied various structures after each earthquake to check their ability to withstand the events (Yeager, 2013). In view of these disasters the regulations of constructions were improved and collected in code

Post-war national codes, such as CP 114 (reinforced concrete) and CP 115 (prestressed concrete) are based on permissible stress method with its fairly arbitrary 'factors of safety' used in the design of structural elements that gave designers a false sense of security. Permissible stress method was widely held to be unsatisfactory and led to the formation, at an international meeting in [illegible] of the Federation International de la Precontraint (FIP). The [illegible] du Beton (CEB) was formed with the objective of international technical [illegible] in 1953. The first CEB FIP model code for the design of structural [illegible] published in 15 languages in 1964 with a second [illegible] CEB united to form FIB in 1998 and its ground-breaking work [illegible] CEB code introduced Limit State principles to the engineering [illegible] to ensure that "the chance of each limit state being reached is substantially constant for all members in a structure, is appropriate for each limit state and that consequently there is an adequate degree of safety against the structure being unfit for use. The two types of limit states are defined which include ultimate limit states and serviceability limit states. The ultimate limit states are concerned with the collapse or other forms of structural failure which might endanger people's safety while serviceability limit states are concerned with states beyond which specified service requirements are no longer met. Limit state codes can take account of the different variations of various materials and loads, and also the consequences of failure. The theoretical aim of the limit state process is that working load and ultimate performance can be verified by the use of the theories of probability. Unfortunately, structural failure results more often from human error than from the intersection of the extreme ends of the frequency distributions for load and strength. Attempts at deriving fully probabilistic methods of structural design all ultimately rely upon calibration, comparing the theoretical conclusions with existing design methods and practice. (Taylor & Burgoyne, 2008)

1.2 How Standards are Written

Essentially they are written by experts, in the past entirely voluntarily, but increasingly now standards are written by experts supported by a combination of industry and public finance. British Standards are published by the British Standards Institute (BSI) itself a venerable institution with over 100 years of history. (Taylor & Burgoyne, 2008)

Although BS 8110 has superseded its immediate predecessor CP 110 (the change of designation from a Code of Practice to a British Standard does not indicate any change of status) which had been in current use for 13 years, an earlier document still, CP 114, is still valid. BS 8110 does not, in essence, differ greatly from CP 110. Perhaps the most obvious change is the overall arrangement of material. Whereas CP 110 incorporated the entire text in Part 1, with the reinforced concrete design charts more usually required (i.e. slabs, beams and rectangular columns) forming Part 2 and the others Part 3, the arrangement in BS 8110 is that Part 1 embodies the 'code of practice for design and construction'. Part 2 covers 'special circumstances' and Part 3 incorporates similar charts to those forming Part 2 of CP 110. There are, as yet, no equivalents to the charts forming Part 3 of CP 110. The material included in Part 2 provides information on rigorous serviceability calculations for cracking and deflection, more comprehensive treatment of fire resistance, and so on. It could be argued that mute logical arrangements of this material would be either to keep all that relating to reinforced concrete design and construction together in Part 1 with that relating to prestressed and composite construction forming Part 2, or to separate the material relating to design and detailing from that dealing with specifications and workmanship. (Reynolds & Steedman, 1999)

This code covers the fields of CP 110 and encompasses the structural use of reinforced and pre-stressed concrete both cast in situ and precast. Although there are no major changes in principle from the previous edition, the text has largely been rewritten with alterations in the order and arrangement of topics. The redrafting and alterations have been made in the light of experience of the practical convenience in using CP 110 they have also been made to meet the criticism of engineers preferring the form of CP 114, in this respect sections two to five have been rewritten with shorter clauses, avoiding as much as possible lengthy paragraphs dealing with matters that could be broken down into separate sub clauses, to make specific references easier to identify. Consideration had been given to including the load factor method which had been introduced into CP114 in 1957. The basic approach to design for safety in all codes is the following. A level of loading is assessed that leads to the worst conditions in the structure which can reasonably be expected to occur in practice. This is commonly referred to as "working" or "service" load stresses. A substantial margin of strength is required between this working condition and the strength of structure which the designer aims to provide. This margin is necessary to take account of uncertainties in the loading, the strength of the materials, the construction process and in the current state of knowledge of structural behaviour. It is in the way in which this margin is provided that the elastic, ultimate load and limit state methods of design differ. The elastic (or permissible stress) approach aims at ensuring that the working stresses do not exceed a set of defined permissible stresses which are obtained by reducing the material strengths by a safety factor and it aims to ensure that the strength of the structure, calculated using the expected actual materials strength, is sufficient to support this ultimate loading. It might appear that these two approaches are, in effect, identical but, in fact, this is only strictly so for materials that are fully elastic up to failure. Nevertheless, by appropriate choice of coefficients in the various design equations, the two methods can be made to give very similar results for most common types of structure. In drafting CP 114: 1957 it was felt that if the two methods were to be expressly permitted in one document then the strict interpretation of load factor theory would have to be modified in order to avoid the confusion of having different design loads and stresses specified for the elastic method with the difference that the plastic stress strain relations were to be assumed in place of Hooke"s law. This has led ever since to a confusion in the minds of designers as to what their calculations were actually predicting. The limit state method of designs introduced in CP 110 in 1972, develops the logic of load factor design rather further, instead of allowance for all the uncertainties being compiled together into a single, global, safety factor, a set of partial safety factors are defined, one for each material and type of load. The relative values of these reflect an assessment of the relative uncertainty associated with the various loads and materials strengths. As well as treating uncertainty more logically, the partial safety factor approach when used for structures subjected simultaneously to different types of loading (for example, vertical load and wind load) where a critical design condition arises when one loading is at its maximum value and the other at its minimum value. The global factor approach automatically increases both the maximum and the minimum load giving a less critical condition than if only the maximum load is increased. BS 8110 covers all the principles of design in CP 110 but some modifications have been made. BS 8110 has been reviewed twice since its inception. Recently the Euro codes were introduced and have been used in Europe as well as in UK. The standard was developed to bring reinforced concrete design up to date (Ajis 2012)

The conventional method of design of concrete structures in Nigeria is based on BS codes. Euro codes have been used in other countries especially in Europe. Nowadays, civil engineers take initiative to introduce the design of concrete structures using Euro codes.

In several countries the British standard has been employed almost exclusively with the exception of variation of nationally determined parameters. In the last three decades however, an alternative set of codes to replace the British and other

European national standards has been developed termed the Eurocodes (ECs). The Eurocodes are a new set of European structural design codes for building and civil engineering works. The Eurocodes have been introduced as part of the wider European harmonization process and not just simply to directly replace any national codes. In the design of concrete structures, the relevant parts of the codes are EC0: Basis of structural design, EC1: Actions on structures and EC2: Design of concrete structures.

The aims of these Eurocodes are collectively to provide common design criteria and methods to fulfil the specified requirements for mechanical resistance, stability and resistance to fire, including aspects of durability and economy. Furthermore they provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction materials. Nowadays, Eurocodes are being introduced and applied for design of concrete structures but still not yet widely used in Nigeria. The Euro codes are intended to be mandatory for European public works and likely to become the standard for the private sectors both in Europe and the world at large. Prior to the emergence of the Euro codes, the British standard codes of practice has been in use to serve the same purpose the Euro codes were intended and it begs a lot of questions as to what the differences are in construction infrastructure. (Nwofor, Sule, & Eme, 2015)

1.2 Problem Statement

The civil engineering field is a profession with ever evolving technology and transitions from one practice to another kind of practice, so therefore for a civil engineer to stay relevant he must keep abreast of the changes and transitions in the field. With the recent enactment of the Euro code of structural design, there is a need for the engineers, especially in Nigeria, to become familiar with these advancements so as to be able to compete with their colleagues in the international market and even stay up-to-date. By comparing structural design using the Euro code and the BS codes (which is what this research is about will enable us to have a better understanding of the new features of the Euro code, similarities and the differences between the two codes and these have a subsequent effect on our ability to comprehend and work with the Euro code and even do better with the BS codes. This research study has recognised a problem, which is the lack of an in-depth knowledge into the Euro code and it is proposed to solve it by the comparative analysis of the structural design of a library complex using the BS codes and Euro code.

1.3 Aim and Objectives

1.3.1 Aim

The aim of this project is to compare the analysis of BS 8110 and Euro code 2 as methods of design using a library complex

1.3.2 Objectives

a) To analyse and identify the fundamental difference between BS 8110 and Eurocode 2

b) To analyse and design the structural elements to BS 8110 and Eurocode 2

c) Production of arrangement and detail drawings

d) To make comparative cost analysis of the results obtained by using the conventional BS code method and Eurocode method

e) To make recommendations based on the outcome of the analysis

1.4 Scope of Study

The scope of this work is to analyse and design a library complex based on BS 8110 and Euro code 2, make comparative analysis on the results obtained and exposition of the various similarities and differences between the two codes in which structural engineers may choose to adopt based on economic and safety considerations.

1.5 Limitation of Study

The scope of this project will be limited to manual design only. No software will be used for the analysis.

CHAPTER TWO

LITERATURE REVIEW

2.1 Code Of Practice

Purpose of Building Codes

The purpose of a building code is to establish minimum requirements necessary to protect public health, safety and welfare in the built environment. Model building codes provide protection from tragedy caused by fire, structural collapse and general deterioration. The primary application of a building code is to regulate new construction. Building codes usually only apply to an existing building if the building undergoes reconstruction, rehabilitation or alteration, or if the occupancy of the existing building changes to a new occupancy as defined by the building code (Council, 2005). Safe buildings are achieved through proper design and construction practices along with a code administration program that ensures compliance. Model codes keep construction costs down by establishing uniformity in the construction industry. This uniformity permits building and materials manufacturers to do business on a larger scale i.e. state-wide, regionally, nationally or internationally. Larger scale allows cost savings to be passed on to the consumer. Codes also help protect real estate investments by providing a minimum level of construction quality and safety (Council, 2005).

Structural design is a major activity of the profession of structural engineering. Structural design is an art wherein the design engineer participates in the creation of a structure, such as a building, a bridge, a tunnel, an antenna, etc. The contribution of the structural engineer is recognized and appreciated by the general public to various degrees, depending on the visibility of the skeleton or the shell of the edifice (Galambos).

The structural design engineers have many paints and brushes in their palettes: mathematics, solid and fluid mechanics, physics, chemistry, etc., i.e. the hard sciences taught in their engineering education; experience; judgement; concern for the common good of the society; consideration for the environment and many more attributes acquired in professional practice. There are some severe restrictions including economics, the needs of all the other members of the design team and the demands posed by the requirements of the structural design standards called codes (Galambos).

Structural design codes exist to insure uniformity of design criteria across structural types and across a region, and to permit control of structural designs by authorities whose duty is to ensure public safety (Galambos).

Structural design codes are mainly directed towards the design of safe new structures in their final occupied state. There is still a lack of comprehensive design standards for a number of design assignments that structural engineers face with increasing frequency. Such challenges as are posed by the following activities have requirements that are different from those that govern the new structures. Conventional practice is to apply the standards for new structures for these altered design conditions. The statistics and the reliability requirements for these situations are different when much is known about the past history of the site, the load history, the structure, etc. Such challenges requiring codification based on available experience and research are,

i. Refurbishment of structures for new uses or occupancies
ii. Repair and strengthening of structures after damage due to catastrophic events
iii. Redesign or re-evaluation due to changed circumstances in occupancy, use or loading. This could be, for instance, increases of demand from higher traffic loads, new insights gained about the nature of a type of demand, or different wind loading on a structure after newer adjacent buildings change the wind climate, etc.
iv. Use of recycled structural elements in new constructions
v. Preservation of heritage buildings (Galambos)

2.2 British Standard (BS 8110)

BS 8110: Part 1:1997 was been published on the orders of Civil engineering and Building Structures Committee Standards. Code of practice is provided with BS 8110:Part2:1985 to replace the CP 110: Part 1:1972. Other than BS 8110, there are other BS codes that have been published in connection with the construction industry (Ajis, 2012).

i. BS 1192 Construction Drawing Practice
ii. BS 1363 Mains Power Plugs and Sockets
iii. BS 1852 Resistor Value Coding
iv. BS 5750 Quality Management
v. BS 6930 Site Investigations
vi. BS 5950 Steel Structure
vii. BS 6879 British Geocodes

In BS8110: Part 1, there consists 8 parts that are divided into several structural elements. Each part is as shown in Table 2.1:

Table 2.1: Parts of BS 8110 (Ajis, 2012)

Parts	General matters relating to the design of reinforced concrete design scope, definitions and related symbols.
Part 1	Design objectives and recommendations on areas such as basic design, structural design, analysis, loading and material properties.
Part 2	Design objectives and general recommendation
Part 3	Design detailing for reinforced concrete
Part 4	Design and detail for prestressed concrete
Part 5	Structural design and details of precast and composite construction
Part 6	Materials, specifications, concrete and construction
Part 7	Specification for the reinforcement
Part 8	Specifications and workmanship for prestressing tendons

BS 8110-1:1997 mentioned that British Standard has been provided to several sub-communities such as Association of Consulting Engineer, British Cement Association and several another communities under Civil Engineering and Building Structures Standard Committee (Ajis, 2012).

There are several elements that have been contained in BS 8110:1997:

Part 1: Code of Practice for Design and Construction

Part 2: Code of Practice for Special Circumstances.

Part 3: Design Chart for single reinforced beam, doubly reinforced beam and rectangular column (Ajis, 2012)

BS 8110: Part 1: 1985 has been provided under Civil Engineering and Building Structures Standard Committee. It produced together with BS8110: Part 2:1985 to replaced CP 110:Part 1:1972. Then, BS 8110: Part 1:1997 combined all the amendment issued to BS8110:Part 1:1985 as stated before:

a) Amendment No. 1 (AMD 5917) published on 31 May 1989.

b) Amendment No. 2 (AMD 6276) published on 22 December 1989.

c) Amendment No. 3 (AMD 7583) published on 15 March 1993.

d) A

mendment No. 4 (AMD 7973) published on 15 September 1993.

2.3 Eurocode 2

2.3.1 Background of Eurocode 2

European Code, or better known as Euro code was initiated by the Commission of European Communities as a standard structural design guide. It was intended to smooth the trading activities among the European countries. Euro code is separated by the use of different construction materials. Euro code 1 covers loading situations. Euro code 2 covers concrete construction. Eurocode 3 covers steel construction while Euro code 4 covers composite construction (Yusoff, 2015).

Eurocode underpins all structural design irrespective of the material construction. It establishes principle and requirements for safety, serviceability and durability of structures. The Euro code uses a statistical approach to determine realistic values for actions that occur in combination with each other (Ajis, 2012).

From 2002 to 2007, there are 58 parts of the 10 Euro code have been published. CEN expected in year 2008 to 2010, most of the European national codes will be replaced by all the published Euro codes. Euro code will be used for construction work in Europe and is designed to help make the construction industry in Europe more competitive and improve the safety of the structure. In addition, Euro code has become the main reference material and guidance to the recent development and provides training to all users (Ajis, 2012).

2.3.2 Objectives of Eurocode

The aims of these ECs are collectively to provide common design criteria and methods to fulfil the specified requirements for mechanical resistance stability and resistance to fire, including aspects of durability and economy. Furthermore they provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products. They also facilitate the exchange of construction services between member states of the European community as well as serve as a common basis for research and development in the construction sector. In addition they improve the functioning of the single market by removing obstacles arising from nationally codified practices. They also improve the competitiveness of the European construction industry (The aims of the Eurocodes, 2008)

2.3.3 Stages of Developments and Layout of Eurocode

Each structural Euro code is established in a number of parts covering a range of applications. These documents are at various stages of developments. Eventually Euro codes will have the same legal standing as the national equivalent design standards or codes of practice. Initially, Eurocodes are published as preliminary standards , ENV (Prenorme Europeenne): equivalent to BSI's

Draft for Development. ENVs are optional, and have a life of three years, with a possible extension of two years. They will then be revised and reissued as European Standards, EN (Norme Europeene) which are mandatory in the sense that conflicting national standards must be withdrawn. All Euro codes are published in conjunction with a National Application Document (NAD), containing supplementary information specific to each member state. The NAD takes precedence over corresponding provisions in the Euro code (1991-1, 1994). At this stage several Eurocodes have been completed and majority of them are still issued as ENVs. Specifically, the main problems faced by the drafting panels for Eurocodes included agreeing to a common terminology acceptable to all the member states, resolving differing opinions on technical issues, taking into account national differences in materials and design and construction practices, and regional differences in climatic conditions. The terminology used in the Euro code is generally similar to that already used in the equivalent UK documents with some minor differences. For example, loads are now called actions while dead and imposed loads are now termed permanent and variable actions, respectively. Similarly, bending moments and axial loads are now called internal moments and internal forces, respectively. It is anticipated that these changes are unlikely to present any major problems, especially to those familiarised with British Standards (Shafii, Omar, Mohammad, & Makhtar, 2001).

A distinct feature of the Euro code is that it's concise as yet it describes the overall aims of design and provides specific guidance on how to achieve these aims in practice. For these purposes, the materials in Eurocodes were divided into principles and application rules. The principles (designated P) are general statements, definitions and analytical methods, etc. for which no alternative is permitted. The application rules are offset to the right of the page and generally are recognised rules which follow the principles and satisfy their requirements. Usually, an application rule provides one suggested method for satisfying the corresponding principle. Eurocodes are quite flexible in a way that it permits the use of alternative design rules provided that it can be shown to satisfy the relevant principles, and not negating other aspects for example, serviceability and durability of the structure (Shafii, Omar, Mohammad, & Makhtar, 2001).

Euro code 2 will be the one design code for all concrete structures in the UK and Europe. EN 1992-1-1 was published in December 2004 and the National Annex was published in December 2005, making it possible to use in the UK. The standard was developed to bring reinforced concrete design up to date. It benefits from over 40 years of knowledge and experience since the forerunners of the UK ultimate limit state codes (BS 8110, BS 8007) were first put together. Eurocode 2 may appear hard comprehend and more complex, at first sight, than UK codes but is undoubtedly more comprehensive (Ajis, 2012).

Euro code 2 has four parts

1. BS EN 1992-1-1 Common rules for buildings and civil engineering structures
2. BS EN 1992-1-2 Structural fire design
3. BS EN 1992-2 Bridges
4. BS EN 1992-3 Liquid retaining structures

Each part has a National Annex (NA) which gives national values for certain partial factors (or Nationally Determined Parameters, NDP's). Besides safety, the NA might also include matters of national custom and practice. These NAs are backed by PD6687 which gives the background to UK choices. PD6687 covers BS EN 1992-11 and BS EN 1992-1-2. During 2008 it is anticipated that PD6687 will be extended to cover BS EN 1992-3 and PD6697-2 covering the NDP's to BS EN 1992-2. in the UK. Eurocode 2 will replace BS 8110 for buildings, BS 5400 for bridges, BS 8007 for water-retaining structures and BS 6349 for maritime structures.

Table 2.2: Parts of Eurocode (Ajis, 2012)

EN	Part	Subject that involved
EN 1990	Euro code	Basis of structural design
EN 1991	Euro code 1	Actions on structures
EN 1992	Euro code 2	Design of concrete structures
EN 1993	Euro code 3	Design of steel structures
EN 1994	Euro code 4	Design of composite steel and concrete structures
EN 1995	Euro code 5	Design of timber structures
EN 1996	Euro code 6	Design of masonry structures
EN 1997	Euro code 7	Geotechnical design
EN 1998	Euro code 8	Design of structures for earthquake resistance
EN 1999	Euro code 9	Design of aluminium structures

2.3.4 Scope of Eurocode 2

EC2 is a guide for the construction of buildings and civil engineering works which are occur in reinforced concrete works and prestressed concrete. The standard Code of practice also complies with the principles and requirements of safety and serviceability as the fundamentals of structural design and verification found in EN 1990: Fundamentals of Structural Design. EC2 emphasizes the need for resistance, serviceability, durability and resistance to fire for concrete structures. In EC2, there are some clauses which are divided according to the behaviour of a structural member.

EC2 describes the use in general for all buildings structures, reinforced and prestressed concrete. Both concrete of normal weight concrete and light weight are included in this section. Each part explains the difference in EN 1992-1-1 basic phenomenon (e.g. flexure, shear, deflection and bending) more than the types of members (e.g. beams, slabs, columns) (Ajis, 2012).

Table 2.3: Relationship between Eurocode and British Standard (Ajis, 2012)

Eurocode	Title	British Standard
BS EN 1990	Basic of structural design	BS 8110:Part 1 - section 2
BS EN 1991-1-1	Densities, self-weight and imposed loads	BS 6399:Part 1
BS EN 1991-1-4	Wind actions	BS 6399: Part 3
BS EN 1992-1-1	General rules of buildings	BS 8110:Part 1,2 and 3
BS EN 1992-1-2	Fire resistance concrete structures	BS 8110:Part 1, table 3.2 and BS 8110:Part 2, section 4

2.4 Differences Between BS 8110 And EC2

i) Terminology and symbols
ii) The contents and clauses
iii) Material characteristics
iv) Durability design
v) Partial safety factor of materials
vi) Stress and strain distribution of the section

Terminology and symbol

Terminology and symbols used in Eurocode is somewhat similar to that found with a BS in 8110. However, there are several different between the two codes in this practice. Table 2.4 and Table 2.5 show the difference:

Table 2.4: Terminology differences between BS 8110 and EC2 (Ajis, 2012)

BS 8110	EC2
Loads	Actions
Dead load	Permanent action
Imposed load	Variable action
Bending moment	Internal moment
Axial forces	Internal forces

Table 2.5: Symbollic differences between BS 8110 and EC2 (Ajis, 2012)

BS8110	EC2
Characteristic dead load, G_k	Characteristic permanent action, G_k
Characteristic imposed load, Q_k	Characteristic variable action, Q_k
Characteristic strength of concrete (cube), f_{cu}	Characteristic strength of reinforcement, f_{ck}
Characteristic strength of reinforcement, f_y	Characteristic strength of reinforcement, f_{yk}
Partial safety factor for load, r-	Partial safety factor for permanent action, γ_G Partial safety factor for variable action, γ_Q

Contents and chronology of clause

In EC2, organization structure is based on behaviour such as shear flexure, shear, deflection and bending while for BS8110, the clause arrangement is by type of structural elements such as slabs, beams, columns and so on.

Material characteristics

In EC2, the formula is based on the design of cylindrical concrete strength 28 days, f_{ck} while BS 8110 using 28 days concrete cube strength, f_{cu}. By estimation, the strength of the cylinder is 80% of the cube strength (Ajis, 2012).

Durability design

BS 8110 and EC2 have identified the durability of concrete structure is closely related to environmental conditions, reinforcement cover, concrete quality and maximum width of the crack. To select the cover in EC2, environmental condition has been considered with classified the environment into 9 sections (EC 2 - Section 4 Durability and Cover to Reinforcement) and BS 8110 does not specify the circumstances but exposed only to classify the situation as mild, moderate, severe, and very severe (BS 8110, Table 3 Nominal Cover To All Reinforcement To Meet Durability Requirement) (Ajis, 2012).

Partial safety factor of materials

Same as BS 8110, EC 2 also use the factor of safety for concrete material γ_m is 1.5 Then, the factor of safety for steel in BS 8110 has been reduced from 1.15 to 1.05. For existing yield strength, BS 8110 has taken 460 N/mm^2 while EC2 taking 500N/mm^2

2.5 Comparison Between Structural Eurocodes And British Standards

The idea to develop a set of harmonised and common structural design codes for European countries started in 1974, originated in 1957 at the Treaty of Rome through the European Economic Community (EEC). The presence of the common codes amongst European member states has been seen advantageous particularly in lowering trade barriers between them and enables engineers, contractors and consultants from the member states to practice within all European countries (EC) and to compete fairly for works within Europe. The use of a common code is also expected to lead to a pooling of resources and sharing of expertise, thereby lowering the production costs.

In the seventies, the international technical and scientific organisations in Europe agreed to prepare works in coordinating the design principles, formulating rules and establishing the state-of-the-art technical reports. Thereafter, the Commission of European Communities (CEC) took the initiative to elaborate these preparatory works by establishing five working expert groups, including one on "Stability of Structures" which listed the main design codes, later became known as Structural Eurocodes. However, at that time there was no legal obligations in using the codes for the codes are only to facilitate commercial exchanges between EEC countries and promoting the use of a single European standard for construction methods, materials, types of buildings and civil engineering works. The formation of the Single European Act in 1986 was the one which provides impetus to tackle the legal issues to the process of harmonisation. This Act provides directives in which no legislation can stop the exchange of European construction products. Each Eurocode has been drafted by a small group of experts from various member states. These groups were formerly under contract to the EC Commission but are now under the direct control of CEN (Committee European de Normalisation), the European Standards Organisation.

A liaison engineer from each member state has been involved in evaluating the final document and discussing with the drafting group on the acceptability of the Eurocode in relation to the national standard from the country which they present.

The Commission of the European Committee (CEC) initiated the work of establishing a set of harmonised technical rules for the design of buildings and civil engineering works which would initially serve as an alternative to the different rules in force in the various member states. These technical rules then became known as the Structural Eurocodes which serve as the European standards for structural design. In 1990, after consulting with the respective member states, the CEC transferred the work of further development, issue and updating the Structural Eurocodes to CEN, and EFTA secretariat agreed to support the CEN work. CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes. The proposed Eurocodes currently under preparation are as follows:

i. EN 1991 - EC 1: Basis of Design and Actions on Structures

ii. EN 1992 - EC 2: Design of Concrete Structures

iii. EN 1993 - EC 3: Design of Steel Structures

iv. EN 1994 - EC 4: Design of Composite Steel and Concrete Structures

v. EN 1995 - EC 5: Design of Timber Structures

vi. EN 1996 - EC 6: Design of Masonry Structures

vii. EN 1997 - EC 7: Geotechnical Design

viii. EN 1998 - EC 8: Design of Structures for Earthquake Resistance

ix. EN 1999 - EC 9: Design of Aluminum Alloy Structures.

The steps brought about by the developments of Eurocodes have significant impacts on British Standard users as considerations must be made in keeping abreast with developments and technologies in current practices. This article aims to provide some fundamental background of the European Structural Codes (Eurocodes), and some introductory aspects particularly on design principles and the differences brought about by the harmonized codes.

Eurocode 2, EC2 is the European proposed standard for the structural design of concrete structures. The British Standards Institution (BSI) has planned to publish 9 separate documents related to concrete design under EC2. The publications, knowns as European Prestandards (ENV) are listed below:

i. DD ENV 1992-1-1: 1992 General rules for buildings.

ii. DD ENV 1992-1-2: 1996 Structural fire design

iii. DD ENV 1992-1-3: 1996 Precast concrete elements and structure

iv. DD ENV 1992-1-4: 1996 Lightweight aggregate concrete

v. DD ENV 1992-1-5: 1996 Structures with unbonded and prestressing tendons

vi. DD ENV 1992-1-6: 1996 Plain concrete structures

vii. ENV 1992-2 : 1996 Concrete bridges

viii. ENV 1992-3 : 1998 Concrete foundations

ix. ENV 1992-4 : 1998 Liquid retaining and containment structures (Ajis, 2012)

Publications with DD notation have been adopted in the UK and accompanied by its corresponding National Application Document (NAD). The NAD provides operational guidance for each country. There are other documents under EC2, apart from those listed, yet to be published. Generally, EC2: Part 1 is broadly comparable to the existing British Standard, BS 8110 Part 1 and 2. Whilst BS 8110 is basically applicable to buildings. EC2 comprised of various parts and covers on the different types of structures. Building is generally covered by EC2: Part 1

EC 2: Part 1 can be distinguished easily from BS 8110 in the way the chapters are described. The former contained chapters dealing with beams, slabs, columns, etc. whereas, EC2: Part 1 has chapters on bending, shear, torsion, buckling etc. The arrangement of chapters in EC2 is basically based on phenomena whilst BS 8110 uses element types.

Typical of any Eurocodes, the Principles stated in EC2 does not allow for alternatives and all designs should comply with them. Application Rules allow for alternative methods provided that it can be demonstrated that they comply with the principles. As stated earlier some of the terms used in Eurocodes are different from British Standards, in that it tries to cover a wide variety of situations. In EC2 the terminology for 'loading' has been replaced by 'actions'. Changes are made on the dead loads definition in EC2. EC2 draws a distinction between loads with small and large variations. If the variations between lower and upper loads is less than 20% of the mean value, then the mean value is used as the characteristic value. If the variation exceeds 20%, then both the lower and upper loads should be considered as characteristic values. BS 8110 does not make such an explicit distinction in the definition of the characteristic value of dead loads. Such considerations are relevant when dealing for example, with the weight of a slab and a wall cast against earth. Other modifications with regard to loads are made to the load combinations and the values of corresponding partial safety factors both at serviceability and ultimate limit states. The partial safety factor for reinforcement does not change. For concrete, EC2 adopt a single value of 1.5 throughout, as oppose to BS 8110 which is using different values for bending, shear and bond. With regard to durability considerations, EC2 does not permit the 'trade-off' between cover and concrete quality as BS 8110 does.

in general, EC2 provides only the basic information required, whereas BS 8110 gives considerably more detailed information. With BS 8110 one can use the coefficients given for various load effects such as bending moments and shear coefficients for continuous beams and slabs. EC2 expects the designer to obtain these from textbooks or manuals. In EC2, design formulae are generally related to the cylinder strength. This is one of important changes that must be noted. As an approximation the cylinder strength can be taken as 80% of the cube strength. Another difference between EC2 and BS 8110 is the load arrangements for buildings Both EC2 and BS 8110 permit redistribution of bending moments in continuous beams. The difference lies in the rules given to cover the ductility and detailing requirements in the two documents. For EC2, 30% redistribution is permitted for high ductility steel, and 15% for normal ductility. EC2 does not permit any redistribution in sway frames, whereas up to 10% redistribution is allowed by BS 8110.

Flexural design of sections using EC2 is rather complicated as compared to BS 8110. EC2 permits the use of stress-strain curve for the reinforcement which is identical to that in BS 8110. EC2 also allows the use of a relationship with a sloping upper branch, which takes strain hardening into account. For stress-strain curve of concrete, EC2 uses the same basic diagram as BS 8110, but slightly simpler to use. EC2 allows the use of simplified stress block. It permits the use of both a rectangular and a bilinear diagram. The expression of shear strength of concrete in EC2 contains all the parameters as in BS 8110. There are some differences with regard to limitations.

BS8110 still continues to enjoy a large degree of prominence on the African continent especially in Nigeria. The need to make a major shift towards full embrace of the EC2 design philosophy as well as provisions is imperative as this will have some impact on the design of all types of structures on the African continent. Consequently it is essential to publish the results of research and other data that narrow and focus the scope of the new design methods on specific elements that practicing are directly involved with. Hence a design aided investigation and comparative study is necessary to highlight points of convergence and difference between EC2 and BS8110. This is proposed to be achieved in the present study based on the analysis and design of a library structure. Similar studies have been carried out in the United Kingdom (Narayanan & Webster, 1994) and these were instrumental in the development of the present investigation.

Different researchers have worked on comparison of structural members to different codes.

a) Liew (2009) "British standard (BS 8110) and Eurocode 2 (EC2) for reinforced concrete column design" The study carried out in Malaysia tried to address the perception designers over there have that design using EC2 is very difficult and that it is not very different from BS 8110. The study conducted a review of the design steps for column design using Eurocode 2.Several types of columns were designed according to the two codes and resulting area of steel reinforcements were compared. Results showed that although the design process of EC2 was more technical, they were still easy to understand and follow and design using EC2 was much more economical.

b) (Alnuaimi & Patel, 2013) presented a comparative calculation study of the deflection, bar anchorage, lap lengths and control of crack width of reinforced concrete beams using the BS 8110 and ACI318 codes. The deflections were calculated using the BS code were smaller than those predicted by the ACI code. short-term deflection decreases with the increase in the dead-to-live load ratio whereas the long-term deflection increases for both codes. The study also showed the BS code maintains a constant bar spacing regardless of the concrete cover, but for the ACI code, it reduces with the increase in concrete cover. With increase in concrete strength, the tension anchorage length decreases for both codes. The BS code requires a greater anchorage length in compression than the ACI code does. The compression lap length requirement in the BS is more than that in ACI code for the concrete of compressive strength less than 37 MPa and the former stipulates longer lap lengths for higher concrete strengths.

Usually in Nigeria, the design of structures is usually guided by the use of British Standard. (BS 8110) in Nigeria (Nwofor, Sule, & Eme, 2015). BS 8110 is a British Standard for the design and construction of reinforced and prestressed concrete structures. BS 8110 is based on limit state design principles. Although used for most civil engineering and building structures, bridges and water-retaining structures are covered by separate standards (BS 5400 and BS 8007 respectively) (Nwofor, Sule, & Eme, 2015).

Structural design is a process of selecting the material type and conducting in-depth calculation of a structure to fulfil its construction requirements (Yusoff, 2015)Design may also be described as a process through which the engineer determines the type, size and materials used through a meticulous calculation until detailed drawing is produced (O'Brien & Dixon, 1995).The main purpose of structural design is to produce a safe, economic and functional building. Structural design should also be an integration of art and science. It is a process of converting an architectural perspective into a practical and reasonable entity at a construction site. In the structural design of concrete structures, reference to standard code is essential. A standard code serves as a reference document with important guidance. The contents of the standard code generally cover comprehensive details of a design. These details include the basis and the concept of design, specifications to be followed, design methods, safety factors, loading values and etc. In present days, many countries have published their own standard codes. These codes were a product of constant research and development, and past experiences of experts in respective fields. Meanwhile, countries or nations that do not publish their own standard codes will adopt a set of readily available code as the national reference. Several factors govern the type of code to be adopted namely suitability of application of the code set in a country with respect to its culture, climate and national preferences as well as the trading volume and diplomatic ties between these countries (Yusoff, 2015).

2.6 Reinforced Concrete Structures

The task of the structural engineer is to design a structure which satisfies the needs of the client and the user. Specifically the structure should be safe, economical to build and maintain, and aesthetically pleasing (Arya, 2009).

Design is a word that means different things to different people. Architects may define design as being the production of drawings and models to show what a new building will actually look like. To civil and structural engineers, however, design is taken to mean the entire planning process for a new building structure, bridge, tunnel, road, etc., from outline concepts and feasibility studies through mathematical calculations to working drawings which could show every last nut and bolt in the project. Together with the drawings there will be bills of quantities, a specification and a contract, which will form the necessary legal and organizational framework within which a contractor, under the supervision of engineers and architects, can construct the scheme. The aims of design include

i. To achieve an acceptable probability that the structure will perform satisfactorily during its intended life
ii. With an appropriate degree of safety, the structure should sustain all the loads and deformations of normal construction and use, have adequate durability, and resistance to the effects of misuse and fire
iii. Calculations alone do not produce safe, serviceable and durable structures. Equally important are the suitability of the materials, quality control and supervision of workmanship during construction
iv. To produce a structure which is economical to construct, maintain and service throughout its design life (Arya, 2009)

There are many inputs into the engineering design process including: client brief, experience, imagination, a site investigation, model and laboratory tests, economic factors and environmental factors (Arya, 2009).

The starting-point for the designer is normally 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 architect's drawings. Experience is crucially important, and a client will always demand that the firm he is employing to do the design has previous experience designing similar structures. Although imagination is thought by some to be entirely the domain of the architect, this is not so. For engineers an imagination of how elements of structure interrelate in three dimensions is essential, as is an appreciation of the loadings to which structures might be subject in certain circumstances. In addition, imaginative solutions to engineering problems are often required to save money, time, or to improve safety or quality. A site investigation is essential to determine the strength and other characteristics of the ground on which the structure will be founded. If the structure is unusual in any way, or subject to abnormal loadings, model or laboratory tests may also be used to help determine how the structure will behave. In today's economic climate a structural designer must be constantly aware of the cost implications of his design. On the one hand design should aim to achieve economy of materials in the structure, but over-refinement can lead to an excessive number of different sizes and components in the structure, and labour costs will rise. In addition the actual cost of the designer's time should not be excessive, or this will undermine the employer's competitiveness. The idea is to produce a workable design achieving reasonable economy of materials, while keeping manufacturing and construction costs down, and avoiding unnecessary design and research expenditure. Attention to detailing and buildability of structures cannot be overemphasized in design.

Most failures are as a result of poor detailing rather than incorrect analysis. Designers must also understand how the structure will fit into the environment for which it is designed. Today many proposals for engineering structures stand or fall on this basis, so it is part of the designer's job to try to anticipate and reconcile the environmental priorities of the public and government. The engineering design process can often be divided into two stages:

(1) a feasibility study involving a comparison of the alternative forms of structure and selection of the most suitable type and

(2) a detailed design of the chosen structure. The success of stage 1, the conceptual design, relies to a large extent on engineering judgement and instinct, both of which are the outcome of many years' experience of designing structures.

Stage 2, the detailed design, also requires these attributes but is usually more dependent upon a thorough understanding of the codes of practice for structural design, e.g. BS 8110 and BS 5950. These documents are based on the amassed 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

2.7 Methods Of Design

The degree of overlap between the two curves can be minimized by using one of three distinct design philosophies, namely:

1. permissible stress design

2. load factor method

3. limit state design.

2.7.1 Permissible Stress Design

In permissible stress design, sometimes referred to as modular ratio or elastic design, the stresses in the structure at working loads are not allowed to exceed a certain proportion of the yield stress of the construction material, i.e. the stress levels are limited to the elastic range. By assuming that the stress–strain relationship over this range is linear, it is possible to calculate the actual stresses in the material concerned. Such an approach formed the basis of the design methods used in CP 114 (the forerunner of BS 8110) and BS 449 (the forerunner of BS 5950). However, although it modelled real building performance under actual conditions, this philosophy had two major drawbacks. Firstly, permissible design methods sometimes tended to overcomplicate the design process and also led to conservative solutions. Secondly, as the quality of materials increased and the safety margins decreased, the assumption that stress and strain are directly proportional became unjustifiable for materials such as concrete, making it impossible to estimate the true factors of safety (Arya, 2009).

2.7.2 Load Factor Design

Load factor or plastic design was developed to take account of the behaviour of the structure once the yield point of the construction material had been reached. This approach involved calculating the collapse load of the structure. The working load was derived by dividing the collapse load by a load factor. This approach simplified methods of analysis and allowed actual factors of safety to be calculated. It was in fact permitted in CP 114 and BS 449 but was slow in gaining acceptance and was eventually superseded by the more comprehensive limit state approach (Arya, 2009).

2.7.3 Limit State Design

This was originally formulated in the former Soviet Union in the 1930s and developed in Europe in the 1960s, regarded as fairly safe. The risk of death or injury due to structural failure is extremely low, but as we spend most of our life in buildings this is perhaps just as well. As far as the design of structures for safety is concerned, it is seen as the process of ensuring that stresses due to loading at all critical points in a structure have a very low chance of exceeding the strength of materials used at these critical points. In design there exist within the structure a number of critical points (e.g. beam mid-spans) where the design process is concentrated Most modern structural codes of practice are now based on the limit states approach. BS 8110 for concrete, BS 5950 for structural steelwork, BS 5400 for bridges and BS 5628 for masonry are all limit state codes. The principal exceptions are the code of practice for design in timber, BS 5268, and the old (but still current) structural steelwork code, BS 449, both of which are permissible stress codes It should be noted, however, that the Euro code for timber (EC5), which is expected to replace BS 5268 around 2010, is based on limit state principles. As limit state philosophy forms the basis of the design methods in most modern codes of practice for structural design, it is essential that the design methodology is fully understood. This then is the purpose of the following subsections (Arya, 2009).

Ultimate And Serviceability Limit States

The aim of limit state design is to achieve acceptable probabilities that a structure will not become unfit for its intended use during its design life, that is, the structure will not reach a limit state. There are many ways in which a structure could become unfit for use, including excessive conditions of bending, shear, compression, deflection and cracking. Each of these mechanisms is a limit state whose effect on the structure must be individually assessed. Some of the above limit states, e.g. deflection and cracking, principally affect the appearance of the structure. Others, e.g. bending, shear and compression, may lead to partial or complete collapse of the structure. Those limit states which can cause failure of the structure are termed ultimate limit states. The others are categorized as serviceability limit states. The ultimate limit states enable the designer to calculate the strength of the structure Serviceability limit states model the behaviour of the structure at working loads. In addition, there may be other limit states which may adversely affect the performance of the structure, e.g. durability and fire resistance, and which must therefore also be considered in design. It is a matter of experience to be able to judge which limit states should be considered in the design of particular structures. Nevertheless, once this has been done, it is normal practice to base the design on the most critical limit state and then check for the remaining limit states. For example, for reinforced concrete beams the ultimate limit states of bending and shear are used to size the beam. The design is then checked for the remaining limit states, e.g. deflection and cracking. On the other hand, the serviceability limit state of deflection is normally critical in the design of concrete slabs. Again, once the designer has determined a suitable depth of slab, he/she must then make sure that the design satisfies the limit states of bending, shear and cracking. In assessing the effect of a particular limit state on the structure, the designer will need to assume certain values for the loading on the structure and the strength of the materials composing the structure (Arya, 2009).

Characteristic and design values

As stated at the outset, when checking whether a particular member is safe, the designer cannot be certain about either the strength of the material composing the member or, indeed, the load which the member must carry. The material strength may be less than intended

(a) because of its variable composition, and

(b) because of the variability of manufacturing conditions during construction, and other effects such as corrosion.

Similarly the load in the member may be greater than anticipated because of the variability of the occupancy or environmental loading, and because of unforeseen circumstances which may lead to an increase in the general level of loading, errors in the analysis, errors during construction, etc. In each case, item (a) is allowed for by using a characteristic value. The characteristic strength is the value below which the strength lies in only a small number of cases. Similarly the characteristic load is the value above which the load lies in only a small percentage of cases. In the case of strength the characteristic value is determined from test results using statistical principles, and is normally defined as the value below which not more than 5% of the test results fall. However, at this stage there are insufficient data available to apply statistical principles to loads. Therefore the characteristic loads are normally taken to be the design loads from other codes of practice, e.g. BS 648 and BS 6399. The overall effect of items under (b) is allowed for using a partial safety factor: γ_m for strength (Arya, 2009).

2.8 Loads

Dead and Imposed Loads

Dead loads

The dead loads are taken as the self-weight of the structure and are gotten by multiplying the cross sectional area of the beam by the unit weight of concrete for both codes. The unit weight of concrete as per BS8110 is given as 24 KN/m^3, while that for Eurocode 2 is 25 KN/m^3. The differences in these principles might result in differences in the amount of load a common member dimension could carry, be it at service or the ultimate limit state. Consequently, the amount of reinforcement required might also be affected.

Dead loads include

a) Weight of concrete

Some typical weights of plain and reinforced concrete, solid concrete slabs, hollow clay-block slabs, concrete products, finishes, lightweight concretes and heavy concrete

b) Other structure materials and finishes

Dead loads include such permanent weights as those of the finishes and linings on walls, floors, stairs, ceilings and roofs, asphalt and other applied waterproofing layers; partitions; , windows, roof lights and pavement lights; superstructure of steelwork, masonry, brickwork or timber; concrete bases for machinery and tanks; fillings of earth, sand, puddled clay, plain concrete or hardcore; cork and other insulating materials; rail tracks and ballasting; refractory linings; and road surfacing.

c) Partitions

The weights of partitions should be included in the dead loads of floors and it is convenient to consider such weights equivalent uniformly distributed loads. The material of which the partition is constructed and the storey height will determine the weight of the partition, and in the design of floors the actual weight and position of a partition, when known, should be allowed for when calculating shearing and bending moments on the slab and beams.

Imposed loads

Imposed loads on structures include the weights of stored solid materials and liquids and the loads imposed by vehicles and moving equipments.

In the beam design, commonly used parameters such as imposed loads and concrete grade were made uniform for both BS8110 and Eurocode 2. Varied parameters were mainly those based on theories of the codes such as the equations governing flexure at the ultimate limit state and shear.

Table 2.6 below shows the basic span/effective depth ratios of a rectangular beam for both codes.

Table 2.6: BS 8110 and Eurocode2 basic span/effective depth ratios for rectangular beams

Support conditions	BS8110-1997[3]	Eurocode 2 [12]
Cantilever	7	[illegible]
Simply supported	20	18
Continuous	26	25
[illegible]	-	[illegible]

Ultimate Design Load

In addition to the varied parameters mentioned above, the ultimate design load formulae in both codes are of great importance in the rate of loading on the structure. Consequently, the moments and shear forces acting on the structure may be affected as a result of these variations in loading. At the ultimate limit state, the maximum design load can be estimated by using equations (1) and (2) for BS8110 and Eurocode2 respectively.

$w = 1.4g_k + 1.6q_k$ (1)

$n = 1.35g_k + 1.5q_k$ (2)

Where:

g_k and q_k are dead loads (including self-weight) and imposed loads respectively. 1.4, 1.6 and 1.35, 1.5 is all partial safety factors for loads for BS8110 and Eurocode2 respectively.

Characteristic and design values

As stated at the outset, when checking whether a particular member is safe, the designer cannot be certain about either the strength of the material composing the member or, indeed, the load which the member must carry. The material strength may be less than intended

(a) because of its variable composition, and

(b) because of the variability of manufacturing conditions during construction, and other effects such as corrosion. Similarly the load in the member may be greater than anticipated (a) because of the variability of the occupancy or environmental loading, and

(b) because of unforeseen circumstances which may lead to an increase in the general level of loading, errors in the analysis, errors during construction, etc

In each case, item (a) is allowed for by using a characteristic value. The characteristic strength is the value below which the strength lies in only a small number of cases. Similarly the characteristic load is the value above which the load lies in only a small percentage of cases. In the case of strength the characteristic value is determined from test results using statistical principles, and is normally defined as the value below which not more than 5% of the test results fall. However, at this stage there are insufficient data available to apply statistical principles to loads. Therefore the characteristic loads are normally taken to be the design loads from other codes of practice, e.g. BS 648 and BS 6399.

Table 2.7: Summary of the differences between BS 8110 and Eurocode 2

S/N	Parameters	BS 8110	Eurocode 2(CEN, 1992)
1.	Concrete strength	Cube strength, f_{cu} $f_{cu} \approx f_{ck}/0.8$	Cylinder strength, f_{ck} $f_{ck} \approx 0.8f_{cu}$
2.	Partial safety factor, γ_m	For concrete in bending = 1.5 For steel = 1.15	For concrete: Fundamental 1.5 Accidental 1.3 For steel: Fundamental 1.5 Accidental 1.0
3.	Yield strength of high yield steel	$f_y = 460\ MPa$ f_y/γ_m $= 400\ MPa$	$f_{yk} = 460\ MPa$ $f_{yk}/\gamma_m = 400\ MPa$
4.	Yield strength of high mild steel	$f_y = 250\ MPa$ f_y/γ_m $= 217\ MPa$	$f_{yk} = 250\ MPa$ $f_{yk}/\gamma_m = 217\ MPa$
5.	Ultimate strain of concrete, ε_{cu}	0.0035 for flexure	0.002 for axial load 0.0035 for flexure
6.	Maximum allowable neutral axis depth, x	0.5d (no redistribution)	0.45d for $f_{ck} \leq 35\ MPa$ 0.35d for $f_{ck} > 35\ MPa$ $0.35d\ for\ plastic\ analysis$
7.	Concrete compression zone depth (simplified rectangular stress block)	0.9x	0.8x
8.	Ultimate moment of resistance, M_u	$M_u = 0.156f_{cu}bd^2$	For $f_{ck} \leq 35$ MPa: M_u $0.167f_{ck}bd^2$ For $f_{ck} > 35$ MPa: M_u $0.128f_{ck}bd^2$
9	Shear	$V_k = v_c b_v d$	$V_{Rd1} = \tau_c b_w d$
10	Links	Longitudinal	Longitudinal spacing

		spacing ≤ 0.75d or 300mm Transverse spacing ≤ d	V_{Rd2}	Lesser of
			0 -0.2	0.8d or 300mm
			0.20-0.67	0.6d or 300mm
			0.67 -1.0	0.3d or 200mm
			Transverse spacing	
			V_{Rd2}	Lesser of
			0-0.2	d or 300mm
			0.20-0.67	0.6d or 300mm
			0.67 - 1.0	0.3d or 200mm

CHAPTER THREE

METHODOLOGY

3.1 Introduction

The proposed building is a three storey (10.8m height). library complex with a ground floor area of $1487.61m^2$, first floor area of $1487.61m^2$ and second floor area of $1051.56m^2$. The architectural plan was prepared using Revit Architecture.

Plate 3.1 shows the ground floor plan. The ground floor consists of garage on one side and archive area on the other side.

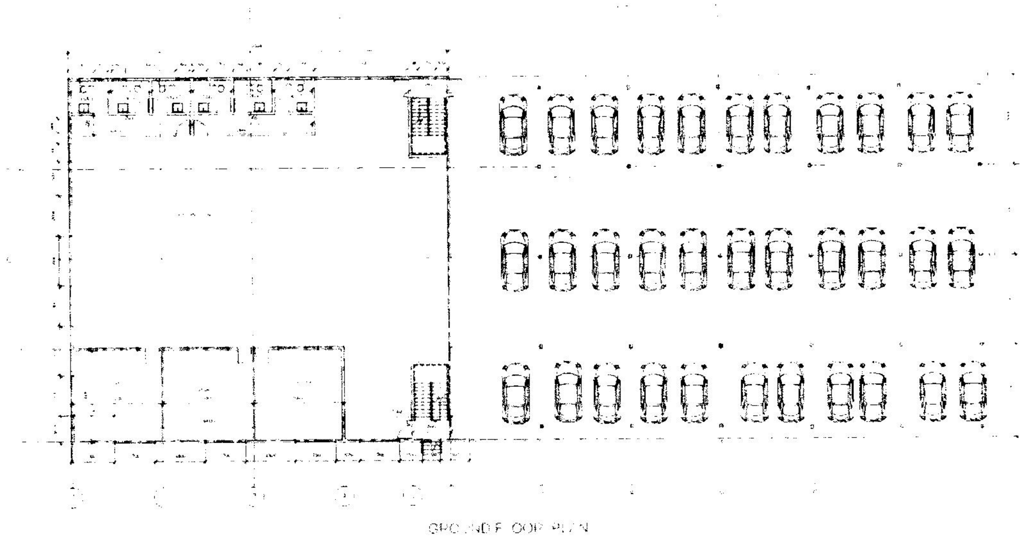


Plate 3.1: Ground floor plan

The first floor is the proposed Undergraduate library and consists of reading area and the bookshelf area. Plate 3.2 shows the first floor plan.

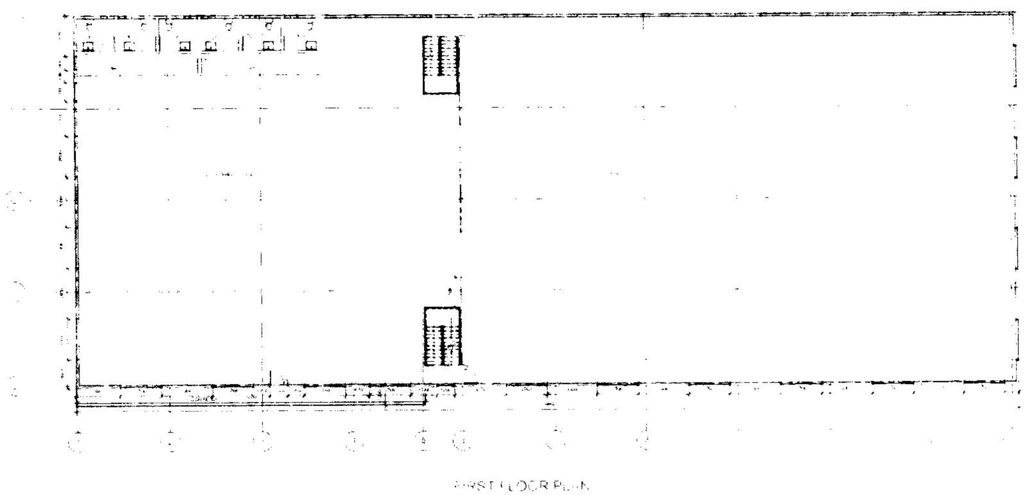


Plate 3.2: First floor plan

Plate 3.3 shows the second floor plan. The second floor is the proposed Postgraduate library and consists of reading area and bookshelf area.

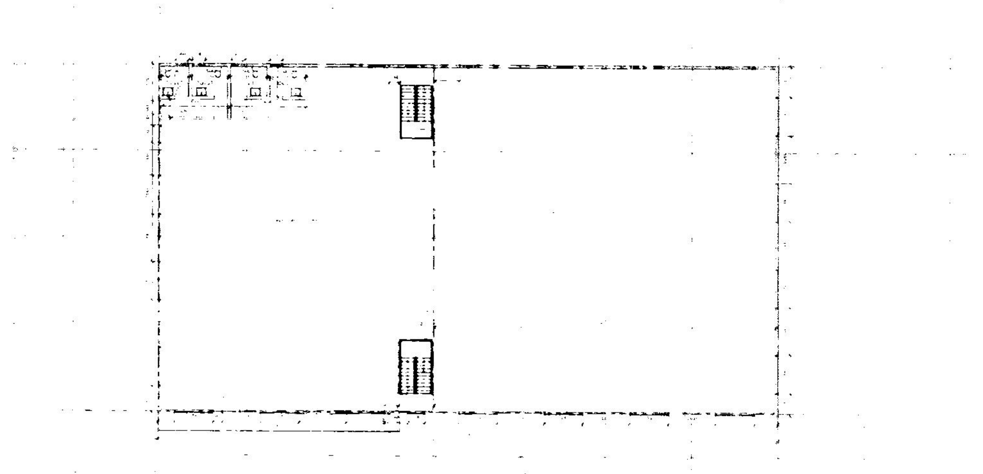


Plate 3.3: Second floor plan

Plate 3.4 shows the roof plan while Plates 3.5, 3.6, 3.7, 3.8 and 3.9 are showing the 3-dimensional views of the proposed library.

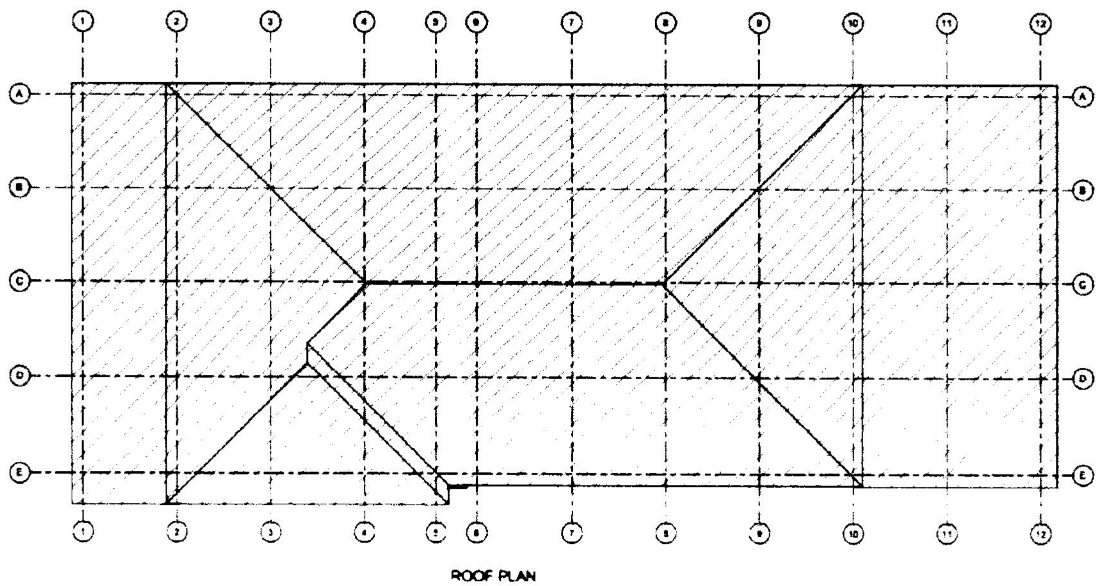


Plate 3.4: Roof plan

Plate 3.5: Approach view

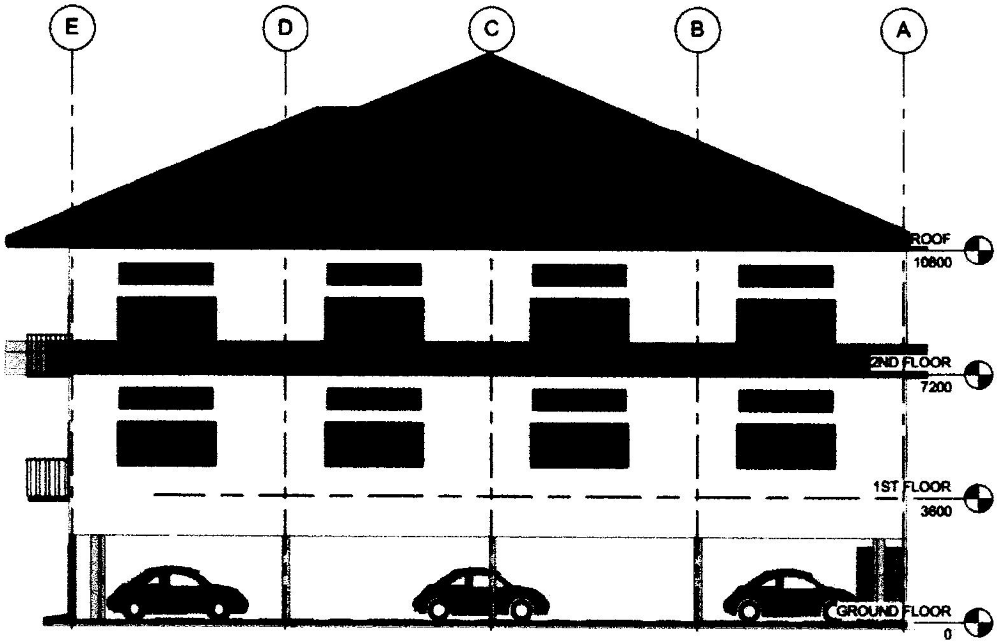


Plate 3.6: Right side view

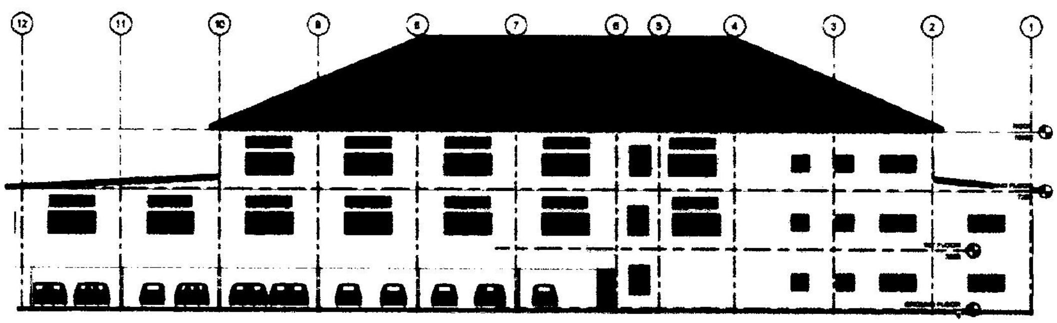


Plate 3.7: Rear view

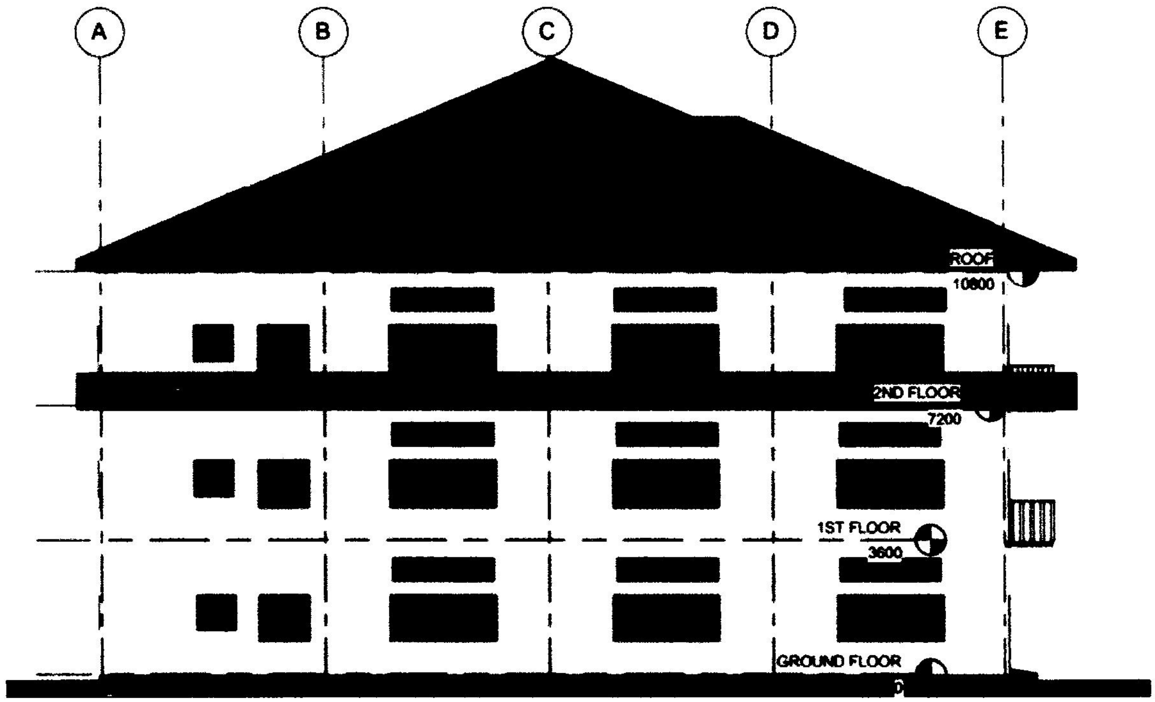


Plate 3.8: Left side view

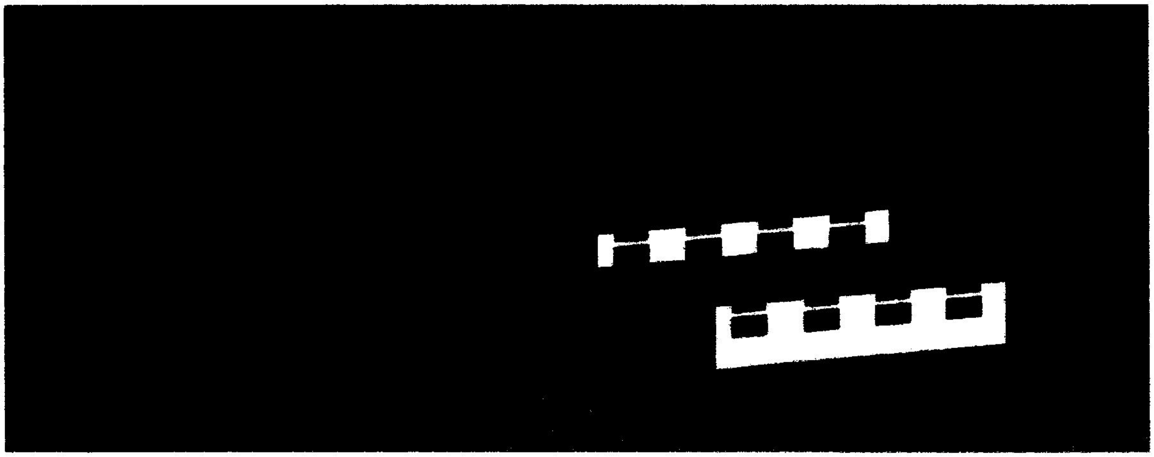


Plate 3.9: 3D view

Plates 3.10 and 3.11 are showing the sectional views of the proposed library

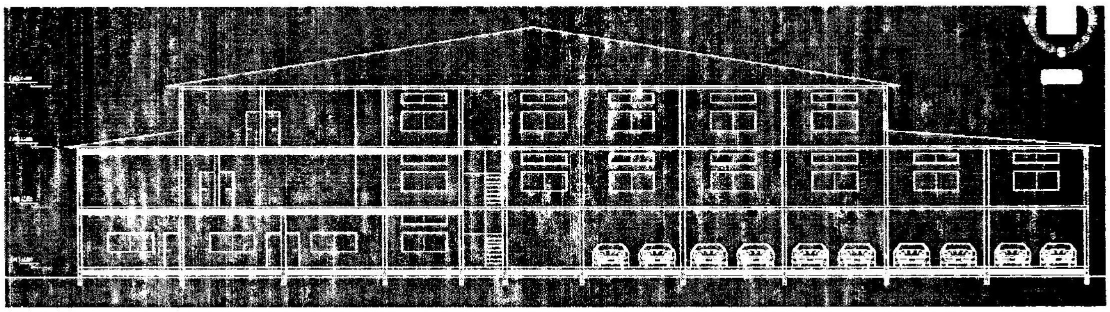


Plate 3.10: Longitudinal section

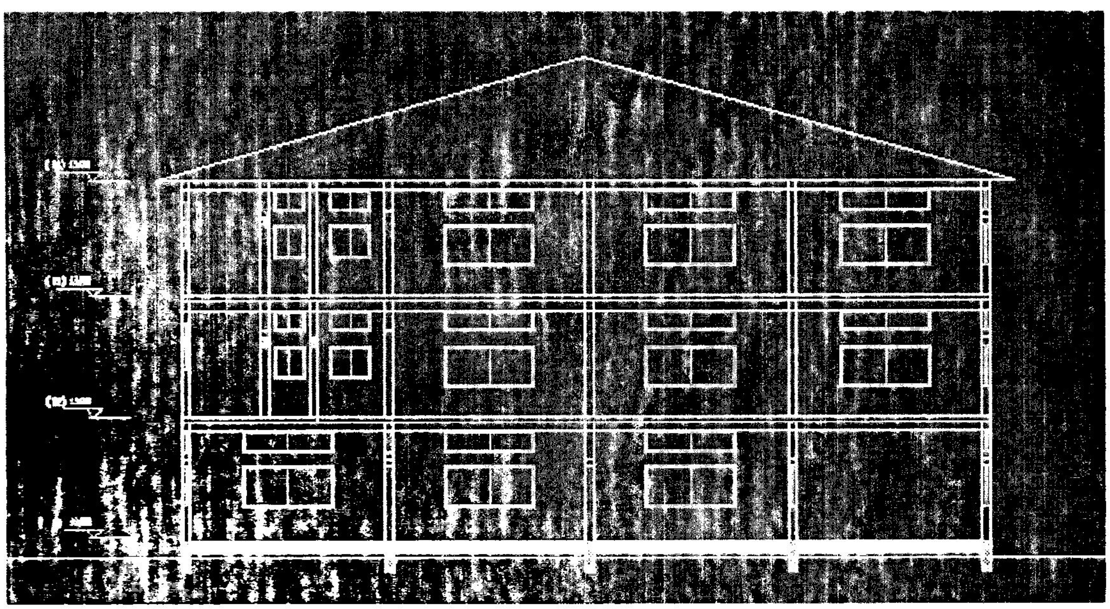


Plate 3.11: Transverse section

3.2 Structural Design

The proposed structure was designed using manual calculations. The dimensions of structural elements were estimated first by preliminary conceptual design. Then the vertical and horizontal loads were defined according to the provisions of BS 8110 and Eurocode 2 separately. The design and geometric parameters of the proposed structure were identical for both codes. The type of concrete used was C20/25 (characteristic cylinder/cubic compressive strength after 28 days). The types of steel used were the same for each code.

Design procedure:

1. Idealization of the structure into load-bearing frames and elements for analysis and design
2. Estimation of loads
3. Analysis to determine the maximum moments, thrusts and shears for design
4. Design of sections and reinforcement arrangements for slabs, beams, columns and walls using the results from 3
5. Production of arrangement and detail drawings and bar schedules

3.3 Structural Elements And Frames

The complete building structure can be broken down into the following elements:
Beams horizontal members carrying lateral loads
Slabs horizontal plate elements carrying lateral loads
Columns vertical members carrying primarily axial load but generally subjected to axial load and moment
Walls vertical plate elements resisting vertical, lateral or in-plane loads
Bases and foundations pads or strips supported directly on the ground that spread the loads from columns or walls so that they can be supported by the ground without
Before commencement of structural analysis and design, the architectural drawings need to be effectively studied and the salient features noted and catered for while modeling. After which, the drawings are schemed, that is, the layout of the ground floor plan was prepared by properly positioning columns, beams, staircase and slab that may be deemed fit.

3.4 Factors Governing The Design Of A Structure

The criteria which govern the design of a structure for a particular purpose may be summarized as follows:

a) Fitness for purpose
b) Safety and reliability
c) Durability
d) Good value for money
e) External appearance
f) User comfort

Fitness for purpose is generally covered by the overall geometry of the structure and its components. It should be possible to have unrestricted and unhindered use of the structure for the purpose for which it's built.

Safety and reliability are assured by following the codes of practice for loading, materials, design, construction and fire resistance.

Good value for money is perhaps the most important criterion. The designer should take into account not only the cost of materials but also the buildability, the time required to build, the cost of temporary structures, the cost of maintenance over a period of time, and in some cases the cost of demolition/decommissioning.

External appearance of a structure changes over a period of time. The designer should be aware of the effects of cracking, leaking, spalling, flaking etc. of the materials in use. The designer should make appropriate allowance to avoid the degradation of appearance.

User comforts are influenced by vibration of the structure due to wind, road rail, or vibrating machinery. Large deflection under load also cause alarm to the users. The designer should pay adequate attention to the alleviation of these anticipated discomforts.

Robustness comes with the chosen structural form and is determined by the additional inherent strength of the structure as a whole to withstand accidental loadings. Collapse of one key member in the structure must not initiate global collapse. The designer must foresee the 'domino effect' in the structure and avoid it by careful planning

3.5 General Arrangement

The General Arrangement (GA) or Layout drawing serve as a guide to loading, analyses, preliminary design/member sizing, and design proper of the structural elements as positioned on the layout. This drawing forms the basis for all drawings generated for the proposed project. The proposed library complex contains all the common types of structural elements: slabs (solid slab and flat slab), horizontal beams and vertical columns, staircase and foundation that are required to sustain all the horizontal and vertical loads on the structure.

The proposed library complex was first designed according to BS 8110 and then designed according to Eurocode 2. A detailed comparison for both codes is then carried out.

3.6 Design Methodology

Step 1: Dead load calculations

This includes the self-weight of the slabs, partition and finishes. Finishes was taken as 1.2kN/m², while partition is taken as 4.0kN/m².

Culled from Reinforced Concrete Designers' Handbook by Reynold and Steedman.

Step 2: Live Load calculations

The live load might not be given in calculations. Use the necessary code of practice (BS 6399). In this case we used a live load of 4.0kN/m²

Step 3: Ultimate design loads

The ultimate design load was calculated from:

At ULS, n = 1.4 Dead load + 1.6 Live load

Step 4: Design data

Before proceeding with calculations of moments, all known design data: strength of concrete (f_{cu}), strength of steel (f_k) used, assumed diameter of main steel, and cover were listed. Effective depth was calculated.

Step 5: Bending moment at mid-span

The bending moment at mid-span was calculated using formulae or using coefficients in BS 8110.

Step 6: Calculation of reinforcing steel at mid-span

Once Moment is calculated, then the following parameters were calculated:

K, z = 0.95d (maximum), $A_{s,req}$ and $A_{s,min}$

For BS 8110

$$k = \frac{M}{f_{cu}bd^2}$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{k}{0.9}}$$

$$Z = l_a d$$

For EC2

$$k = \frac{M}{f_{cK}bd^2}$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{k}{1.134}}$$

$$Z = l_a d$$

Step 7: Bending moment at support if beam is continuous

The bending moment at mid-span was calculated using formulae or using coefficients in

BS 8110 and EC 2 respectively.

Step 8: Calculation of reinforcing steel at mid-span (Design as rectangular section)

Once M is calculated, then the following parameters need to be calculated:

K, z = 0.95d (maximum), $A_{s,req}$ and $A_{s,min}$

Step 9: Deflection check At mid-span, Service stress fs needs to be calculated M/bd2. Modification ratio, Basic l/d

shall be known. Permissible l/d calculated and Actual l/d calculated

For deflection criteria to be satisfied, the permissible l/d ratio shall be greater than the actual l/d ratio.

Step 10: Shear Check

Maximum shear from the support center-line shall be determined and the shear stress calculated using:Shear stress = V/(bd) < 5Mpa or formula $8\sqrt{f_{cu}}$

Step 11: Topping reinforcement

Minimum steel shall be provided in the topping 0.13%bh

Solid Slab

Design to BS8110

REFERENCE	CALCULATIONS	OUTPUT

SHORT SPAN

Continuous Edge	Mid span
$A_{s\,req} = \frac{M}{0.95 f_y Z}$	$A_{s\,req} = \frac{M}{0.95 f_y Z}$
$= \frac{30.[illegible] \times 10^6}{0.95 \times 410 \times 158.86}$	$= \frac{28.19 \times 10^6}{0.95 \times 410 \times 160.55}$
$= [illegible]\ mm^2/m$	$= 450.79\ mm^2/m$
Provide Y_{12} mm @ 175 mm c/c	Provide Y_{12} mm @ 200mm c/c
$A_{s\,prov} = 646\ mm^2/m$	$A_{s\,prov} = 566\ mm^2/m$
$A_{s\,min} = 0.13\%\,bh$	$A_{s\,min} = 0.13\%\,bh$
$= 0.0013 \times 1000 \times 200$	$= 0.0013 \times 1000 \times 200$
$= 260\ mm^2/m$	$= 260\ mm^2/m$
Since $A_{s\,prov} > A_{s\,min}$	Since $A_{s\,prov} > A_{s\,min}$

PROVISION IS O.K.

LONG SPAN

Continuous Edge	Mid span
$B_{sy}^- = 0.045$	$B_{sy}^+ = 0.034$
$M_{sy}^- = B_{sy} n l_x^2$	$M_{sy}^+ = B_{sy}^+ n l_x^2$
$= 0.045 \times 21.75 \times 6^2$	$= 0.034 \times 21.75 \times 6^2$
$= 35.24\ kNm$	$= 26.62\ kNm$
$d = h - c - \phi - \phi/2$	$d = h - c - \phi - \phi/2$
$= 200 - 25 - 12 - 6$	$= 200 - 25 - 12 - 6$
$= 157mm$	$= 157mm$
$K = \frac{35.24 \times 10^6}{25 \times 1000 \times 157^2}$	$K = \frac{26.62 \times 10^6}{25 \times 1000 \times 157^2}$
$= 0.057$	$= 0.043$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.057}{0.9}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.043}{0.9}}$
$l_a = 0.93$	$l_a = 0.95$
Since l_a is less than 0.95,	Since $l_a = 0.95$,
l_a is o.k	l_a is o.k
$Z = l_a d = 0.93 \times 157$	$Z = l_a d = 0.95 \times 157$
$= 146.01mm$	$= 149.15mm$
$A_{s\,req} = \frac{35.24 \times 10^6}{0.95 \times 410 \times 146.01}$	$A_{s\,req} = \frac{26.62 \times 10^6}{0.95 \times 410 \times 149.15}$
$= 6[illegible]\ mm^2/m$	$= 458.22\ mm^2/m$
Provide Y12mm @ 175 mm c/c	Provide Y12mm @ 200mm c/c
$A_{s\,prov} = 646\ mm^2/m$	$A_{s\,prov} = 566\ mm^2/m$
$A_{s\,min} = 0.13\%\,bh$	$A_{s\,min} = 0.13\%\,bh$
$= 0.0013 \times 1000 \times 200$	$= 0.0013 \times 1000 \times 200$
$= 260\ mm^2/m$	$= 260\ mm^2/m$
$A_{s\,prov} > A_{s\,min}$	Since ...

PROVISION IS

REFERENCE	CALCULATIONS	OUTPUT

DEFLECTION CHECK

We check for deflection at short - span - mid-span

Service stress $f_s = \frac{2}{3} \times f_y \times \frac{A_{s\,req}}{A_{s\,prov}}$

$= \frac{2}{3} \times 410 \times \frac{450.79}{566}$

$= 217.70\ N/mm^2$

Modification factor $= 0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}$

$\frac{M}{bd^2} = \frac{28.19 \times 10^6}{1000 \times 169^2} = 0.99$

$M_F = 0.55 + \frac{477 - 217.70}{120(0.9 + 0.99)}$

$= 1.69$

Limiting span depth ratio $= 1.69 \times 23$

$= 38.95$

Actual span depth ratio $= \frac{l_x}{d} = \frac{6000}{169}$

$= 35.50$

Since Actual span-depth ratio is less than limiting span depth ratio, deflection is O.K — DEFLECTION IS SATISFACTORY.

REFERENCE	CALCULATIONS	OUTPUT

Panel 2

Aspect ratio

$l_y/l_x = \frac{6000}{6000} = 1.0$

Since $l_y/l_x < 2.0$, the slab is a two-way spanning slab.

$C = 25mm$, $h = 175mm$

$b = 1000mm$

One short edge discontinuous

Loadings

Concrete own weight $= 0.175 \times 24 = 4.2 KN/m^2$

Finishes $= 1.2 \qquad = 1.2 KN/m^2$

Partitions $= 1.0 kN/m^2$

Imposed load $= 4.0 kN/m^2$

$n = 1.4 G_k + 1.6 Q_k$

$= 1.4(4.2 + 1.2 + 1) + 1.6(4)$

$= 15.36 kN/m^2$

Additional partition load

$= 0.41 \times 1.4 \times 3.425 \times (6 + 6)$ KN

$= 19.66$ KN

This can be assumed to be distributed over the entire area of $6.0 \times 6.0m$

Hence the load $= \frac{19.66}{6 \times 6} KN/m^2$

$= 5.55 KN/m^2$

Total UDL $= 5.55 + 15.36$

$= 20.91 KN/m^2$

SHORT SPAN

Continuous edge	Mid span
$\beta_{sx}^- = 0.039$	$\beta_{sx}^+ = 0.030$
$M_{sx}^- = \beta_{sx}^- n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.039 \times 20.91 \times 6^2$	$= 0.030 \times 20.91 \times 6^2$
$= 29.36 KNm$	$= 22.58 KNm$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144mm$
$k = M$	$k = M$

REFERENCE	CALCULATIONS	OUTPUT

SHORT SPAN

Continuous Edge

$K = \dfrac{29.36 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.057$

Lever arm

$la = 0.5 + \sqrt{0.25 - k/0.9}$

$= 0.5 + \sqrt{0.25 - \dfrac{0.057}{0.9}}$

$= 0.93$

$Z = lad \quad (0.93)$

$= 0.93 \times 144$

$= 133.92\,mm$

$A_{s,eq} = \dfrac{M}{0.95 f_y Z}$

$= \dfrac{29.36 \times 10^6}{0.95 \times 410 \times 133.92}$

$= [illegible]\,mm^2/m$

Provide Y12mm @ 175mm c/c

$A_{s,prov} = 646\,mm^2/m$

$A_{s,min} = 0.13\%\,bh$

$= \dfrac{0.13}{100} \times 1000 \times 175$

$= 227.5\,mm^2/m$

Since $A_{s,prov} > A_{s,min}$, provision is O.K

Mid span

$K = \dfrac{22.58 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.044$

Lever arm

$la = 0.5 + \sqrt{0.25 - k/0.9}$

$= 0.5 + \sqrt{0.25 - \dfrac{0.044}{0.9}}$

$= 0.95$

$Z = lad$

$= 0.95 \times 144$

$= 136.8\,mm$

$A_{s,eq} = \dfrac{M}{0.95 f_y Z}$

$= \dfrac{22.58 \times 10^6}{0.95 \times 410 \times 136.8}$

$= 423.77\,mm^2/m$

Provide Y12mm @ 200mm c/c

$A_{s,prov} = 566\,mm^2/m$

$A_{s,min} = 0.13\%\,bh$

$= \dfrac{0.13}{100} \times 1000 \times 175$

$= 227.5\,mm^2/m$

Since $A_{s,prov} > A_{s,min}$ provision is O.K

PROVISION is O.K.

LONG SPAN

Continuous Edge

$B_{sy}^{-} = 0.037$

$M_{sy}^{-} = B_{sy}^{-}\, n l_x^2$

$= 0.037 \times 20.91 \times 6^2$

$= 27.85\,KNm$

$d = h - c - \phi - \phi/2$

$= 175 - 25 - 12 - 12/2$

$= 132\,mm$

$K = \dfrac{M}{f_{cu} b d^2}$

$= \dfrac{27.85 \times 10^6}{[illegible]}$

Midspan

$B_{sy}^{+} = 0.028$

$M_{sy}^{+} = B_{sy}^{+}\, n l_x^2$

$= 0.028 \times 20.91 \times 6^2$

$= 21.07\,KNm$

$d = h - c - \phi - \phi/2$

$= 175 - 25 - 12 - 6$

$= 132\,mm$

$K = \dfrac{M}{f_{cu} b d^2}$

$= \dfrac{21.07 \times 10^6}{[illegible]}$

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous edge	mid-span
$l_a = 0.5 + \sqrt{0.25 - \frac{0.064}{0.9}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.048}{0.9}}$
$= 0.92$	$= 0.94$
$z = l_a d = 0.92 \times 132$	$z = l_a d = 0.94 \times 132$
$= 121.44mm$	$= 124.08mm$
$A_{s\,req} = \frac{27.85 \times 10^6}{0.95 \times 410 \times 121.44}$	$A_{s\,req} = \frac{21.07 \times 10^6}{0.95 \times 410 \times 124.08}$
$= 588.78 mm^2/m$	$= 435.97 mm^2/m$
Provide Y12mm @ 175mm c/c	Provide Y12mm @ 200mm c/c
$A_{s\,prov} = 646 mm^2/m$	$A_{s\,prov} = 566 mm^2/m$
$A_{s\,min} = 0.13\% bh$	$A_{s\,min} = 0.13\% bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5 mm^2/m$	$= 227.5 mm^2/m$
$A_{s\,prov} > A_{s\,min}$	$A_{s\,prov} > A_{s\,min}$

OUTPUT: Provision is O.K.

DEFLECTION CHECK

Deflection is checked at short-span- mid span

Service stress, $f_s = \frac{2}{3} \times f_y \times \frac{A_{s\,req}}{A_{s,prov}}$

$f_s = \frac{2}{3} \times 410 \times \frac{423.77}{566}$

$= 204.65 N/mm^2$

Modification factor $= 0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}$

$M.F = 0.55 + \frac{477 - 204.65}{120\left(0.9 + \frac{22.58 \times 10^6}{1000 \times 144^2}\right)}$

$\frac{M}{bd^2} = \frac{22.58 \times 10^6}{1000 \times 144^2} = 1.09$

$M.F = 1.69$

Limiting span $= 1.69 \times 26 = 43.94$

Actual span $= \frac{l_x}{d} = \frac{6000}{144} = 41.67$

Since actual span is less than limiting span, deflection is OK.

OUTPUT: Deflection is O.K.

REFERENCE	CALCULATIONS	OUTPUT

Panel 3

Aspect ratio

$l_y/l_x = \frac{6000}{4593} = 1.3$

Since $l_y/l_x < 2.0$ the slab is a 2-way spanning slab

$c = 25mm$, $h = 175mm$,

$b = 1000mm$, $n = 15.36 KN/m^2$

Additional Partition load

$= 2.41 \times 1.4 \times 3.425 \times (6 + 4.593)$ kN

$= 176.25$ kN

This can be assumed to be distributed over the entire area of $6m \times 4.593m$

Hence the UDL $= \frac{176.25}{6 \times 4.593} KN/m^2$

$= 6.39 KN/m^2$

Total UDL $= 6.4 + 15.36$

$= 21.76 KN/m^2$.

SHORT SPAN

Continuous Edge

$\beta_{sx}^- = 0.052$

$M_{sx}^- = \beta_{sx}^- n l_x^2$

$= 0.052 \times 21.76 \times 4.593^2$

$= 23.87 KNm$

$d = h - c - \phi/2$

$= 175 - 25 - 12/2$

$= 144mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{23.87 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.05$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.5 + \sqrt{0.25 - \frac{0.05}{0.9}}$

$= 0.94$

Since $l_a < 0.95$ l_a is O.K

$Z = l_a d = 0.94 \times 144$

$= 135.36 mm$

$A_s req = \frac{M}{0.95 f_y Z}$

Mid-Span

$\beta_{sx}^+ = 0.039$

$M_{sx}^+ = \beta_{sx}^+ n l_x^2$

$= 0.039 \times 21.76 \times 4.593^2$

$= 17.90 KNm$

$d = h - c - \phi/2$

$= 175 - 25 - 12/2$

$= 144mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{17.90 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.03$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.96$

Since $l_a > 0.95$, we use $l_a = 0.95$

$Z = l_a d = 0.95 \times 144$

$= 136.8mm$

$A_s req = \frac{M}{0.95 f_y Z}$

REFERENCE	CALCULATIONS	OUTPUT
	SHORT SPAN	

Continuous Edge

$A_s req = \frac{[illegible] \times 10^6}{0.95 \times 410 \times 135.36}$

$= 453.75 mm^2/m$

Provide [illegible] mm @ [illegible] mm c/c

$A_s prov = [illegible] mm^2/m$

$A_s min = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

Since $A_s prov > A_s min$

BS Mid span

$A_s req = \frac{17.90 \times 10^6}{0.95 \times 410 \times 136.8}$

$= 335.94 mm^2/m$

Provide Y12mm @ 250mm c/c

$A_s prov = 452 mm^2/m$

$A_s min = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

Since $A_s prov > A_s min$ — PROVISION IS O.K.

LONG SPAN

Continuous Edge

[illegible]

$= 0.034$

$= [illegible] \times 21.76 \times 4.593^2$

[illegible]

$= 0.5 + \sqrt{0.25 - \frac{0.04}{0.9}}$

[illegible] O.K.

$A_s req = \frac{[illegible]}{0.95 \times 410 \times 125.4}$

$= 347 mm^2/m$

Provide [illegible] mm c/c

$A_s prov = 452 mm^2/m$

$A_s min = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

$A_s prov > A_s min$

Mid span

$\beta_{sy} = 0.028$

$M_{sy} = \beta_{sy} n l_x^2$

$= 0.028 \times 21.76 \times 4.593^2$

$= 12.85 KNm$

$d = h - c - \phi - \phi_s$

$= 175 - 25 - 12 - 6$

$= 132 mm$

$K = \frac{12.85 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.03$

$la = 0.5 + \sqrt{0.25 - \frac{0.03}{0.9}}$

$= 0.96$

Since $la > 0.95$, we use $la = 0.95$

$Z = lad = 0.95 \times 132$

$= 125.4 mm$

$A_s req = \frac{12.85 \times 10^6}{0.95 \times 410 \times 125.4}$

$= 263.08 mm^2/m$

Provide Y12mm @ 300mm c/c

$A_s prov = 377 mm^2/m$

$A_s min = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

Since $A_s prov > A_s min$ — PROVISION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

DEFLECTION CHECK

We check for deflection on the short-span mid span

Service stress, $f_s = \frac{2}{3} \times f_y \times \frac{A_{s,req}}{A_{s\,prov}}$

$= \frac{2}{3} \times 410 \times \frac{335.94}{452}$

$= 203.15\ N/mm^2$

Modification factor $= 0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}$

$\frac{M}{bd^2} = \frac{[illegible] \times 10^6}{1000 \times 144^2} = 0.86$

M.F. $= 0.55 + \frac{477 - 203.15}{120(0.9 + 0.86)}$

$= 1.8[illegible]$

Limiting span/depth ratio $= 1.85 \times 26$

$= 48.1$

Actual $L/d = \frac{4593}{144} = 31.9$

Since actual L/d is less than limiting L/d, deflection is O.K.

DEFLECTION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

PANEL 5

$\frac{l_y}{l_x} = \frac{6000}{6000} = 1$

Since $l_y/l_x < 2$, the slab will be designed as a two-way spanning slab

$n = 15.36\ kN/m^2$

$h = 175mm$

$c = 25$, $b = 1000mm$

SHORT SPAN

Continuous edge	Midspan
[illegible]	$\beta_{sx}^{+} = 0.029$
[illegible]	$M_{sx}^{+} = \beta_{sx}^{+} n l_x^2$
[illegible]	$= 0.029 \times 15.36 \times 6^2$
[illegible]	$= 16.04\ kNm$
[illegible]	$d = h - c - \phi/2$
[illegible]	$= 175 - 25 - 12/2$
[illegible]	$= 144mm$
$\frac{\ldots \times 10^6}{\ldots \times 1000 \times 144^2}$	$K = \frac{M}{f_{cu}bd^2} = \frac{16.04 \times 10^6}{25 \times 1000 \times 144^2}$
	$= 0.03$
	$Z = l_a d$
$+ 0.5$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.03}{0.9}}$
	$= 0.96$
$l_a = 0.95$, l_a is o.k	Since $l_a > 0.95$
$l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8mm$	$= 136.8mm$
$A_{s\,req} = \frac{M}{0.95 f_y Z}$	$A_{s\,req} = \frac{M}{0.95 f_y Z}$
$= \frac{21.57 \times 10^6}{0.95 \times 410 \times 136.8}$	$= \frac{16.04 \times 10^6}{0.95 \times 410 \times 136.8}$
$404.8mm^2/m$	$= 301.03mm^2/m$
$A_{s\,min} = 0.0013 \times 1000 \times 175$	$A_{s\,min} = 0.13\% bh$
	$= 0.0013 \times 1000 \times 175$
$= 227.5mm^2/m$	$= 227.5mm^2/m$
$A_{s\,min} < A_{s\,req}$	Since $A_{s\,min} < A_{s\,req}$ we provide
Provide Y12mm @ 250mm c/c	Y12mm @ 300mm c/c
$A_{s\,prov} = 452mm^2/m$	$A_{s\,prov} = 377mm^2/m$

PROVISION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge	Mid span
[illegible]	$\beta^{+}_{sy} = 0.028$
$M = \beta_{sy} n l_x^2$	$M^{+}_{sy} = \beta^{+}_{sy} n l_x^2$
$= 0.031 \times 15.36 \times 6^2$	$= 0.028 \times 15.36 \times 6^2$
[illegible] KNm	$= 15.48\,KNm$
$d = h - c - \phi/_2 - \phi$	$d = h - c - \phi/_2 - \phi$
$= 175 - 25 - 6 - 12$	$= 175 - 25 - 6 - 12$
[illegible]	$= 132mm$
[illegible]	$K = \frac{15.48 \times 10^6}{25 \times 1000 \times 132^2}$
	$= 0.04$
[illegible]	$l_a = 0.5 + \sqrt{0.25 - \frac{0.04}{0.9}}$
	$l_a = 0.95$
[illegible] O.K	Since $l_a = 0.95$, l_a is O.K
[illegible]	$Z = l_a d = 0.95 \times 132$
[illegible]	$= 125.40mm$
[illegible]	$A_{s\,req} = \frac{15.48 \times 10^6}{0.95 \times 410 \times 125.40}$
$= 423.35\,mm^2/m$	$= 316.93\,mm^2/m$
$A_{s\,min} = 0.13\% bh$	$A_{s\,min} = 0.13\% bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5\,mm^2/m$	$= 227.5\,mm^2/m$
Since $A_{s\,min} < A_{s\,req}$	Since $A_{s\,min} < A_{s\,req}$
Use $A_{s\,req}$ for provision	Use $A_{s\,req}$ for provision
Provide Y12mm @ 200mm c/c	Provide Y12mm @ 300mm c/c
$A_{s\,prov} = 566\,mm^2/m$	$A_{s\,prov} = 377\,mm^2/m$

PROVIDE [illegible]

DEFLECTION CHECK:

We check for deflection at the short span midspan

Service stress, $f_s = \frac{2}{3} \times f_y \times \frac{A_{s\,req}}{A_{s\,prov}}$

$$= \frac{2}{3} \times 410 \times \frac{301.03}{377}$$

$$= 213.46\,N/mm^2$$

Modification factor $= 0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}$

$$\frac{M}{bd^2} = \frac{16.04 \times 10^6}{1000 \times 144^2} = 0.77$$

$$M.F = 0.55 + \frac{477 - 213.46}{120(0.9 + 0.77)} = 1.87$$

REFERENCE	CALCULATIONS	OUTPUT
	Limiting span/depth ratio = 1.87 × 26	
	= 48.62	
	Actual span/depth ratio = $\frac{6000}{144}$	
	= 41.67	
	since limiting span > actual span,	DEFLECTION IS
	Deflection is O.K	OK

REFERENCE	CALCULATIONS	OUTPUT
	PANEL 6	

$\frac{l_y}{l_x} = \frac{6000}{6000} = \underline{1}$

$l_y/l_x < 2$, the slab is a 2-way spanning slab.

$c = 25mm$, $h = 175mm$

$b = 1000mm$,

$n = 15.36 kN/m^2$.

SHORT SPAN

Continuous Edge

$\beta_{sx} = 0.031$

$M_{sx} = \beta_{sx} n l_x^2$

$= 0.031 \times 15.36 \times 6^2$

$= 17.14 kNm$

$d = h - c - \phi/2$

$= 175 - 25 - 12/2$

$= 144mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{17.14 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.03$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.5 + \sqrt{0.25 - 0.03/0.9}$

$= 0.96$

Since $l_a > 0.95$, we use $l_a = 0.95$

$Z = l_a d = 0.95 \times 144$

[illegible]

$A_{s\,req} = \frac{M}{0.95 f_y Z}$

$= \frac{17.14 \times 10^6}{[illegible] \times 410 \times 136.8}$

$A_{s\,req} = 321.67 mm^2/m$

Provide [illegible]mm @ [illegible]0mm c/c

[illegible]

$A_{s\,min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5mm$

$A_{s\,prov} > A_{s\,min}$

Mid-span

$\beta^+_{sx} = 0.024$

$M^+_{sx} = \beta^+_{sx} n l_x^2$

$= 0.024 \times 15.36 \times 6$

$= 13.27 kNm$

$d = h - c - \phi/2$

$= 175 - 25 - 12/2$

$= 144mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{13.27 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.03$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.5 + \sqrt{0.25 - 0.03/0.9}$

$= 0.96$

Since $l_a > 0.95$, we use $l_a = 0.95$

$Z = l_a d$

$= 0.95 \times 144$

$= 136.8mm$

$A_{s\,req} = \frac{M}{0.95 f_y Z}$

$= \frac{13.27 \times 10^6}{0.95 \times 410 \times 136.8}$

$A_{s\,req} = 249.04 mm^2/m$

Provide Y12mm @ 300mm c/c

$A_{s\,prov} = 377 mm^2/m$

$A_{s\,min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5mm$

$A_{s\,prov} > A_{s\,min}$

OUTPUT: PROVISION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge	Midspan
[illegible]	$\beta^{+}_{sy} = 0.024$
[illegible]	$M^{+}_{sy} = \beta^{+}_{sy} n l_x^2$
[illegible] × 36	$= 0.024 \times 15.36 \times 36$
[illegible]	$= 13.27 kN m$
[illegible]	$d = h - c - \phi - \phi/2$
[illegible]	$= 175 - 25 - 12 - 6$
[illegible]	$= 132 mm$
[illegible]	$K = \dfrac{13.27 \times 10^6}{25 \times 1000 \times 132^2}$
[illegible]	$= 0.03$
[illegible]	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$
[illegible]	$= 0.96$
[illegible] is OK	Since $l_a > 0.96$, we use $l_a = 0.95$
[illegible] × 132	$Z = l_a d = 0.95 \times 132$
[illegible] mm	$= 125.4 mm$
[illegible] × 410 × 125.4	$A_{s\,req} = \dfrac{13.27 \times 10^6}{0.95 \times 410 \times 125.4}$
[illegible] mm²/m	$= 271.69 mm^2/m$
[illegible] mm c/c	Provide Y12 mm @ 300 mm c/c
[illegible] mm²/m	$A_{s\,prov} = 377 mm^2/m$
[illegible]	$A_{s\,min} = 0.13\% bh$
[illegible] mm²/m	$= 227.5 mm^2/m$
[illegible]	$A_{s\,prov} > A_{s\,min}$

PROVISION IS O.K

DEFLECTION CHECK

We check for deflection at the short span [illegible]

[illegible] stress, $f_s = \dfrac{2}{3} \times f_y \times \dfrac{A_{s\,req}}{A_{s\,prov}}$

$= \dfrac{2}{3} \times 410 \times \dfrac{249.04}{377}$

$= 180.6 N/mm^2$

[illegible] factor $= 0.55 + \dfrac{477 - f_s}{120(0.9 + M/bd^2)}$

$\dfrac{M}{bd^2} = \dfrac{13.27 \times 10^6}{1000 \times 144^2}$

$= 0.64$

REFERENCE	CALCULATIONS	OUT-PUT
	$MF = 0.55 + \dfrac{477 - 182.6}{120(0.9 + 0.64)}$	
	$= 2.15$	
	$MF = 2$	
	Limiting span-depth ratio $= 2 \times 26$ $= 52$	
	Actual span-depth ratio $= \dfrac{l_x}{d} = \dfrac{6000}{144}$ $= 41.67$	
	Since actual span-depth ratio is less than limiting span-depth ratio, deflection is O K	DEFLECTION IS O.K

REFERENCE	CALCULATIONS	OUTPUT

[illegible]

Aspect ratio

$l_y / l_x = \frac{7183}{6000} = 1.2$

since $l_y/l_x < 2$, the slab is a 2-way spanning slab.

$c = 25mm$, $h = 175mm$,
$b = 1000mm$, $n = 15.36 KN/m^2$.

SHORT SPAN

[illegible]	Midspan
[illegible]	$\beta^+_{sx} = 0.032$
[illegible]	$M^+_{sx} = \beta^+_{sx} n l_x^2$
$\ldots \times 6^2$	$= 0.032 \times 15.36 \times 6^2$
[illegible]	$= 17.69 KNm$
[illegible]	$d = h - c - \phi/2$
[illegible]	$= 175 - 25 - 12/2$
[illegible]	$= 144mm$
$= \frac{23.22 \times 10^6}{\ldots \times 1000 \times 144^2}$	$K = \frac{17.69 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.045$	$= 0.034$
$\ldots + \sqrt{0.25 - K/0.9}$	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$
$\ldots + \sqrt{\ldots - \frac{0.045}{0.9}}$	$= 0.5 + \sqrt{0.25 - \frac{0.034}{0.9}}$
[illegible]	$= 0.96$
$\ldots 0.95$, l_a is O.K	since $l_a > 0.95$, we use $l_a = 0.95$
$\ldots = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8mm$	$= 136.8$
$\frac{M}{0.95 f_y Z}$	$As_{req} = \frac{M}{0.95 f_y Z}$
$= \frac{23.22 \times 10^6}{0.95 \times 410 \times 136.8}$	$= \frac{17.69 \times 10^6}{0.95 \times 410 \times 136.8}$
$= 435.78 mm^2/m$	$= 332 mm^2/m$
$\ldots$ Y12mm @ 250mm c/c	Provide Y12mm @ 300mm c/c
$\ldots 452 mm^2/m$	$A_{s prov} = 377 mm^2/m$.
$\ldots 0.13\% bh$	$A_{s min} = 0.13\% bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5 mm^2/m$	$= 227.5 mm^2/m$
$A_{s prov} > A_{s min}$	$A_{s prov} > A_{s min}$

PROVISION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge

$\beta_{sy}^- = 0.032$

$M_{sy}^- = \beta_{sy}^- n l_x^2$

$= 0.032 \times 15.36 \times 6^2$

$= 17.69\,kNm$

$d = h - c - \phi/2 - \phi$

$= 175 - 25 - 12 - 6$

$= 132\,mm$

$K = \frac{M}{f_{cu} b d^2}$

$= \frac{17.69 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.04$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.95$

Since l_a = 0.95, l_a is O.K

$Z = l_a d = 0.95 \times 132$

$= 125.4\,mm$

$As_{req} = \frac{M}{0.95 f_y Z}$

$= \frac{17.69 \times 10^6}{0.95 \times 410 \times 125.4}$

$= 362.18\,mm^2/m$

Provide Y12mm @ 250mm c/c

$As_{prov} = 452\,mm^2/m$

$As_{min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5\,mm^2/m$

$As_{prov} > As_{min}$

Mid Span

$\beta_{sy}^+ = 0.024$

$M_{sy}^+ = \beta_{sy}^+ n l_x^2$

$= 0.024 \times 15.36 \times 6^2$

$= 13.27\,kNm$

$d = h - c - \phi/2 - \phi$

$= 175 - 25 - 12 - 6$

$= 132\,mm$

$K = \frac{M}{f_{cu} b d^2}$

$= \frac{17.69 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.03$

$l_a = 0.5 + \sqrt{0.25 - K/0.9}$

$= 0.96$

Since l_a > 0.96, we use $l_a = 0.95$

$Z = l_a d = 0.95 \times 132$

$= 125.4\,mm$

$As_{req} = \frac{M}{0.95 f_y Z}$

$= \frac{13.27 \times 10^6}{0.95 \times 410 \times 125.4}$

$= 271.69\,mm^2/m$

Provide Y12mm @ 300mm c/c

$As_{prov} = 377\,mm^2/m$

$As_{min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5\,mm^2/m$

$As_{prov} > As_{min}$ — PROVISION IS O.K

DEFLECTION CHECK

We check for deflection at the short-span mid-span

Service stress, $f_s = \frac{2}{3} \times f_y \times \frac{As_{req}}{As_{prov}}$

$= \frac{2}{3} \times 410 \times \frac{332}{377}$

$= 240.7\,N/mm^2$

SOLID SLAB DESIGN TO EUROCODE.

Reference	Calculation	Output.

Panel 1

$l_y/l_x = \frac{6000}{6000} = 1.0$

Since $l_y/l_x < 2.0$, the Slab is a two-way Spanning Slab.

$C = 25mm$, $h = 200mm$.

$b = 1000mm$.

[illegible] Partition Load :-

= [illegible] 47 × 1.35 × 3.425 × (6.0 + 6.0) kN.

= 192.53 kN.

[illegible] be assumed to distributed Over the [illegible] area of 6.0 × 6.0m.

Hence, the UDL $= \frac{192.53}{6\times6}$ kN/m²

$= 5.35$ kN/m²

Total UDL = $5.35 + 15.45 = 20.8$ kN/m²

Short Span

Continuous Edge	Mid-Span
$\beta_{sx}^- = 0.047$	$\beta_{sx}^+ = 0.036$
$M_{sx}^- = \beta_{sx} n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.047 \times 20.8 \times 6^2$	$= 0.036 \times 20.8 \times 6^2$
$= 35.19$ kN.m.	$= 26.96$ kN.m
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 200 - 25 - 12/2$	$= 200 - 25 - 12/2$
$= 169mm$	$= 169mm$
$K = \frac{M}{f_{ck} b d^2} = \frac{35.19\times10^6}{25\times1000\times169^2}$	$= \frac{26.96\times10^6}{25\times1000\times169^2}$
$= 0.049$	$= 0.038$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.5 + \sqrt{0.25 - \frac{0.049}{1.134}}$	$= 0.5 + \sqrt{0.25 - \frac{0.038}{1.134}}$

Reference	Calculations		Output
	Short Span		
	Continuous Edge	Mid-Span.	
	[illegible] 0.95, la is 0.k.	Since $l_a > 0.95$, $l_a = 0.95$	
	$Z = l_a d = 0.95 \times 169$	$Z = l_a d = 0.95 \times 169$	
	$Z = 160.55$	$Z = 160.55$	
	$A_{sreq} = \frac{M}{0.87 f_y z}$	$A_{sreq} = \frac{M}{0.87 f_y z}$	
	$= \frac{[illegible] \times 10^6}{0.87 \times [illegible] \times 160.55}$	$= \frac{26.96 \times 10^6}{0.87 \times 410 \times 160.55}$	
	$= [illegible]\ mm^2/m$	$= 470.77\ mm^2/m.$	
	Provide [illegible] mm c/c	Provide Y12mm @ 200 mm c/c	
	$A_{sprov} = 646\ mm^2/m$	$A_{sprov} = 566\ mm^2/m.$	
	$A_{smin} = 0.0013bd$	$A_{smin} = 0.0013bd$	Provision is O.K.
	$A_{smin} = 219.7\ mm^2/m$	$A_{smin} = 219.7\ mm^2/m.$	
	Since $A_{sprov} > A_{smin}$	Since $A_{sprov} > A_{smin}$	
	Long Span		
	Continuous Edge	Mid-Span.	
	[illegible]	$\beta_{sy}^{+} = 0.034$	
	$M_{sy} = \beta_{sy} n l_x^2$	$M_{sy}^{+} = \beta_{sy}^{+} n l_x^2$	
	$= [illegible] \times 26.6 \times 6^2$	$= 0.034 \times 20.8 \times 6^2$	
	$= [illegible]\ kN\,m$	$= 25.46\ kN\,m$	
	$d = h - c - \phi/2 - \phi$	$d = h - c - \phi/2 - \phi$	
	$= 200 - 25 - 6 - 12$	$= 200 - 25 - 6 - 12$	
	$= 157mm$	$= 157mm.$	
	$K = \frac{[illegible] \times 10^6}{25 \times 1000 \times 157^2}$	$K = \frac{25.46 \times 10^6}{25 \times 1000 \times 157^2}$	
	$= 0.055$	$= 0.041$	
	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	
	$= 0.5 + \sqrt{0.25 - \frac{0.055}{1.134}}$	$= 0.5 + \sqrt{0.25 - \frac{0.041}{0.9}}$	

Reference	Calculations	Output

Long Span

Continuous Edge	Mid-Span.
[illegible] la is equal to 0.95,	Since la = 0.95, la is
[illegible]	O.K.
[illegible] 0.95×157	$Z = l_a d = 0.95 \times 157$
149.15	$= 149.15$
[illegible] M	$A_{sreq} = \frac{M}{0.87 f_{yk} z}$
[illegible] f_{yk} [illegible]	
[illegible] 1×10^6 / 0.87×410×149.15	$= \frac{25.46 \times 10^6}{0.87 \times 410 \times 149.15}$
655.44mm²/m	$= 478.56\ mm^2/m$.
Provide Y12mm @ 150mm c/c	Provide Y12mm @ 200mm c/c
[illegible] 754mm²/m	$A_{sprov} = 566 mm^2/m$.
[illegible] 0.0013bd	$A_{smin} = 0.0013bd$
[illegible] ×1000×157	$= 0.0013 \times 1000 \times 157$
[illegible] mm²/m	$= 204.1 mm^2/m$.
[illegible] A_{smin}	Since $A_{sprov} > A_{smin}$

Provision is O.K.

Deflection Check

[illegible] to deflection at Short Span-

[illegible]

[illegible] Steel Stress $\sigma_s = \frac{5}{8} \times f_{yk} \times \frac{A_{sreq}}{A_{sprov}}$

$\frac{5}{8} \times 410 \times \frac{496.77}{566} = 213.14 N/mm^2$

Modification factor $= \frac{310}{\sigma_s} = \frac{310}{213.14} = 1.45$

Interpolating to get the basic Span ratio

0.5	26	$\frac{0.28 - 0.5}{0.15 - 0.5} = \frac{x - 28}{38 - 28}$
0.2[illegible]	x	
[illegible]	38	$x = 34.29$

[illegible] $= 1.45 \times 34.29 = 49.72$

[illegible] Span $\frac{l_n}{d} = \frac{6000}{169} = 35.5$

[illegible] Actual Span is less than limiting Span,

Deflection is Satisfactory.

Reference	Calculations	Output

PANEL 2:

$l_y/l_x = \frac{6000}{6000} = 1.0$

Since $l_y/l_x < 2.0$, the Slab is a two Way Spanning Slab

$C = 25mm$, $h = 175mm$.

$b = 1000mm$.

[illegible] Load;

[illegible] $55 \times 3.425 \times (6.0+6.0)$ kN

[illegible]

[illegible] assumed to distributed Over the [illegible] $6.0 \times 6.0m$

[illegible] $= \frac{192.53}{6 \times 6}$ kN/m²

$= 5.35$ kN/m²

[illegible] $5.35 + 14.64 = 19.99$ kN/m²

SHORT SPAN

Continuous Edge	Mid-Span.
β [illegible] 0.039	$\beta_{sx}^{+} = 0.029$.
$M_{sx}^{-} = \beta_{sx}^{-} n l_x^2$	$M_{sx}^{+} = \beta_{sx}^{+} n l_x^2$
$= 0.039 \times 19.99 \times 6^2$	$= 0.029 \times 19.99 \times 6^2$
$= 28.07$ kN·m	$= 20.87$ kN·m
$d = h - c - \phi/2$	$d = h - c - \phi/2$.
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$.
$= 144mm$	$= 144mm$.
$K = \frac{M}{f_{ck} b d^2}$	$K = \frac{M}{f_{ck} b d^2}$
$= \frac{28.07 \times 10^6}{25 \times 1000 \times 144^2}$	$= \frac{20.87 \times 10^6}{25 \times 1000 \times 144^2}$
[illegible]	$= 0.040$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.5 + \sqrt{0.25 - \frac{0.054}{1.134}}$	$= 0.5 + \sqrt{0.25 - \frac{0.04}{1.134}}$
[illegible]	$= 0.96$
Since l_a [illegible], l_a is O.K.	Since $l_a > 0.96$, Use $l_a = 0.95$

Reference	Calculations	Output

Short Span

Continuous Edge

$z = l_a d = 0.95 \times 144$

$= 136.8$

$A_{sreq} = \frac{M}{0.87 f_{yk} z}$

$= \frac{20.67 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 515.25 mm^2/m$

Provide Y12mm @ 175mm c/c

$A_{sprov} = 646 mm^2/m$

$A_{smin} = 0.0013bd$

$A_{smin} = 0.0013 \times 1000 \times 144$

$= 187.2 mm^2/m$

Since $A_{sprov} > A_{smin}$.

Mid-Span.

$z = l_a d = 0.95 \times 144$

$= 136.8$

$A_{sreq} = \frac{M}{0.87 f_{yk} z}$

$= \frac{20.87 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 427.69 mm^2/m$.

Provide Y12mm @ 200mm c/c.

$A_{sprov} = 566 mm^2/m$.

$A_{smin} = 0.0013bd$

$A_{smin} = 0.0013 \times 1000 \times 144$

$= 187.2 mm^2/m$.

Since $A_{sprov} > A_{smin}$.

Provision is O.K.

Long Span

Continuous Edge

$l_a = 0.94$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.06}{1.134}}$

$l_a < 0.95$,

Mid-Span.

$\beta_{sy}^{+} = 0.028$

$M_{sy}^{+} = \beta_{sy}^{+} n l_x^2$.

$= 0.028 \times 19.99 \times 6^2$

$= 20.15 kN \cdot m$.

$d = h - c - \phi/2 - \phi$

$= 175 - 25 - 12 - 6$

$= 132mm$.

$k = \frac{M}{f_{ck} bd^2}$

$= \frac{20.15 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.046$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.046}{1.134}}$

$= 0.96$

Since $l_a = 0.96 > 0.95$,

Use $l_a = 0.95$.

Reference | Calculations | Output

Long Span

Continuous Edge	Mid-Span
[illegible] 0.95×132	$Z = l_a d = 0.95 \times 132$
= 125.4	$= 125.4$
[illegible] $\frac{M}{0.87 f_{yk} Z}$	$A_{sreq} = \frac{M}{0.87 f_{yk} Z}$
[illegible]	$= \frac{20.15 \times 10^6}{0.87 \times 410 \times 125.4}$
[illegible] mm²/m	$= 450.48\,mm^2/m$
[illegible] 175mm c/c	Provide Y_{12}mm @ 200 mm c/c
[illegible]	$A_{sprov} = 566\,mm^2/m$
[illegible]	$A_{smin} = 0.0013bd$
[illegible]	$= 0.0013 \times 1000 \times 132$
[illegible]	$= 171.6\,mm^2/m$
[illegible]	Since $A_{sprov} > A_{smin}$

Provision is O.K.

Deflection

[illegible] deflection at the Short Span - Mid Span.

[illegible] $= \frac{5}{8} \times f_{yk} \times \frac{A_{sreq}}{A_{sprov}}$

[illegible] $\frac{427.69}{566} = 193.63\,N/mm^2$

[illegible] factor $\frac{310}{\sigma_s} = \frac{310}{193.63} = 1.6$

[illegible] $\frac{427.69 \times 100}{1000 \times 144} = 0.28$

[illegible] I get the basic Span ratio

[illegible] 8 $\quad \frac{0.28 - 0.5}{0.15 - 0.5} = \frac{x - 28}{38 - 28}$

$x = 34.30$

[illegible] $= 1.6 \times 34.3 = 54.88$

[illegible] $\frac{6000}{144} = 41.67$

[illegible] Span is less than limiting

[illegible] deflection is O.K

Deflection is Satisfactory!!!

Reference	Calculations	Output

Ec 3-1

$$l_y/l_x = \frac{6000}{4593} = 1.30$$

Since $l_y/l_x < 2.0$, the is a 2-Way Spanning Slab.

$C = 25mm$, $h = 175mm$.

$b = 1000mm$, $n = 14.64 kN/m^2$

Additional Partition load

$= 1.4 \times 1.35 \times 3.425 \times (6.0 + 4.593)$ kN

$= 169.96$ kN.

This can be assumed to be distributed over the area of $6 \times 4.593m$.

Hence, the UDL $= \frac{169.96}{6 \times 4.593} = 6.17 kN/m^2$

Total UDL $= 6.17 + 14.64 = 20.81 kN/m^2$.

Short Span

Continuous Edge	Mid-Span.
$\beta_{sx}^- = 0.052$	$\beta_{sx}^+ = 0.039$.
$M_{sx}^- = \beta_{sx}^- n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$.
$= 0.052 \times 20.81 \times 4.593^2$	$= 0.039 \times 20.81 \times 4.593^2$
$= 22.83$ kN m.	$= 17.12$ kN.m.
$d = h - c - \phi/2$	$d = h - c - \phi/2$.
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$.	$= 144mm$.
$k = \frac{M}{f_{ck} b d^2} = \frac{22.83 \times 10^6}{25 \times 1000 \times 144^2}$	$k = \frac{M}{f_{ck} b d^2} = \frac{17.12 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.044$	$= 0.033$.
$l_a = 0.5 + \sqrt{0.25 - k/1.134}$	$l_a = 0.5 + \sqrt{0.25 - k/1.134}$
$= 0.5 + \sqrt{0.25 - \frac{0.044}{1.134}}$	$= 0.5 + \sqrt{0.25 - \frac{0.033}{1.134}}$
$= 0.96$	$= 0.97$.
Since $l_a = 0.96 > 0.95$,	Since $l_a = 0.97 > 0.95$,
Use $l_a = 0.95$	Use $l_a = 0.95$.

Reference	Calculation	Output

Short Span

Mid-Span.

$Z = l_a d = 0.95 \times 144$

$= 136.8$

$A_{sreq} = \dfrac{M}{0.87 f_{yk} Z}$

$= \dfrac{17.12 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 350.84\ mm^2/m.$

Provide $Y_{12}mm$ @ 250mm c/c.

$A_{sprov} = 452\ mm^2/m.$

$A_{smin} = 0.0013bd.$

$= 0.0013 \times 1000 \times 144$

$= 187.2\ mm^2/m.$

Since $A_{sprov} > A_{smin}$. — Provision is O.K.

Long Span

Mid-Span.

$B_{sy}^{+} = 0.028$

$M_{sy}^{+} = B_{sy}^{+} n l_x^2$

$= 0.028 \times 20.81 \times 4.593^2$

$= 12.29\ kNm.$

$d = h - c - \phi - \phi/2.$

$= 175.25 - 12 - 6.$

$= 132\ mm.$

$k = \dfrac{12.29 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.028$

$l_a = 0.5 + \sqrt{0.25 - \dfrac{0.028}{1.134}}$

$= 0.97$

Since $l_a = 0.97 > 0.95,$

Use $l_a = 0.95.$

Reference	Calculations	Output

Long Span

Continuous Edge	Mid-Span.
$Z = 0.95 \times 132$	$Z = l_a d = 0.95 \times 132.$
125.4	$Z = 125.4$
$A_{s,req} = \frac{M}{0.87 f_y Z}$	$A_{sreq} = \frac{M}{0.87 f_y Z}$
$= \frac{16.24 \times 10^6}{0.87 \times 410 \times 125.4}$	$= \frac{12.29 \times 10^6}{0.87 \times 410 \times 125.4}$
$= 363.01\ mm^2/m.$	$= 274.76\ mm^2/m.$
Provide Y12mm @ 250mm c/c	Provide Y12mm @ 250mm c/c
$A_{sprov} = 452mm^2/m.$	$A_{sprov} = 452mm^2/m.$
$A_{smin} = 0.0013bd$	$A_{smin} = 0.0013bd.$
$= 0.0013 \times 132 \times 1000$	$= 0.0013 \times 132 \times 1000$
$= 171.6\ mm^2/m$	$= 171.6\ mm^2/m.$
Since $A_{sprov} > A_{smin}$	Since $A_{sprov} > A_{smin}$.

Provision is O.K.

Deflection Check.

Take check for deflection on the Short Span –

Mid Span

Service Stress $\sigma_s = \frac{5}{8} \times f_y \times \frac{A_{s,req}}{A_{sprov}}$

$= \frac{5}{8} \times 410 \times \frac{350.84}{452}$ $= 198.90\ N/mm^2.$

Modification factor $= \frac{310}{\sigma_s} = \frac{310}{198.90} = 1.56$

Percentage $= \frac{350.84}{1000 \times 144} \times 100 = 0.24\%$

Interpolating to get Basic Span Ratio:-

$0.5 \quad 28$

$0.15 \quad 38$

$0.24 \quad x$

$\frac{0.24 - 0.5}{0.15 - 0.5} = \frac{x - 28}{38 - 28}$

$x = 35.43$

Limiting Span $= 1.56 \times 35.43 = 55.27$

Actual Span $= \frac{4593}{144} = 31.895$

Actual Span is less than Limiting Span,

Deflection is Satisfactory

REFERENCE	CALCULATION	OUTPUT

$l_y/l_x = \frac{6000}{6000} = 1$

Since $l_y/l_x < 2$, the slab is a 2-way spanning slab.

$n = 14.64\ kN/m^2$ $h = 175mm$

$C = 25mm$ $b = 1000mm$

SHORT SPAN

Continuous Edge	Mid-Span
[illegible]	$\beta_{sx}^{+} = 0.029$
[illegible]	$M_{sx}^{+} = \beta_{sx}^{+} n l_x^2$
[illegible]	$= 0.029 \times 14.64 \times 6^2$
[illegible]	$= 15.28 kN \cdot m$
[illegible]	$d = h - c - \phi/2$
[illegible]	$= 175 - 25 - 12/2$
[illegible]	$= 144 mm$
[illegible]	$K = \frac{M}{f_{cu} b d^2} = \frac{15.28 \times 10^6}{25 \times 1000 \times 144^2}$
0.04	$= 0.03$
[illegible]	$Z = l_a d$
[illegible]	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
[illegible]	$= 0.5 + \sqrt{0.25 - 0.03/1.134}$
0.95	$= 0.97$
[illegible] 0.95	Use $l_a = 0.95$ Since $0.97 > 0.95$
[illegible]	$z = l_a d = 0.95 \times 144$
[illegible]	$= 136.8$
[illegible]	$A_{sreq} = \frac{M}{0.87 f_y z}$
[illegible]	$= \frac{15.28 \times 10^6}{0.87 \times 410 \times 136.8}$
[illegible]	$= 313.1\ mm^2/m$
[illegible] bd	$A_{smin} = 0.0013 \times bd$
[illegible] $\times 144$	$= 0.0013 \times 1000 \times 144$

REFERENCE	CALCULATION	OUTPUT.

SHORT SPAN

Mid - Span

$AS_{min} < AS_{req}$

Provide for As_{req}

Y12 @ 250mm c/c

$AS_{prov} = 452\ mm^2/m$

PROVISION IS OK

LONG SPAN

Mid - Span

$\beta_{sy}^{+} = 0.028$

$Ms_y^{+} = \beta_{sy}^{+}\, n l_x^2$

$= 0.028 \times 14.64 \times 6^2$

$= 14.76\ KN.m$

$d = h - c - \phi/2 - \phi$

$= 175 - 25 - 12/2 - 12$

$= 132mm$

$$K = \frac{M}{f_{cu}\, b\, d^2}$$

$$= \frac{14.76 \times 10^6}{25 \times 1000 \times 132^2} = 0.03$$

$la = 0.5 + \sqrt{0.25 - K/1.134}$

$= 0.5 + \sqrt{0.25 - 0.03/1.134}$

$= 0.97$

Use $la = 0.95$; Since $0.97 > 0.95$

$Z = lad = 0.95 \times 132$

$= 125.4$

$$As_{req} = \frac{M}{0.87\, f_y\, Z}$$

$$= \frac{14.76 \times 10^6}{0.87 \times 410 \times 125.4}$$

$= 330 ...$

Reference	Calculation	Output

LONG SPAN

Continuous Edge	Mid - Span
	$As_{min} = 0.13\% bd$
	$= 0.0013 \times 1000 \times 132$
	$= 171.6 mm^2/m$
	$As_{min} < As_{req}$
	Provide using As_{req}
	Prov. Y12 @ 250 mm c/c
	$As_{prov} = 452 mm^2/m$

PROVISION IS O.K.

DEFLECTION CHECK.

Mid-span

$\frac{5}{8} \times 410 \times \frac{313.1}{452}$

177.5 N/mm²

$\frac{310}{177.5} = 1.75$

$\frac{6000}{144} = 41.67$

Ratio ($As/bd \times 100\%$ $\frac{313.1}{1000 \times 144} \times 100 = 0.2$

$= \frac{(2 - 0.5}{0.15 - 0.5}$; $\frac{x - 28}{10} = \frac{+0.3}{+0.35}$

8.6 , $x = 8.6 + 28 = 36.6$

6.405

Span ; DEFLECTION IS O.K.

DEFLECTION IS O.K.

Reference	Calculations	Output

Panel 6:

$l_y/l_x = \frac{6000}{6000} = 1$

Since $l_y/l_x < 2$, the Slab is a two Way Spanning Slab.

$C = 25$ mm, $h = 175$mm.

$b = 1000$mm, $n = 14.64$ kN/m²

SHORT SPAN

Continuous Edge	Mid-Span.
$\beta_{sx}^{-} = 0.031$	$\beta_{sx}^{+} = 0.024$
$M_{sx}^{-} = \beta_{sx}^{-} n l_x^2$	$M_{sx}^{+} = \beta_{sx}^{+} n l_x^2$
$= 0.031 \times 14.64 \times 6^2$	$= 0.024 \times 14.64 \times 6^2$
$= 16.34$ KN.m.	$= 12.05$ KN.m.
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$d = 175 - 25 - 12/2$	$d = 175 - 25 - 12/2$.
$= 144$mm.	$= 144$mm.
$K = \frac{16.34 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{12.65 \times 10^6}{25 \times 100 \times 144^2}$
$= 0.032$	$= 0.024$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.032}{1.134}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.024}{1.134}}$
$= 0.97$	$= 0.98$
Since $l_a = 0.97 > 0.95$,	Since $l_a = 0.98 > 0.95$,
Use $l_a = 0.95$	Use $l_a = 0.95$
$Z = l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$Z = 136.80$	$Z = 136.80$.
$A_{sreq} = \frac{16.34 \times 10^6}{0.87 \times 410 \times 136.8}$	$A_{sreq} = \frac{12.65 \times 10^6}{0.87 \times 410 \times 136.8}$
$= 334.86$ mm²/m	$= 259.24$ mm²/m.
Provide Y12mm @ 250 mm c/c	Provide Y12mm @ 250mm c/c
$A_{sprov} = 452$ mm²/m	$A_{sprov} = 452$ mm²/m.
$A_{smin} = 0.0013 \times 1000 \times 144$	$A_{smin} = 0.0013 \times 1000 \times 144$
$= 187.2$ mm²/m	$= 187.2$ mm²/m

Provision is O.K!!

Reference	Calculations	Output

Long Span

Continuous Edge

$\beta_{sy}^{-} = 0.032$

$M_{sy} = \beta_{sy} n l_x^2$

$= 0.032 \times 14.04 \times 6^2$

$= 16.61\ kN\,m$

$d = h - c - \phi - \phi/2$

$= 175 - 25 - 12 - 6$

$= 132mm$

$K = \dfrac{16.64 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.039$

$l_a = 0.5 + \sqrt{0.25 - \dfrac{0.039}{1.134}}$

$= 0.96$

Since $l_a = 0.96 > 0.95$,

Use $l_a = 0.95$

$z = 0.95 \times 132 = 125.4$

$A_{sreq} = \dfrac{16.64 \times 10^6}{0.87 \times 410 \times 125.4}$

$=$ [illegible] mm^2/m

Provide Y12mm @ 250mm c/c

$A_{sprov} = 452 mm^2/m$

$A_{smin} = 0.0013 \times 1000 \times 132$

$= 171.6\ mm^2/m$

Since $A_{sprov} > A_{smin}$

Mid-Span

$\beta_{sy}^{+} = 0.024$

$M_{sy} = \beta_{sy}^{+} n l_x^2$

$= 0.024 \times 14.04 \times 6^2$

$= 12.65\ kN\,m$

$d = h - c - \phi - \phi/2$

$= 175 - 25 - 12 - 6$

$= 132mm$

$K = \dfrac{12.65 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.029$

$l_a = 0.5 + \sqrt{0.25 - \dfrac{0.029}{1.134}}$

$= 0.97$

Since $l_a = 0.97 > 0.95$,

Use $l_a = 0.95$

$z = 0.95 \times 132 = 125.4$.

$A_{sreq} = \dfrac{12.65 \times 10^6}{0.87 \times 410 \times 125.4}$

$= 282.81 mm^2/m$.

Provide Y12mm @ 250mm c/c

$A_{sprov} = 452 mm^2/m$.

$A_{smin} = 0.0013 \times 1000 \times 132$

$= 171.6 mm^2/m$.

Since $A_{sprov} > A_{smin}$

Provision is O.K.

Reference	Calculations	Output

Deflection Check

The check for deflection at the Short Span -

Mid span -

[illegible] Stress, $f_s = \frac{5}{8} \times 410 \times \frac{259.24}{452} = 146.97\ N/mm^2$

Modification factor, $M.f = \frac{310}{f_s} = \frac{310}{146.97} = 2.1$

[illegible] $Mf > 2.0$, Adopt $Mf = 2.0$

[illegible] to get the basic Span Ratio

[illegible] $= \frac{259.24 \times 1000}{1000 \times 144} = 0.18$

0.5	—	28
0.18	—	x
0.15	—	38

$$\frac{0.18 - 0.5}{0.15 - 0.5} = \frac{x - 28}{38 - 28}$$

$x = 37.14$

[illegible] Span ratio = 37.14

Limiting Span = 2.0 × 37.14 = 74.28

Actual Span = $\frac{l_x}{d} = \frac{6000}{144} = 41.67$

∴ Actual Span is less than Limiting

[illegible] deflection is O.K.

Output: Deflection is Satisfactory.

REFERENCE	CALCULATION	OUTPUT
TABLE 7		

Aspect ratio

$l_y/l_x = \frac{7183}{6000} = 1.2$

$l_y/l_x < 2$, the slab is a 2-way spanning slab.

$C = 25mm$, $h = 175mm$, $b = 1000mm$, $n = 14.64 kN/m^2$.

SHORT SPAN

Continuous Edge	Mid span
$\beta_{sx}^- = 0.042$	$\beta_{sx}^+ = 0.032$
$M_{sx}^- = \beta_{sx} n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.042 \times 14.64 \times 6^2$	$= 0.032 \times 14.64 \times 6^2$
$= 22.13 kNm$	$= 16.86 kNm$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 6$	$= 175 - 25 - 6$
$= 144 mm$	$= 144mm$
$K = \frac{M}{f_{ck} b d^2} = \frac{22.13 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{16.86 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.04$	$= 0.03$
$Z = l_a d$	$Z = l_a d$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.5 + \sqrt{0.25 - \frac{0.04}{1.134}}$	$= 0.5 + \sqrt{0.25 - \frac{0.03}{1.134}}$
$= 0.96$	$= 0.97$
Since $0.96 > 0.95$, we use $l_a = 0.95$	Since $l_a > 0.95$, we use $l_a = 0.95$
$Z = l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8mm$	$= 136.8mm$
$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$	$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{22.13 \times 10^6}{0.87 \times 410 \times 136.8}$	$= \frac{16.86 \times 10^6}{0.87 \times 410 \times 136.8}$
$= 453.5 mm^2/m$	$= 345.5 mm^2/m$
$A_{s\,min} = 0.13\% bd$	$A_{s\,min} = 0.13\% bd$
$= 0.0013 \times 1000 \times 144$	$= 0.0013 \times 1000 \times 144$
[illegible] mm^2/m	$= 187.2 mm^2/m$
Provide Y12mm @ 200mm c/c	Provide Y12mm @ 300mm c/c
$A_{s\,prov} = 566 mm^2/m$	$A_{s\,prov} = 377 mm^2/m$
Since $A_{s\,prov} > A_{s\,min}$	Since $A_{s\,prov} > A_{s\,min}$

PROVISION IS O.K.

Ecode 7-(2)

REFERENCE	CALCULATION	OUTPUT

SHORT SPAN

	Mid - Span
	$AS_{min} < AS_{req}$
	Provide for AS_{req}
	Provide Y12 @ 250mm c/c
	$A_{s\,prov}$: 452 mm²/m

PROVISION IS OK

LONG SPAN.

	Mid - Span
	$\beta_{sy}^{+} = 0.024$
	$M = \beta_{sy}^{+} n l_x^2$
	$= 0.024 \times 14.64 \times 6^2$
	$= 12.65$ kN.m
	$d = h - c - \phi - \phi/2$
	$= 175 - 25 - 12 - 6$
	$= 132$mm
	$K = \dfrac{M}{f_{cu} bd^2} = \dfrac{12.65 \times 10^6}{25 \times 1000 \times 132^2}$
	$= 0.03$
	$z = d[0.5 + \sqrt{0.25 - K/0.9}]$
	$= 0.97$
	$0.97 > 0.95$ use $= 0.95$
	$z = 0.95d = 0.95 \times 132$
	$= 125.4$
	$A_{s\,req} = \dfrac{M}{0.87 f_y z}$
	$= \dfrac{12.65 \times 10^6}{0.87 \times 410 \times 125.4}$
	$= 282.8$ mm²/m
	$A_{s\,min} = 0.13\% \, bd$
	$= 0.0013 \times 1000 \times 132$
	$= 171.6$ mm²/m
	$AS_{min} < AS_{req}$
	Provide Using AS_{req}

	CALCULATION	OUTPUT
	[illegible] Code	7-(3)

[illegible]ECTION CHECK.

[illegible] for Short Span Mid-span.

$$[illegible] \; f_y \frac{A_{s\,req}}{A_{s\,prov}} \quad \frac{5 \times 410 \times 345.5}{8 \times 452}$$

$$= 195.9 \; N/mm^2$$

[illegible] $\frac{310}{[illegible]} = \frac{310}{195.9} = 1.58$

[illegible] Span [illegible] $\frac{6000}{144} = 41.67$

[illegible] $\frac{[illegible]}{[illegible]} \times 100\% = 0.24$

[illegible] (Interpolate To get basic span ratio)

$$\frac{x - 30}{33 - 30} = \frac{0.24 - 0.5}{0.15 - 0.5} \; ; \; \frac{x - 30}{3} = \frac{-0.26}{-0.35}$$

$$x - 30 = 2.23 \; ; \; x = 2.23 + 30$$
$$= 32.23$$

[illegible] × basic Span Ratio

$$= [illegible] \times 32.23$$
$$= 50.9$$

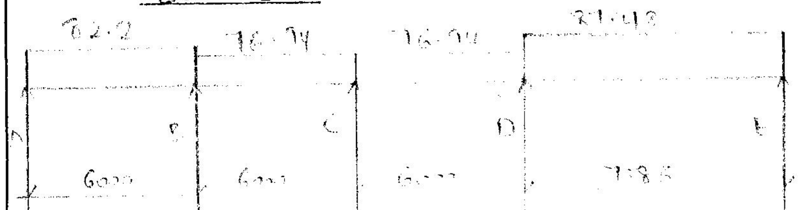
[illegible] Limiting Span Deflection is O.K

DEFLECTION IS O.K.

BEAM DESIGN TO EUROCODE.

REFERENCE	CALCULATIONS	OUTPUT

BEAM 3



Self-weight of beam = 0.225 × 0.4 × 24 × 1.35 = 2.92KN/m	Self-weight of beam = 2.92KN/m	Self weight of beam = 2.92KN/m	Self weight of beam = 2.92KN/m
Load from Panel 6 = 31.62KN/m	Load from Panel 7 = 28.99KN/m	Load from Panel 8 = 28.99KN/m	Load from Panel 9 = 34.26KN/m
Load from Panel 10 = 31.62KN/m	Load from Panel 11 = 28.99KN/m	Load from Panel 12 = 28.99KN/m	Load from Panel 13 = 34.26KN/m
Partition load = 3.47 × 3.425 × 1.35 = 16.04KN/m	Partition load = 16.04KN/m	Partition load = 16.04KN/m	Partition load = 16.04KN/m
Total load = 82.2KN/m	Total load = 76.94KN/m	Total load = 76.94KN/m	Total Load = 87.48KN/m

FIXED END MOMENT

SPAN A-B

$$FEM_{AB} = \frac{wl^2}{8} = \frac{82.2 \times 6^2}{8} = 369.9KNm$$

SPAN B-C

$$FEM_{BC} = \frac{wl^2}{12} = \frac{76.94 \times 6^2}{12} = 230.82KNm$$

SPAN C-D

$$FEM_{CD} = \frac{wl^2}{12} = \frac{76.94 \times 6^2}{12} = 230.82KNm$$

SPAN D-E

$$FEM_{DE} = \frac{wl^2}{8} = \frac{87.48 \times 7.183^2}{8} = 564.20KNm$$

STIFFNESS, K

$$K_{AB} = \frac{0.75}{L} = \frac{0.75}{6} = 0.125$$

$$K_{BC} = \frac{1}{L} = \frac{1}{6} = 0.167$$

$$K_{CD} = \frac{1}{L} = \frac{1}{6} = 0.167$$

REFERENCE	CALCULATIONS	OUTPUT

DISTRIBUTION FACTOR

$$D.F_{BA} = \frac{K_{AB}}{K_{AB} + K_{BC}} = \frac{0.125}{0.125 + 0.167} = 0.43$$

$$D.F_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = \frac{0.167}{0.167 + 0.125} = 0.57$$

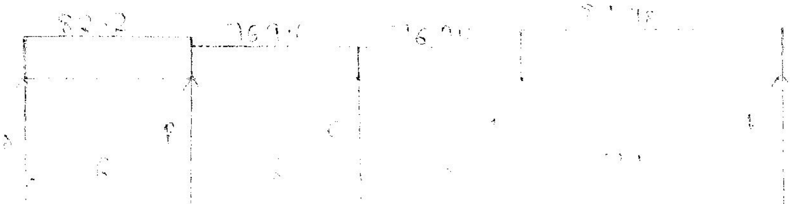
$$D.F_{CB} = \frac{K_{CB}}{K_{CB} + K_{CD}} = \frac{0.167}{0.167 + 0.167} = 0.5$$

$$D.F_{CD} = \frac{K_{CD}}{K_{CD} + K_{CB}} = \frac{0.167}{0.167 + 0.167} = 0.5$$

$$D.F_{DC} = \frac{K_{DC}}{K_{DC} + K_{DE}} = \frac{0.167}{0.167 + 0.104} = 0.62$$

$$D.F_{DE} = \frac{K_{DE}}{K_{DE} + K_{DC}} = \frac{0.104}{0.104 + 0.167} = 0.38$$

MOMENT DISTRIBUTION



0	0.43	0.57	0.5	0.5	0.62	0.38	0
	−	+	−	+	−	+	
	369.9	230.82	230.82	230.82	230.82	564.20	
	+59.80	+79.28	0	0	−206.7	−126.68	
		0	+39.64	−103.35	0		
		+15.73	+31.86	+31.86	+15.93		
	−6.85	−9.08			−9.88	6.05	
0	−316.95	+316.95	−159.32	+159.32	−431.47	+431.47	0

SHEAR FORCE

$$\text{REAL SHEAR FORCE}_{AB} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_A - M_B}{L_{AB}}\right)$$

$$= \frac{82.2 \times 6}{2} + \left(\frac{0 - 316.95}{6}\right)$$

$$= 193.77\ KN$$

$$\text{REAL SHEAR FORCE}_{BA} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_B - M_A}{L_{BA}}\right)$$

$$= \frac{82.2 \times 6}{2} + \frac{316.95 - 0}{6}$$

$$= 299.43\ KN$$

$$\text{REAL SHEAR FORCE}_{BC} = \frac{W_{BC}L_{BC}}{2} + \left(\frac{M_B - M_C}{L_{BC}}\right)$$

$$= \frac{76.94 \times 6}{2} + \left(\frac{316.95 - 159.32}{6}\right)$$

$$= 257.09\ KN$$

REAL SHEAR FORCE

REFERENCE	CALCULATIONS	OUTPUT

$$\text{REAL SHEAR FORCE}_{CD} = \frac{W_{CD}L_{CD}}{2} + \left(\frac{M_C - M_D}{L_{CD}}\right)$$

$$= \frac{76.94 \times 6}{2} + \left(\frac{159.32 - 431.47}{6}\right)$$

$$= 185.46\,kN$$

$$\text{REAL SHEAR FORCE}_{DC} = \frac{W_{DC}L_{DC}}{2} + \left(\frac{M_D - M_C}{L_{DC}}\right)$$

$$= \frac{76.94 \times 6}{2} + \left(\frac{431.47 - 159.32}{6}\right)$$

$$= 276.18\,kN$$

$$\text{REAL SHEAR FORCE}_{DE} = \frac{W_{DE}L_{DE}}{2} + \left(\frac{M_D - M_E}{L_{DE}}\right)$$

$$= \frac{87.48 \times 7.183}{2} + \left(\frac{431.47 - 0}{7.183}\right)$$

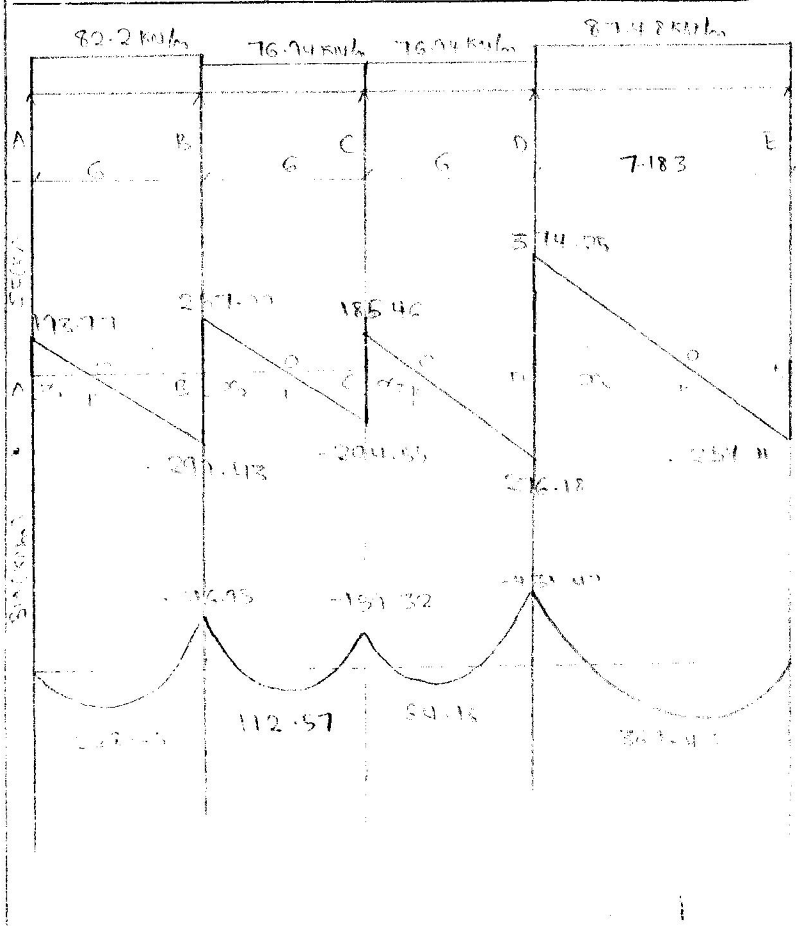
$$= 374.25\,kN$$

$$\text{REAL SHEAR FORCE}_{ED} = \frac{W_{ED}L_{ED}}{2} + \frac{M_E - M_D}{L_{ED}}$$

$$= \frac{87.48 \times 7.183}{2} \quad \frac{0 - 431.47}{7.183}$$

$$= 254.11\,kN$$

SHEAR FORCE AND BENDING MOMENT DIAGRAM.



REFERENCE	CALCULATIONS	OUTPUT

Span moment A-B = Area of triangle AOQ

$= \frac{1}{2} \times x_1 \times 193.77$

x_1; $193.77 - 82.2x_1 = 0$

$x_1 = 2.36m$

Span moment A-B $= \frac{1}{2} \times 2.36 \times 193.77$

$= 228.65 kNm$

Span moment B-C = Area of triangle BOQ − support moment B

$= \frac{1}{2} \times x_2 \times 257.09 - 316.95$

x_2; $257.09 - 76.94 x_2 = 0$

$x_2 = 3.34m$

Span moment B-C $= \frac{1}{2} \times 3.34 \times 257.09 - 316.95$

$= 112.57 kNm$

Span moment C-D = Area of triangle COQ − support moment C

$= \frac{1}{2} \times x_3 \times 185.46 - 159.32$

x_3; $185.46 - 76.94 x_3 = 0$

$x_3 = 2.41m$

Span moment C-D $= \frac{1}{2} \times 2.41 \times 185.46 - 159.32$

$= 64.16 kNm$

Span moment D-E = Area of triangle DOQ − Support moment D

$= \frac{1}{2} \times x_4 \times 374.25 - 431.47$

x_4; $374.25 - 87.48 x_4 = 0$

$x_4 = 4.28m$

Span moment D-E $= \frac{1}{2} \times 4.28 \times 374.25 - 431.47$

$= 369.43 kNm$

DESIGN OF SUPPORT REINFORCEMENT

M_{max} @ support $= 431.47 kNm$

$d = h - c - \phi_{link} - \phi/2$

$= 700 - 30 - 32/2 - 10$

$= 644mm$

$$K = \frac{M}{f_{ck} b d^2} = \frac{431.47 \times 10^6}{25 \times 300 \times 644^2}$$

$= 0.139$

Since $K < 0.167$, compression reinforcement is not required.

$$l_a = 0.5 + \sqrt{0.25 - K/1.134}$$

$$= 0.5 + \sqrt{0.25 - \frac{0.139}{1.134}}$$

$= 0.85$

REFERENCE	CALCULATIONS	OUTPUT
	$Z = l_a d = 0.85 \times 644$	
	$= 547.4mm$	
	$As_{req} = \frac{M}{0.87 f_{yk} Z} = \frac{431.47 \times 10^6}{0.87 \times 410 \times 547.4}$	
	$= 2209.75 mm^2$	
	Provide 5-Y25mm	
	$As_{prov} = 2450mm^2$	
	$b_f = b_w + 0.17l$	
	$= 300 + 0.17(7183) = 1521.11mm$	
	$b_w/b_f = \frac{300}{1521.11} = 0.2$	
	Since flange is in tension and the beam is a 'T' beam	
	$As_{min} = 0.13\% bh$	
	$= \frac{0.13}{100} \times 300 \times 700 = 273mm^2$	
	DESIGN OF SPAN REINFORCEMENT	
Table 8.9	$M_{max} = 369.43 KNm$	
	$K = \frac{M}{b_f f_{ck} d^2} = \frac{369.43 \times 10^6}{1521 \times 644^2 \times 25}$	
	$= 0.023$	
	$l_a = 0.5 + \sqrt{0.25 - \frac{0.023}{1.134}}$	
	$= 0.96$	
	Since $l_a > 0.95$, we use $l_a = 0.96$	
	$Z = l_a d = 0.95 \times 644$	
	$= 611.8 mm$	
	$As_{req} = \frac{369.43 \times 10^6}{0.87 \times 410 \times 611.8}$	
	$= 1692.85mm^2$	
	Provide 6Y20mm, $As_{prov} = 1890mm^2$	

REFERENCE	CALCULATIONS	OUTPUT

DESIGN OF SHEAR REINFORCEMENT

$V_{max} = 374.25\,kN$

$V_{Rd} = 0.3\,v f_{ck} b_w d$; $v = 0.675$

$= 0.3 \times 0.675 \times 25 \times 300 \times 644$

$= 978.08\,kN$

$V_{max} < V_{Rd}$ (section is ok to cater for shear).

$$A_{sw} = \frac{1.28\,s\,(V_{sd} - V_{Rd1})}{f_{yk}\,d}$$

$V_{sd} = V_{max}$

$$\frac{100 A_s}{bd} = \frac{100 \times 1890}{300 \times 644}$$

$= 0.98$

Table 5.16

$v_{Rd1} = 0.48$

$V_{Rd_1} = v_{Rd1} b_w d$

$= 0.48 \times 300 \times 644$

$= 92736$

$$A_{sw} = \frac{1.28\,s\,(V_{sd} - V_{Rd1})}{f_{yk}\,d}$$

$$157 = \frac{1.28\,s\,(374250 - 92736)}{250 \times 644}$$

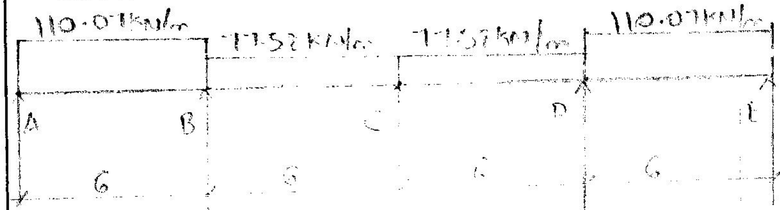
$S = 70.15\,mm$

Provide 2 legs of R10mm @ 150mm c/c.

REFERENCE	CALCULATIONS	OUTPUT
	DEFLECTION CHECK	
	Service stress, $\sigma_s = \frac{5}{8} \times f_{yk} \times \frac{A_{s\,req}}{A_{s\,prov}}$	
	$= \frac{5}{8} \times 410 \times \frac{1692.85}{1890}$	
	$= 229.52\ N/mm^2$	
	Modification factor $= \frac{310}{\sigma_s} = \frac{310}{229.52}$	
	$= 1.35$	
Table 5.0	$\frac{A_s}{bd} = \frac{1890}{1521.11 \times 644}$	
	$= 0.002$	
	$= 0.2\%$	
	To get basic span ratio	
	0.15 — 38	
	0.2 — x	
	0.5 — 28	
	$x = 36.57$	
	Limiting span = M.F × basic span ratio	
	$1.35 \times 36.57 = 49.37$	
	Actual span-depth ratio $= \frac{l_x}{d} = \frac{7183}{644} = 11.15$	
	Actual span-depth ratio < limiting span-depth ratio,	DEFLECTION IS
	Deflection is O.K.	SATISFACTORY

REFERENCE	CALCULATIONS	OUTPUT

BEAM 6



S/w of beam = 0.225 × 0.4 × 24 × 1.35 = 2.92KN/m	S/w of beam = 2.92KN/m	S/w of beam = 2.92KN/m	S/w of beam = 2.92KN/m
Load from Panel 1 = 49.92KN/m	Load from Panel 5 = 31.62KN/m	Load from Panel 5 = 31.62KN/m	Load from Panel 1 = 49.92KN/m
Load from Panel 2 = 43.18KN/m	Load from Panel 6 = 28.99KN/m	Load from Panel 6 = 28.99KN/m	Load from Panel 2 = 43.18KN/m
Partition load = 3.47 × 3 × 1.35 = 14.05KN/m	Partition load = 14.05KN/m	Partition load = 14.05KN/m	Partition load = 14.05KN/m
Total load = 110.07KN/m	Total load = 77.58KN/m	Total load = 77.58KN/m	Total load = 110.07KN/m

FIXED END MOMENT

$$FEM_{AB} = \frac{wl^2}{8} = \frac{110.07 \times 6^2}{8} = 498.15KNm$$

$$FEM_{BC} = \frac{wl^2}{12} = \frac{77.58 \times 6^2}{12} = 232.74KNm$$

$FEM_{BC} = FEM_{CD} = 232.74KNm$

$FEM_{DE} = FEM_{AB} = 498.15KNm$

STIFFNESS (K):

$$K_{AB} = \frac{0.75}{L_{AB}} = \frac{0.75}{6} = 0.125$$

$$K_{BC} = \frac{1}{L_{BC}} = \frac{1}{6} = 0.167$$

$K_{CD} = K_{BC} = 0.167$

$K_{DE} = K_{AB} = 0.125$

DISTRIBUTION FACTOR (Df):

$$D.F_{BA} = \frac{K_{AB}}{K_{AB} + K_{BC}} = \frac{0.125}{0.125 + 0.167} = 0.43$$

REFERENCE	CALCULATIONS	OUTPUT

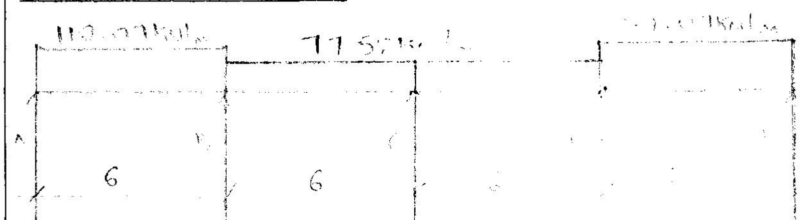
$$D.F_{CB} = \frac{K_{CB}}{K_{CB} + K_{CD}} = \frac{0.167}{0.167 + 0.167} = 0.5$$

$$D.F_{CD} = D.F_{CB} = 0.5$$

$$D.F_{DC} = \frac{K_{CD}}{K_{CD} + K_{DE}} = \frac{0.167}{0.167 + 0.125} = 0.57$$

$$D.F_{DE} = \frac{K_{DE}}{K_{CD} + K_{DE}} = \frac{0.125}{0.125 + 0.167} = 0.43$$

MOMENT DISTRIBUTION



0	0.43	0.57	0.5	0.5	0.57	0.43	0
	+	−	+	−	+	−	
	498.15	232.74	232.74	232.74	232.74	498.15	
	−114.13	−151.28	0	0	+151.28	+114.13	
		0	−75.64	+75.64	0		
	+384.02	−384.02	+157.1	−157.1	+384.02	−384.02	

SHEAR FORCE

$$\text{Real Shear Force}_{AB} = \frac{W_{AB} L_{AB}}{2} + \left(\frac{M_A - M_B}{L_{AB}}\right)$$

$$= \frac{110.07 \times 6}{2} + \left(\frac{0 - 384.02}{6}\right)$$

$$= 266.21 kN$$

$$\text{Real Shear Force}_{BA} = \frac{W_{BA} L_{BA}}{2} + \left(\frac{M_B - M_A}{L_{BA}}\right)$$

$$= 394.21 kN$$

$$\text{Real Shear Force}_{BC} = \frac{W_{BC} L_{BC}}{2} + \left(\frac{M_B - M_C}{L_{BC}}\right)$$

$$= \frac{77.58 \times 6}{2} + \left(\frac{384.02 - 157.1}{6}\right)$$

$$= 270.56 kN$$

$$\text{Real Shear Force}_{CB} = \frac{W_{CB} L_{CB}}{2} + \frac{M_C - M_B}{L_{CB}}$$

$$= \frac{77.58 \times 6}{2} + \left(\frac{157.1 - 384.02}{6}\right)$$

$$= 194.92 kN$$

REFERENCE	CALCULATIONS	OUTPUT

Real shear Force$_{CD}$ $= \dfrac{W_{CD} L_{CD}}{2} + \left(\dfrac{M_C - M_D}{L_{CD}}\right)$

$= \dfrac{77.58 \times 6}{2} + \left(\dfrac{157.1 - 384.02}{6}\right)$

$= 194.92 kN$

Real shear force$_{DC}$ $= \dfrac{W_{DC} L_{DC}}{2} + \left(\dfrac{M_D - M_C}{L_{DC}}\right)$

$= \dfrac{77.58 \times 6}{2} + \left(\dfrac{384.02 - 157.1}{6}\right)$

$= 270.56 kN$

Real shear force$_{DE}$ $= \dfrac{W_{DE} L_{DE}}{2} + \left(\dfrac{M_D - M_E}{L_{DE}}\right)$

$= \dfrac{110.07 \times 6}{2} + \left(\dfrac{384.02 - 0}{6}\right)$

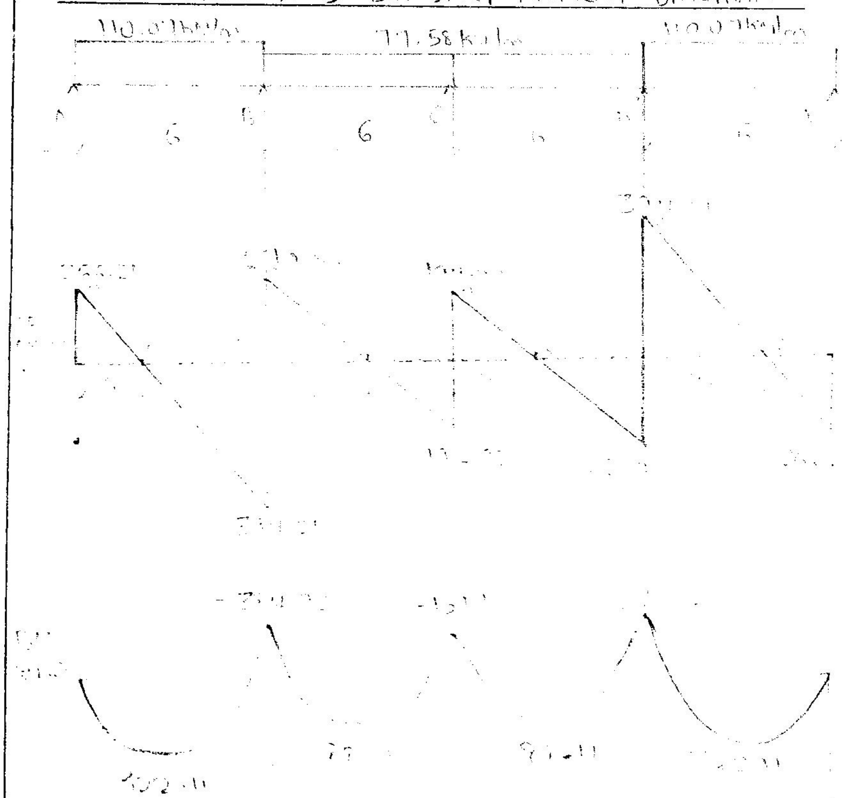
$= 394.21 kN$

Real shear force$_{ED}$ $= \dfrac{W_{ED} L_{ED}}{2} + \left(\dfrac{M_E - M_D}{L_{ED}}\right)$

$= \dfrac{110.07 \times 6}{2} + \dfrac{0 - 384.02}{6}$

$= 266.21 kN.$

SHEAR FORCE AND BENDING MOMENT DIAGRAM



REFERENCE	CALCULATIONS	OUTPUT

Span moment A-B = Area of triangle AOQ

$= \frac{1}{2} \times x_1 \times 266.21$

x_1; $266.21 - 110.07x_1 = 0$

$x_1 = 2.42m$

$\therefore$ Span moment A-B = $\frac{1}{2} \times 2.42 \times 266.21$

$= 322.11 KNm$

Span moment$_{B-C}$ = Area of triangle BOP − Support moment B

$= \frac{1}{2} \times x_2 \times 270.56 - 384.02$

x_2; $270.56 - 77.58x_2 = 0$

$x_2 = 3.49m$

Span moment$_{B-C}$ = $\frac{1}{2} \times 3.49 \times 270.56 - 384.02$

$= 88.11 KNm$

Span moment C-D = Area of triangle COP − support moment C

$= \frac{1}{2} \times x_3 \times 194.92 - 157.1$

x_3; $194.92 - 77.58x_3 = 0$

$x_3 = 2.51m$

$\therefore$ Span moment C-D = $\frac{1}{2} \times 2.51 \times 194.92 - 157.1$

$= 88.11 KNm$

Span moment D-E = Area of triangle DOP − support moment D

$= \frac{1}{2} \times x_4 \times 394.21 - 384.02$

x_4; $394.21 - 110.07x_4 = 0$

$x_4 = 3.58m$

$\therefore$ Span moment D-E = $\frac{1}{2} \times 3.58 \times 394.21 - 384.02$

$= 322.11 KNm$

DESIGN OF SUPPORT REINFORCEMENT

$M_{max} = 384.02 KNm$

$h = 800mm$

$d = h - c - \phi_{link} - \phi/2$

$= 800 - 30 - 32/2 - 10$

$= 744mm$

$k = \frac{M}{f_{ck} b d^2} = \frac{384.02 \times 10^6}{25 \times 225 \times 744^2} = 0.123$

$l_a = 0.5 + \sqrt{0.25 - k/1.134}$

$= 0.88$

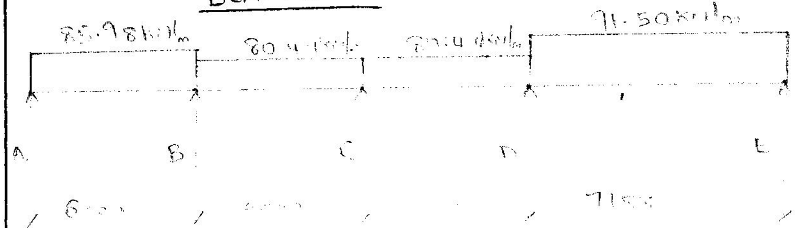
REFERENCE	CALCULATIONS	OUTPUT
	$As_{req} = \frac{M}{0.87f_{yk}Z} = \frac{384.02 \times 10^6}{0.87 \times 410 \times 654.72}$	
	$= 1644.35mm^2$.	
	Provide 4Y25mm Top	
	$As_{prov} = 1960mm^2$	
	DESIGN OF SPAN REINFORCEMENT	
	M_{max} @ span $= 322.11KNm$	
Table 8.9	$b_f = b_w + 0.17l$ (end span)	
	$= 225 + 0.17 \times 6000$	
	$= 1245mm$	
	$K = \frac{M}{f_{ck}bd^2} = \frac{322.11 \times 10^6}{25 \times 1245 \times 744^2}$	
	$= 0.02$	
	$l_a = 0.5 + \sqrt{0.25 - 0.02/1.134}$	
	$= 0.98$	
	since $l_a > 0.98$, we use $l_a = 0.95$	
	$\therefore Z = l_a d$	
	$= 706.8mm$	
	$As_{req} = \frac{M}{0.87f_{yk}Z} = \frac{322.11 \times 10^6}{0.87 \times 410 \times 706.8}$	
	$= 1277.63mm^2$	
	Provide 5-Y20mm Top, $As_{prov} = 1570mm^2$	
	$As_{min} = 0.13\%bh$	
	$= 0.0013 \times 300 \times 800$	
	$= 312mm^2$	
	DESIGN OF SHEAR REINFORCEMENT	
	$V_{max} = 394.21KN$	
	$V_{Rd,2} = 0.3 \upsilon f_{ck} b_w d$, $\upsilon = 0.575$	
	$= 0.3 \times 0.575 \times 25 \times 225 \times 744$	
	$= 721912.5N$	
	$V_{max} < V_{Rd,2}$ (section is O.K to cater for shear)	
	$A_{sw} = \frac{1.28s(V_{sd} - V_{rd,1})}{f_{ywk}d}$ $V_{sd} = V_{max}$	
	$V_{Rd1} = ?$ $\frac{100As}{bd} = \frac{100 \times 1960}{225 \times 744} = 1.17$	
	$V_{Rd1} = 0.48$	
	$V_{Rd1} = V_{Rd1} b_w d = 0.48 \times 225 \times 744$	

REFERENCE	CALCULATIONS	OUTPUT
	$157 = \frac{1.28s\,(394.210 - 80352)}{250 \times 744}$	
	$s = 72.689$	
	Provide 2 legs R10mm @ 100mm c/c	
	DEFLECTION CHECK	
	Service stress; $\sigma_s = \frac{5}{8} \times f_y \times \frac{A_{s\,req}}{A_{s\,prov}}$	
	$= \frac{5}{8} \times 410 \times \frac{1277.63}{1570}$	
	$= 208.53\ N/mm^2$	
	Modification factor $= \frac{310}{\sigma_s} = \frac{310}{208.53}$	
	$= 1.49$	
Table 5.0	$\frac{A_s}{bd} = \frac{1570}{1245 \times 744} = 0.0017 = 0.17\%$	
	To get basic span-depth ratio	
	0.15 —— 38	
	0.17 —— x	
	0.5 —— 28	
	$x = 37.43$	
	Limiting span-depth ratio $= 1.49 \times 37.43$	
	$= 55.77$	
	Actual span $= l_x/d = \frac{6000}{744}$	
	$= 8.06$	
	Actual < limiting; deflecting is OK	DEFLECTION IS O.K

BEAM DESIGN TO BS8110.

REFERENCE	CALCULATIONS	OUTPUT

BEAM 3



self-weight of beam $=0.225\times0.4\times24\times1.4$ $=3.024kN/m$	self-weight of beam $=0.225\times0.4\times24\times1.4$ $=3.024kN/m$	self-weight of beam $=0.225\times0.4\times24\times1.4$ $=3.024kN/m$	self-weight of beam $=0.225\times0.4\times24\times1.4$ $=3.024kN/m$
Load from panel 6 $=33.18kN/m$	Load from Panel 7 $=30.41kN/m$	Load from Panel 8 $=30.41kN/m$	Load from Panel 9 $=35.94kN/m$
Load from Panel 10 $=33.18kN/m$	Load from Panel 11 $=30.41kN/m$	Load from Panel 12 $=30.41kN/m$	Load from panel 13 $=35.94kN/m$
Partition Load $=3.47\times3.425\times1.4$ $=16.6kN/m$	Partition Load $=16.6kN/m$	Partition Load $=16.6kN/m$	Partition Load $=16.6kN/m$
Total Load $=85.98kN/m$	Total Load $=80.44kN/m$	Total Load $=80.44kN/m$	Total Load $=91.50kN/m$

FIXED END MOMENT

SPAN A-B

$$FEM_{AB} = \frac{wl^2}{8} = \frac{85.98\times6^2}{8} = 386.76\,kNm$$

SPAN B-C

$$FEM_{BC} = \frac{wl^2}{12} = \frac{80.44\times6^2}{12} = 241.32\,kNm$$

SPAN C-D

$$FEM_{CD} = \frac{wl^2}{12} = \frac{80.44\times6^2}{12} = 241.32\,kNm$$

SPAN D-E

$$FEM_{DE} = \frac{wl^2}{8} = \frac{91.5\times7.183^2}{8} = 590.12\,kNm$$

STIFFNESS, K

$$K_{AB} = \frac{0.75}{L} = \frac{0.75}{6} = 0.125$$

$$K_{BC} = \frac{1}{L} = \frac{1}{6} = 0.167$$

$$K_{CD} = \frac{1}{L} = \frac{1}{6} = 0.167$$

$$K_{DE} = 0.75 = 0.75$$

REFERENCE	CALCULATIONS	OUTPUT

DISTRIBUTION FACTOR

$$D.F_{BA} = \frac{K_{AB}}{K_{AB} + K_{BC}} = \frac{0.125}{0.125 + 0.167} = 0.43$$

$$D.F_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = \frac{0.167}{0.167 + 0.125} = 0.57$$

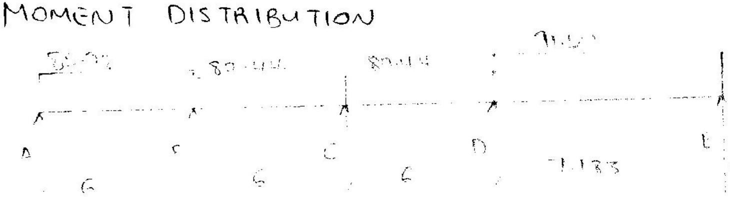
$$D.F_{CB} = \frac{K_{BC}}{K_{BC} + K_{CD}} = \frac{0.167}{0.167 + 0.167} = 0.5$$

$$D.F_{CD} = \frac{K_{CD}}{K_{CD} + K_{BC}} = \frac{0.167}{0.167 + 0.167} = 0.5$$

$$D.F_{DC} = \frac{K_{DC}}{K_{DC} + K_{DE}} = \frac{0.167}{0.167 + 0.104} = 0.62$$

$$D.F_{DE} = \frac{K_{DE}}{K_{DE} + K_{DC}} = \frac{0.104}{0.167 + 0.104} = 0.38$$

MOMENT DISTRIBUTION



0   0.43 0.57   0.5 0.5   0.62 0.38   0

− +   − +   − +

386.76 241.32 241.32 241.32 241.32 590.12

62.54 82.90 0 0 246.26 132.54

0 41.45 108.13 0

33.34 33.64

16.67 16.67

7.17 9.50 10.34 6.33

0   −331.39 +331.39 166.5 166.5 451.8 449.25   0

SHEAR FORCE

$$\text{REAL SHEAR FORCE}_{AB} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_A - M_B}{L_{AB}}\right)$$

$$= \frac{85.98 \times 6}{2} + \frac{0 - 331.39}{6}$$

$$= 202.71 \text{ KN}$$

$$\text{REAL SHEAR FORCE}_{BA} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_B - M_A}{L_{AB}}\right)$$

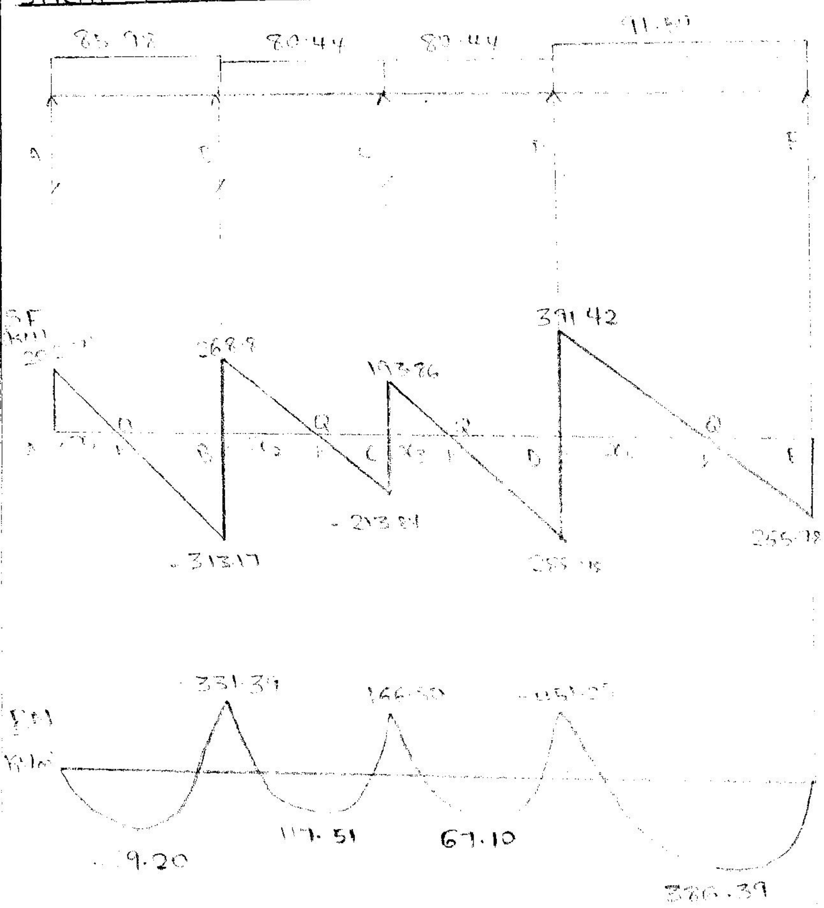
$$= \frac{85.98 \times 6}{2} + \left(\frac{331.39 - 0}{6}\right)$$

$$= 313.17 \text{KN}$$

REFERENCE	CALCULATIONS	OUTPUT
	$\text{REAL SHEAR FORCE}_{BC} = \frac{W_{BC}L_{BC}}{2} + \left(\frac{M_B - M_C}{L_{BC}}\right)$	
	$= \frac{80.44 \times 6}{2} + \frac{331.39 - 166.5}{6}$	
	$= 268.8\,kN$	
	$\text{REAL SHEAR FORCE}_{CB} = \frac{W_{CB}L_{CB}}{2} + \left(\frac{M_C - M_B}{L}\right)$	
	$= \frac{80.44 \times 6}{2} + \frac{166.5 - 331.39}{6}$	
	$= 213.84\,kN$	
	$\text{REAL SHEAR FORCE}_{CD} = \frac{W_{CD}L_{CD}}{2} + \frac{(M_C - M_D)}{L_{CD}}$	
	$= \frac{80.44 \times 6}{2} + \frac{166.5 - 451.25}{6}$	
	$= 193.86\,kN$	
	$\text{REAL SHEAR FORCE}_{DC} = \frac{W_{DC}L_{DC}}{2} + \left(\frac{M_D - M_C}{L_{DC}}\right)$	
	$= \frac{80.44 \times 6}{2} + \frac{451.25 - 166.5}{6}$	
	$= 288.78\,kN$	
	$\text{REAL SHEAR FORCE}_{DE} = \frac{W_{DE}L_{DE}}{2} + \left(\frac{M_D - M_E}{L_{DE}}\right)$	
	$= \frac{91.5 \times 7.183}{2} + \frac{451.25 - 0}{7.183}$	
	$= 328.6 + 62.82$	
	$= 391.42\,kN$	
	$\text{REAL SHEAR FORCE}_{ED} = \frac{W_{ED}L_{ED}}{2} + \left(\frac{M_E - M_D}{L_{ED}}\right)$	
	$= 265.78\,kN$	

REFERENCE	CALCULATIONS	OUTPUT

SHEAR FORCE AND BENDING MOMENT DIAGRAM



Span moment A-B = Area of triangle AOQ

$= \frac{1}{2} \times x_1 \times 202.71$

x_1; $202.71 - 85.98(x_1) = 0$

$x_1 = 2.36m$

Moment @ span A-B = $\frac{1}{2} \times 2.36 \times 202.71$

$= 239.2$ kNm

Span moment B-C = Area of triangle BOQ - Support moment B

$= \frac{1}{2} \times x_2 \times 268.8 - 331.39$

x_2, $268.8 - 80.44 x_2 = 0$

$x_2 = 3.34m$

Moment @ span B-C = $\frac{1}{2} \times 3.34 \times 268.8 - 331.39$

$= 117.51$ kNm

Span moment C-D = Area of triangle COQ - support moment C

$= \frac{1}{2} \times x_3 \times 193.86 - 166.50$

x_3; $193.86 - 80.44 x_3 = 0$

$x_3 = 2.41m$

Moment at span B-C = $\frac{1}{2} \times 2.41 \times 193.86 - 166.50$

REFERENCE	CALCULATIONS	OUTPUT
	Span moment D-E = Area of triangle DOQ − Support moment D $= \frac{1}{2} \times x_4 \times 391.42 - 451.25$ x_4; $391.42 - 91.5x_4 = 0$ $x_4 = 4.28m$ Moment @ span D-E $= \frac{1}{2} \times 4.28 \times 391.42 - 451.25$ $= 386.39 KNm$ DESIGN OF SUPPORT REINFORCEMENT M_{max} @ support = 451.25KNm $d = h - c - \phi_{link} - \phi/_2$ $= 700 - 30 - 32/_2 - 10$ $= 644$ mm $K = \frac{M}{f_{cu}bd^2} = \frac{451.25 \times 10^6}{25 \times 300 \times 644^2}$ $= 0.145$ Since $K < 0.156$, Compression reinforcement is not required. Lever arm, $l_a = 0.5 + \sqrt{0.25 - K/_{0.9}}$ $= 0.5 + \sqrt{0.25 - 0.145/_{0.9}}$ $= 0.5 + 0.8$ $= 0.8$ $\therefore Z = l_a d = 0.80 \times 644$ $= 515.2$ mm $As_{req} = \frac{M}{0.95 f_y Z} = \frac{451.25 \times 10^6}{0.95 \times 410 \times 515.2}$ $= 2248.7 mm^2$ Provide 5 - Y25mm $As_{prov} = 2450 mm^2$. $b_f = b_{eff} + l_z/_5$ $b_f = 300 + \frac{0.85 \times 6000}{5} = 1020mm$ Since here flange is in tension and the beam is 'T' beam: $As_{min} = 0.26\% bh$ $= \frac{0.26}{100} \times 300 \times 700 = 546mm^2$. Since As_{req} is greater than As_{min}, Provision is O.K	PROVISION IS O.K

REFERENCE	CALCULATIONS	OUTPUT

DESIGN OF SPAN REINFORCEMENT:

$M_{max} = 386.39 kNm$

$$K = \frac{M}{b_f f_{cu} d^2} = \frac{386.39 \times 10^6}{1020 \times 644^2 \times 25}$$

$$= 0.03$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.036}{0.9}}$$

$$= 0.95$$

Since $l_a = 0.95$, we use $l_a = 0.95$

$$Z = l_a d = 0.95 \times 644$$

$$= 611.8 mm.$$

$$A_{s\,req} = \frac{388.39 \times 10^6}{0.95 \times 410 \times 611.8}$$

$$= 1621.47 mm^2.$$

Provide 6Y20mm, $A_{s\,prov} = 1890 mm^2$.

$$b_w / b_f = \frac{300}{1020} = 0.29$$

Since here web is in tension and $b_w/b < 0.4$,

$$A_{s\,min} = 0.18\% bh = \frac{0.18}{100} \times 300 \times 700$$

$$= 378 mm^2.$$

Since $A_{s\,req}$ is greater than $A_{s\,min}$; Provision is O.K. — PROVISION IS O.K.

DESIGN OF SHEAR REINFORCEMENT:

$V_{max} = 391.42 KN$

$$v = \frac{V_{max}}{bd} = \frac{391.42 \times 10^3}{300 \times 644}$$

$$= 2.03 N/mm^2.$$

$$\frac{100 A_s}{bd} = \frac{100 \times 2450}{300 \times 644} = 1.27$$

$$\frac{400}{d} = \frac{400}{644} = 0.62$$

$$V_c = \frac{0.79 (100 A_s/bd)^{1/3} (400/d)^{1/4}}{\lambda_m}$$

$$= \frac{0.79 \times (1.27)^{1/3} (0.62)^{1/4}}{1.25}$$

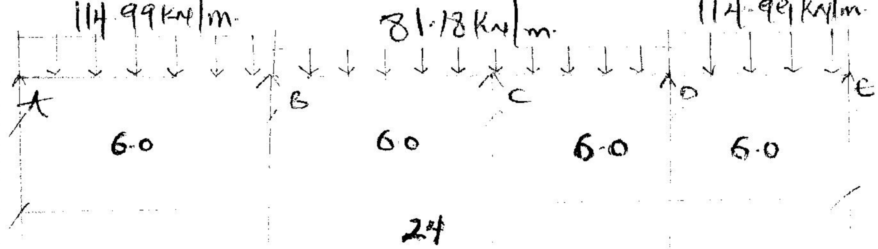
$$= 0.61 N/mm^2$$

$(V_c + 0.4) < 0.8\sqrt{f_{cu}}$ or $5 N/mm^2$ → Satisfies this condition, therefore, $S_v = \frac{A_{sv}\, 0.95 f_{yv}}{0.4 b_v}$

REFERENCE	CALCULATIONS	OUTPUT
	DEFLECTION CHECK	
	Service stress, $f_s = \frac{2}{3} \times f_y \times \frac{A_{s\,req}}{A_{s\,prov}}$	
	$= \frac{2}{3} \times 410 \times \frac{1621.47}{1890}$	
	$= 234.50\ N/mm^2$	
	$M.F = 0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}$	
	$\frac{M}{bd^2} = \frac{386.39 \times 10^6}{1020 \times 644^2}$	
	$= 0.91$	
	$M.F = 0.55 + \frac{477 - 234.50}{120(0.9 + 0.91)}$	
	$= 1.67$	
	Since $\frac{b_w}{b_f} = 0.29 < 0.3$, basic span ratio $= 20.8$	
	Limiting span-depth ratio $= 1.67 \times 20.8 = 34.74$	
	Actual span-depth $= \frac{l_x}{d} = \frac{7.183}{644} = 11.15$	
	Since actual span-depth ratio is less than limiting span-depth ratio, deflection is o.k.	DEFLECTION IS SATISFACTORY

Reference	Calculations	Output

Beam 6 (Six)



S/W of beam = 0.225×0.4×24×1.4 = 3.024 kN/m.	S/W of beam = 3.024 kN/m	S/W of beam = 3.024 kN/m	S/W of beam = 3.024 kN/m.
Load from Panel 1 = 52.2 kN/m.	Load from Panel 5 = 33.18 kN/m.	Load from Panel 5 = 33.18 kN/m	Load from Panel 1 = 52.2 kN/m.
Load from Panel 2 = 45.2 kN/m	Load from Panel 6 = 30.41 kN/m.	Load from Panel 6 = 30.41 kN/m.	Load from Panel 2 = 45.2 kN/m
Partition Load = 3.47×3.0×1.4 = 14.57 kN/m.	Partition Load = 3.47×3.0×1.4 = 14.57 kN/m	Partition Load = 3.47×3.0×1.4 = 14.57 kN/m.	Partition Load = 14.57 kN/m.
Total Load = 114.99 kN/m	Total Load = 81.18 kN/m.	Total Load = 81.18 kN/m	Total Load = 114.99 kN/m

Fixed End Moment:-

$$F.E.M_{A-B} = \frac{Wl^2}{8} = \frac{114.99 \times 6^2}{8} = 517.46 \text{ kN.m.}$$

$$F.E.M_{BC} = \frac{Wl^2}{12} = \frac{81.18 \times 6^2}{12} = 243.54 \text{ kN.m}$$

$$F.E.M_{BC} = F.E.M_{CD} = 243.54 \text{ kN.m.}$$

$$F.E.M_{DE} = \frac{Wl^2}{8} = \frac{114.99 \times 6^2}{8} = 517.46 \text{ kN.m.}$$

Stiffness (K):-

$$K_{AB} = \frac{0.75}{L_{AB}} = \frac{0.75}{6} = 0.125.$$

$$K_{BC} = \frac{1}{L_{BC}} = \frac{1}{6} = 0.167$$

$$K_{BC} = K_{CD} = 0.167.$$

Reference	Calculations	Output

Distribution Factor (Df):-

$$D.f_{BA} = \frac{K_{AB}}{K_{AB}+K_{BC}} = \frac{0.125}{0.125+0.167} = 0.43.$$

$$D.f_{BC} = \frac{K_{BC}}{K_{AB}+K_{BC}} = \frac{0.167}{0.125+0.167} = 0.57.$$

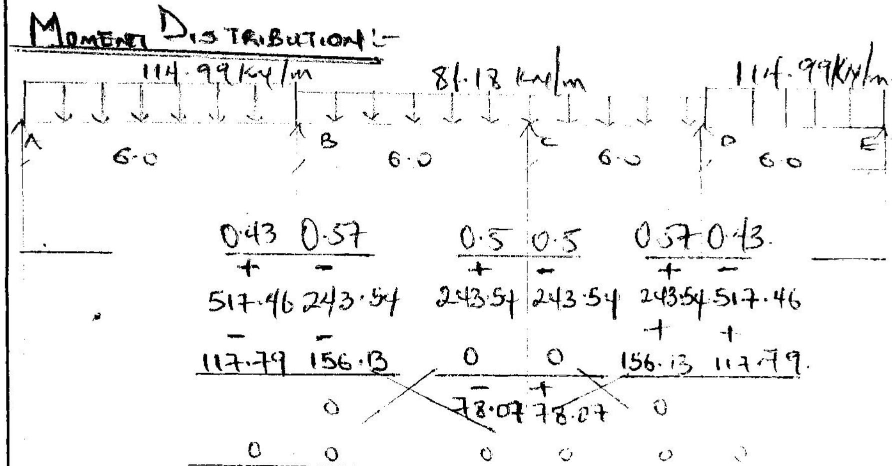
$$D.f_{CB} = \frac{K_{BC}}{K_{BC}+K_{CD}} = \frac{0.167}{0.167+0.167} = 0.50$$

$$D.f_{CB} = D.f_{CD} = 0.5$$

$$D.f_{DC} = \frac{K_{CD}}{K_{CD}+K_{DE}} = \frac{0.167}{0.167+0.125} = 0.57.$$

$$D.f_{DE} = \frac{K_{DE}}{K_{CD}+K_{DE}} = \frac{0.125}{0.167+0.125} = 0.43,$$

Moment Distribution:-



0.43	0.57	0.5	0.5	0.57	0.43
+	−	+	−	+	−
517.46	243.54	243.54	243.54	243.54	517.46
−	−			+	+
117.79	156.13	0	0	156.13	117.79
	0	−78.07	+78.07	0	
0	0	0	0	0	0
+399.67	−399.67	+165.47	−165.47	+399.67	−399.67

Shear Force

$$\text{Real Shear force}_{AB} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_A - M_B}{L_{AB}}\right)$$

$$= \frac{114.99\times 6}{2} + \left(\frac{0 - 399.67}{6.0}\right)$$

$$= 278.36 \text{ KN}.$$

Reference	Calculations	Output.

$$\text{Real Shear force}_{BA} = \frac{W_{AB}L_{AB}}{2} + \left(\frac{M_B - M_A}{L_{AB}}\right)$$

$$= \frac{114.99\times 6}{2} + \left(\frac{399.67-0}{6.0}\right)$$

$$= 411.58 \text{ kN.}$$

$$\text{Real Shear force}_{BC} = \frac{W_{BC}L_{BC}}{2} + \left(\frac{M_B - M_C}{L_{BC}}\right)$$

$$= \frac{81.18\times 6}{2} + \left(\frac{399.67-165.47}{6}\right)$$

$$= 282.57 \text{ kN.}$$

$$\text{Real Shear force}_{CB} = \frac{W_{BC}L_{BC}}{2} + \left(\frac{M_C - M_B}{L_{BC}}\right)$$

$$= \frac{81.18\times 6}{2} + \left(\frac{165.47-399.67}{6}\right)$$

$$= 204.51 \text{ kN.}$$

$$\text{Real Shear force}_{CD} = \frac{W_{CD}L_{CD}}{2} + \left(\frac{M_C - M_D}{L_{CD}}\right)$$

$$= \frac{81.18\times 6}{2} + \left(\frac{165.47-399.67}{6}\right)$$

$$= 204.51 \text{ kN.}$$

$$\text{Real Shear force}_{DC} = \frac{W_{CD}L_{CD}}{2} + \left(\frac{M_D - M_C}{L_{CD}}\right)$$

$$= \frac{81.18\times 6}{2} + \left(\frac{399.67-165.47}{6}\right)$$

$$= 282.57 \text{ kN.}$$

$$\text{Real Shear force}_{DE} = \frac{W_{DE}L_{DE}}{2} + \left(\frac{M_D - M_E}{L_{DE}}\right)$$

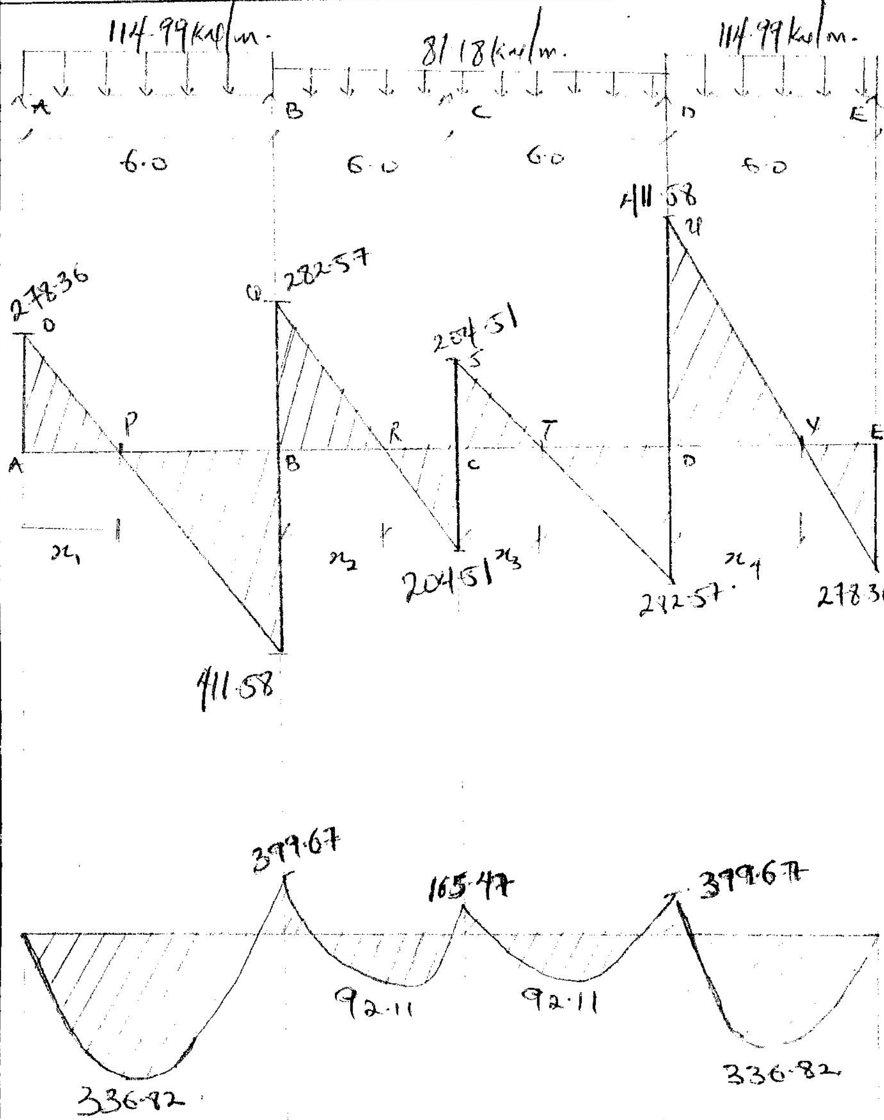
$$= \frac{114.99\times 6}{2} + \left(\frac{399.67-0}{6}\right)$$

$$= 411.58$$

$$\text{Real Shear force}_{ED} = \text{Real Shear force}_{AB} = 278.36 \text{kN.}$$

REFERENCE	CALCULATIONS	OUTPUT.

SHEAR FORCE & BENDING MOMENT DIAGRAM :-



$\text{Span Moment}_{A-B} = \text{Area of triangle } A\hat{O}P$

$= \frac{1}{2} \times x_1 \times 278.36$

$x_1;\ 278.36 - 114.99(x_1) = 0$

$\therefore x_1 = \dfrac{278.36}{114.99} = 2.42\text{m}.$

$\therefore \text{Span Moment}_{A-B} = \frac{1}{2} \times 2.42 \times 278.36 = 336.82\text{kN m}$

$\text{Span Moment}_{A-B} = 336.82 \text{ kN.m}.$

$\text{Span Moment}_{B-C} = \text{Area of triangle } B\hat{Q}R - \text{Support moment } B$

$= \left(\frac{1}{2} \times x_2 \times 282.57\right) - 399.67$

$x_2; = \quad 282.57 - 81.18(x_2) = 0.$

Reference	Calculations	Output

$$\text{Span Moment}_{B-C} = \left(\tfrac{1}{2} \times 3.48 \times 282.37\right) - 399.67$$

$$= 92.\cancel{44} \text{ kN·m.}$$

$$\text{Span Moment}_{C-D} = \text{Area of triangle } C\hat{S}\hat{T} - (\text{Support moment } C)$$

$$= \left(\tfrac{1}{2} \times 204.51 \times x_3\right) - 165.47$$

x_3; $204.51 - 81.18(x_3) = 0$

$$\therefore x_3 = \frac{204.51}{81.18} = 2.52$$

$$\text{Span Moment}_{CD} = \left(\tfrac{1}{2} \times 204.51 \times 2.52\right) - 165.47$$

$$= 92.11$$

$$\text{Span Moment}_{D-E} = \left(\tfrac{1}{2} \times x_4 \times 411.58\right) - 399.67$$

x_4; $411.58 - 114.99(x_4) = 0$

$$\therefore x_4 = \frac{411.58}{114.99} = 3.58$$

$$= \left(\tfrac{1}{2} \times 3.58 \times 411.58\right) - 399.67 = 336.82 \text{ kN·m.}$$

Design Support Reinforcement:

$M_{max} = 399.67$ kN·m $h = 800$mm.

$d = h - c - \phi/2 - \phi_{links} = 800 - 30 - 32/2 - 10 = 744$mm.

$$K = \frac{M}{f_{cu} b d^2} = \frac{399.67 \times 10^6}{25 \times 225 \times 744^2} = 0.128$$

$$l_a = 0.5 + \sqrt{0.25 - K/0.9} = 0.5 + \sqrt{0.25 - \frac{0.128}{0.9}}$$

$l_a = 0.83$.

$Z = l_a d = 0.83 \times 744 = 617.52$

$$A_{sreq} = \frac{M}{0.95 f_y Z} = \frac{399.67 \times 10^6}{0.95 \times 410 \times 617.52} = 1661.66 \text{mm}^2.$$

Provide 4 - Y25mm Top, $A_{sprov} = 1960 \text{mm}^2$

REFERENCE	CALCULATIONS	OUTPUT.

$b_f = b_{eff} + l_z/5$

$b_f = 225 + \frac{0.85 \times 6000}{5} = 1245\,mm.$

$b_w/b_f = \frac{225}{1245} = 0.18$

Since here flange is in tension, & the beam is "T"

$A_{s\,min} = 0.26\%\,bh = \frac{0.26}{100} \times 223 \times 800 = 468\,mm^2$

Since $A_{s\,req}$ is greater than $A_{s\,min}$,
Provision is O.K!!

DESIGN SPAN REINFORCEMENT:-

$M_{max} = 336.82\,kN.m.$

$K = \frac{M}{b_f f_{cu} d^2} = \frac{336.82 \times 10^6}{1245 \times 744^2 \times 25} = 0.02$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.02}{0.9}} = 0.97$

Since $l_a = 0.97$ is greater than 0.95, Use

$l_a = 0.95$

$l_a Z = l_a d = 0.95 \times 744 = 706.8$

$A_{s\,req} = \frac{336.82 \times 10^6}{0.95 \times 410 \times 706.8} = 1223.47\,mm^2.$

Provide 4-Y20 mm Top, $A_{s\,prov} = 1260\,mm^2$

Since here Web is in tension and $b_w/b < 0.4$,

$A_{s\,min} = 0.18\%\,b_w h = \frac{0.18}{100} \times 225 \times 800$

$= 324\,mm^2$

Since $A_{s\,req}$ is greater than $A_{s\,min}$, Provision is O.K!!

DESIGN SHEAR REINFORCEMENT:-

$V_{max} = 411.58\,kN$ $\quad v = \frac{V_{max}}{bd}$

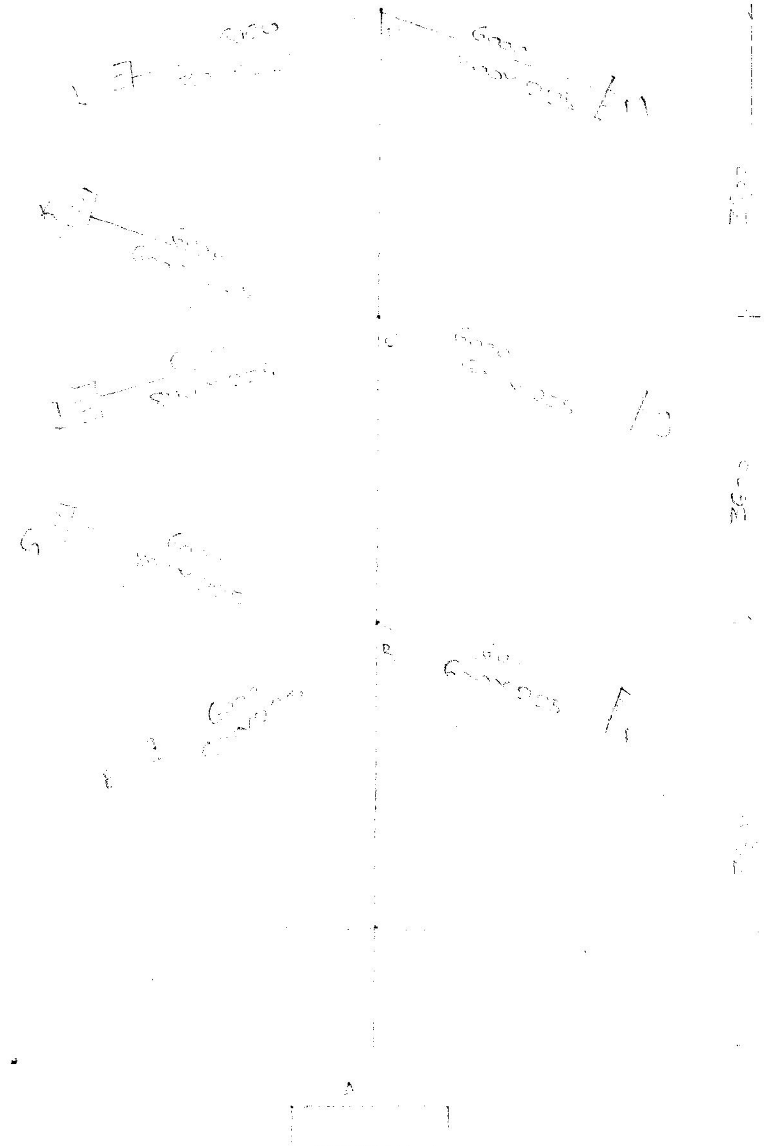
$v = \frac{411.58 \times 10^3}{\ldots} = 2.46\,N/mm^2.$

Reference	Calculations	Output.
	$\frac{100A_s}{bd} = \frac{100 \times 1960}{225 \times 744} = 1.17$	
	$\frac{400}{d} = \frac{400}{744} = 0.54$	
	$v_c = \frac{0.79\left(\frac{100A_s}{bd}\right)^{1/3}\left(400/d\right)^{1/4}}{\lambda_m} = 0.79 \times \frac{1.17^{1/3} \times 0.54^{1/4}}{1.25}$	
	$v_c = 0.571\ N/mm^2$,	
	$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or $5\ N/mm^2 \rightarrow$ Satisfies this condition, therefore $S_v = \frac{A_{sv}\,0.95 f_{yv}}{0.4 b_v}$	
	$S_v = \frac{157 \times 0.95 \times 250}{0.4 \times 225} = 414.31\,mm$	
	Provide 2 legs R10mm @ 400mm c/c	

COLUMN DESIGN TO BS8110
& EUROCODE.

REFERENCE	CALCULATIONS	OUTPUT

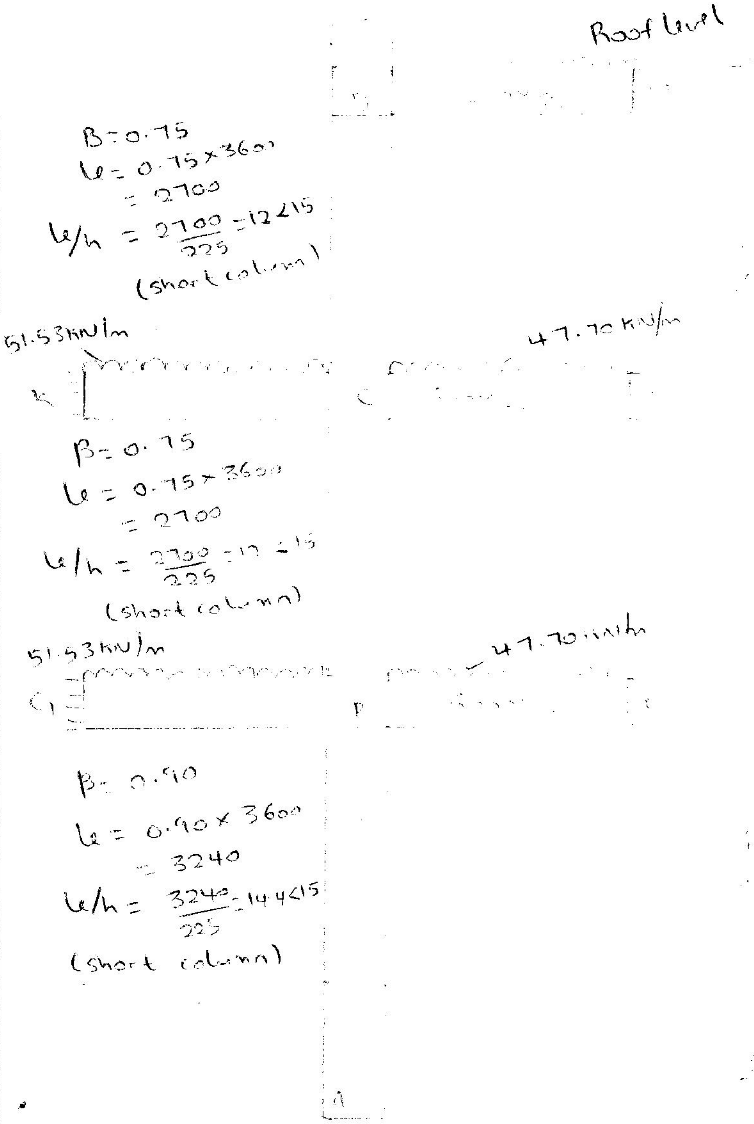
COLUMN DESIGN



REFERENCE	CALCULATIONS	OUTPUT

SLENDERNESS CALCULATIONS

Considering plane AGBFKGJDM



$\beta = 0.75$
$l_e = 0.75 \times 3600$
$= 2700$
$l_e/h = \frac{2700}{225} = 12 < 15$
(short column)

$\beta = 0.75$
$l_e = 0.75 \times 3600$
$= 2700$
$l_e/h = \frac{2700}{225} = 12 < 15$
(short column)

$\beta = 0.90$
$l_e = 0.90 \times 3600$
$= 3240$
$l_e/h = \frac{3240}{225} = 14.4 < 15$
(short column)

$K_{col\,BA} = \frac{0.75 I}{L} = \frac{0.75 bh^3}{12L} = \frac{0.75 \times 225 \times 225^3}{12 \times 4600}$

$= 0.35 \times 10^5$

Beam, $k_{BG} = \frac{1}{2}\left(\frac{225 \times 600^3}{12 \times 6000}\right) = 3.37 \times 10^5$

Beam, $k_{BF} = \frac{1}{2}\left(\frac{225 \times 600^3}{12 \times 6000}\right) = 3.37 \times 10^5$

$K_{col\,BC} = \frac{I}{L} = \frac{225 \times 225^3}{12 \times 3600} = 0.59 \times 10^5$

Beam, $K_{CK} = \frac{1}{2}\left(\frac{225 \times 600^3}{12 \times 6000}\right) = 3.37 \times 10^5$

REFERENCE	CALCULATIONS	OUTPUT

Beam $K_{CJ} = \frac{1}{2}\left(\frac{225 \times 600^3}{12 \times 6000}\right) = 3.37 \times 10^5$

$K_{cd\,co} = \frac{I}{L} = \frac{225 \times 225^3}{12 \times 3600} = 0.59 \times 10^5$

Beam, $K_{DM} = \frac{1}{2}\left(\frac{225 \times 00^3}{12 \times 6000}\right) = 0.84 \times 10^5$

PLANE ABECIDL;

$\beta = 0.75$

$l_e = 0.75 \times 3600$
$= 2700$

$l_e/h = \frac{2700}{225} = 12 < 15$

(short column)

$\beta = 0.75$

$l_e = 0.75 \times 3600$
$= 2700$

$l_e/h = \frac{2700}{225} = 12 < 15$

(short column)

$\beta = 0.90$

$l_e = 0.90 \times 3600$
$= 3240$

$l_e/h = \frac{3240}{225} = 14.4 < 15$

(short column)

REFERENCE	CALCULATIONS	OUTPUT
	$K_{colBA} = \frac{0.75I}{L} = \frac{0.75 \times 225 \times 225^3}{12 \times 4600} = 0.35 \times 10^5$	
	Beam, $K_{BG} = \frac{I}{L} = \frac{1}{2}\left(\frac{225 \times 800^3}{12 \times 6000}\right) = 8.0 \times 10^5$	
	Beam, $K_{CI} = \frac{1}{2}\left(\frac{225 \times 800^3}{12 \times 6000}\right) = 8.0 \times 10^5$	
	$K_{colBC} = \frac{I}{L} = \frac{225 \times 225^3}{12 \times 3600} = 0.59 \times 10^5$	
	$K_{col,CD} = \frac{225 \times 225^3}{12 \times 3600} = 0.59 \times 10^5$	
	Beam, $K_{DL} = \frac{1}{2}\left(\frac{225 \times 300^3}{12 \times 6000}\right) = 0.45 \times 10^5$	
	Moment of Members on Plane ABECIDL:	
	$F.E.M_{DL} = \frac{wl^2}{12} = \frac{4.27 \times 6^2}{12}$	
	$= 12.81 KNm$	
	$Moment_{DC} = \frac{12.81 \times 0.59}{0.42 + 0.59} = 7.48 KNm$	
	$F.E.M_{CI} = \frac{114.99 \times 6^2}{12} = 344.97 KNm$	
	$Moment_{CD} = \frac{344.97 \times 0.59}{0.59 + 8 + 0.59} = 22.17 KNm$	
	$Moment_{CD} = Moment_{CB} = 22.17 KNm$	
	(since they have the same stiffness)	
	Joint B; $Moment_{BE} = \frac{wl^2}{12} = \frac{114.99 \times 6^2}{12}$	
	$= 344.97 KNm$	
	$Moment_{BC} = \frac{344.97 \times 0.59}{0.59 + 8 + 0.35} = 22.77 KNm$	
	$Moment_{BA} = \frac{344.97 \times 0.35}{0.59 + 8 + 0.35} = 13.51 KNm$	
	Moment of Members on Plane AGBFHCJDM	
	$FEM_{DM} = \frac{4.27 \times 6^2}{12} = 12.81 KNm$	
	$Moment_{DC} = \frac{12.81 \times 0.84}{0.84 + 0.59} = 7.52 KNm$	

REFERENCE	CALCULATIONS	OUTPUT

$ULS = 1.4(5.4) + 1.6(4) = 13.96\,KN/m^2$ —— (A)

$S.L.S = 1.0(5.4) = 5.4$ —— (B)

$\text{Factor} = \frac{B}{A} = \frac{5.4}{13.96} = 0.39$

Joint C;

$51.33 \times 0.39 = 20.01\,KN/m$

$FEM_{CK} = \frac{wl^2}{12} = \frac{20.01 \times 6^2}{12} = 60.03\,KNm$

$FEM_{CJ} = \frac{47.7 \times 6^2}{12} = 143.1\,KNm$

Out of balance moment $= 143.1 - 60.03$
$= 83.07\,KNm$

$M_{CD} = \frac{83.07 \times 0.59}{0.59 + 3.37 + 3.37} = 6.69\,KNm$

$M_{CD} = M_{CB} = 6.69\,KNm$

Joint B;

$51.53 \times 0.39 = 20.01\,KN/m$

$FEM_{BG} = \frac{20.01 \times 6^2}{12} = 60.03\,KNm$

$FEM_{BF} = \frac{47.7 \times 6^2}{12} = 143.1\,KNm$

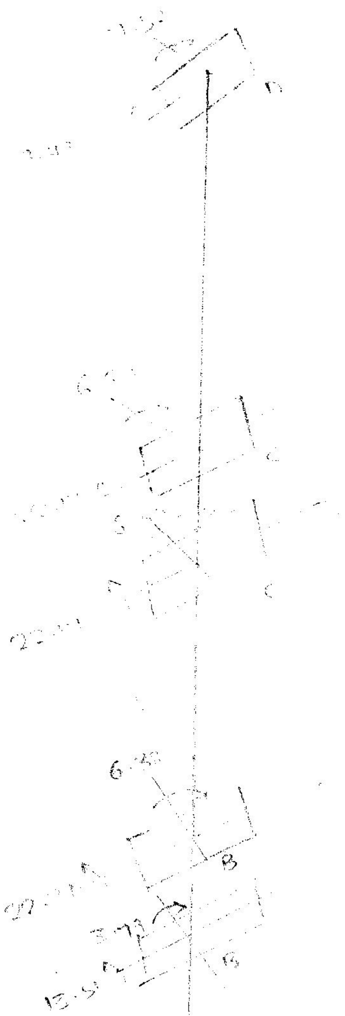
Out of balance moment $= 143.1 - 60.03$
$= 83.07\,KNm$

$M_{BC} = \frac{83.07 \times 0.59}{(0.59 + 3.37 + 3.37 + 0.35)} = 6.38\,KNm$

$M_{BA} = \frac{83.07 \times 0.35}{(0.59 + 3.37 + 3.37 + 0.35)} = 3.79\,KNm$

REFERENCE	CALCULATIONS	OUTPUT
	a. At roof level; from DM = 0.5 × 4.27 × 6 = 12.81 KN	
	from DL = 0.5 × 4.27 × 6 = 12.81 KN	
	25.62 KN	
	Selfweight of column =	
	0.225 × 0.225 × 3.6 × 24(1).4 = 6.12 KN	
	31.74 KN	
	b. At the 2nd floor level	
	From CK = 0.5 × 51.53 × 6 = 154.59 KN	
	CJ = 0.5 × 47.7 × 6 = 143.10 KN	
	CI = 0.5 × 114.99 × 6 = 344.97 KN	
	642.67 KN	
	31.74	
	Selfweight of column 6.12	
	680.53 KN	
	c. At first floor level	
	From BG = 0.5 × 51.53 × 6 = 154.59 KN	
	BF = 0.5 × 47.7 × 6 = 143.10 KN	
	BE = 0.5 × 114.99 × 6 = 344.97 KN	
	642.67	
	680.53	
	Selfweight of column 6.12	
	1329.32 KN	

REFERENCE	CALCULATIONS	OUTPUT



REFERENCE	CALCULATIONS	OUTPUT
	DESIGNING THE MOST LOADED COLUMN	

$M_x = 13.51 kNm$

$M_y = 3.79 kNm$

$N = 1329.32 kN$

$$\frac{M_x}{h'} = \frac{13.51}{(225-25-10-12.5)} = 0.08$$

$$\frac{M_y}{b'} = \frac{3.79}{177.5} = 0.02$$

$$\frac{N}{bhf_{cu}} = \frac{1329.32 \times 10^3}{225 \times 225 \times 25} = 1.05 \geq 0.6$$

$\beta = 0.3$

Since $M_x/h' \geq M_y/b'$

Therefore, $M_x' = M_x + \frac{\beta h'}{b'} M_y$

$$M_x' = 13.51 + 0.3\left(\frac{177.5}{177.5}\right) 3.79$$

$$= 13.51 + 1.137$$

$$= 14.65 \text{ kNm}$$

$$\frac{N}{bh} = \frac{1329.32 \times 10^3}{225 \times 225} = 26.26$$

$$\frac{M}{bh^2} = \frac{14.65 \times 10^6}{225 \times 225^2} = 1.29$$

$$\frac{100 A_{sc}}{bh} = 4$$

$$A_{sc} = \frac{4 \times 225 \times 225}{100} = 2025 mm^2$$

Provide 4-Y25mm + 2.Y16 mm

As prov = $2591.82 mm^2$

REFERENCE	CALCULATIONS	OUTPUT

DESIGNING THE MOST LOADED COLUMN.

$M_z = 13.51 kNm$

$M_y = 3.79$

$N = 1329.32 kN$

$$e_z = \frac{M_z}{N_{ED}} = \frac{13.51 \times 10^6}{1329.32 \times 10^3} = 10.16$$

$$e_y = \frac{M_y}{N_{ED}} = \frac{3.79 \times 10^6}{1329.32 \times 10^3} = 2.85$$

Thus;

$$\frac{e_z}{h} \Big/ \frac{e_y}{b} = \frac{10.16}{225} \Big/ \frac{2.85}{225} = 3.56 > 0.2$$

Therefore, both columns can be designed as biaxially loaded column.

$$\frac{M_z}{h'} = \frac{13.51}{177.5} = 0.08$$

$$\frac{M_y}{b'} = \frac{3.79}{177.5} = 0.02$$

Since $M_z/h' > M_y/b'$

Therefore, $M_z' = M_z + \beta \frac{b'}{h'} M_y$

$$\frac{N_{ED}}{bhf_{ck}} = \frac{1329.32 \times 10^3}{225 \times 225 \times 25} = 1.05 \geq 0.7$$

$\therefore \beta = 0.3$

$$M_z' = 13.51 + 0.3 \left(\frac{177.5}{177.5}\right) 3.79$$

$= 14.65 kNm$

$$\frac{M_{ED}}{bh^2 f_{ck}} = \frac{14.65 \times 10^6}{225 \times 225^2 \times 25} = 0.05$$

$$\frac{N}{bhf_{ck}} = \frac{1329.32 \times 10^3}{225 \times 225 \times 25} = 1.05$$

REFERENCE	CALCULATIONS	OUTPUT
	From design chart;	
	$\frac{A_s f_{yk}}{bhf_{ck}} \times 0.65$	
	$A_s = \frac{0.65 \times bhf_{ck}}{f_{yk}}$	
	$= \frac{0.65 \times 225 \times 225 \times 25}{410}$	
	$= 2006.48$	
	$= 2007mm^2$	
	Provide 4Y25mm + 2Y16mm	
	$A_{s,prov} = 2365.62$	

ROOF BEAM DESIGN TO EUROCODE & BS8110

REFERENCE	CALCULATION	OUTPUT

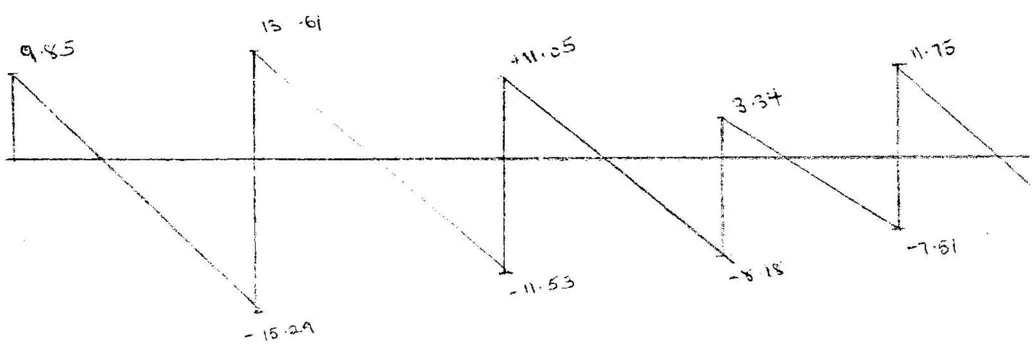
ROOF BEAM.

4.19kN/m

S/w of the beam	S/w of the beam	S/w of the beam	S/w of beam	S/w of the
0.225 X 0.3x 24x 1.35 = 2.19 kN/m	2.19 kN/m	2.19 kN/m	2.19	2.19
Roof Load	ROOF LOAD	Roof Load	Roof Load	Roof L
2 kN/m	2kN/m	2kN/m	2 kN/m	2kN/m

TOTAL LOAD.

2 + 2.19 = 4.19kN/m



REFERENCE	CALCULATION	OUTPUT

DESIGN OF SHEAR REINFORCEMENT.

$V_{max} = 15.29 \, KN$

$V_{RD} = 0.5 \, v f_{ck} b_w d$

$= 0.5 \times 0.575 \times 25 \times 225 \times 247.5$

$= 9606.09 \, N$

$$A_{sw} = \frac{1.28 \, s \, (V_{sd} - V_{Rd1})}{f_{yk} \, d} =$$

$V_{Rd1} = 0.48$

$V_{Rd1} = 0.48 \times 225 \times 247.5 = 26730$

$$157 = \frac{1.28 \, s \, (26730 - 15290)}{250 \times 247.5}$$

$S = 250.49 \, mm$

Provide 2legs Y10mm at 250mm c/c.

DEFLECTION CHECK.

Service Stress, $\sigma_s = \frac{5}{8} \times 410 \times \frac{139.5}{402}$

$= 88.92 \, N/mm^2$

$M.f = \frac{310}{\sigma_s} = \frac{310}{88.92} = 3.5$; Since $m.f > 2$ use $m.f = 2$

$\frac{A_s}{bd} = \frac{402}{225 \times 247.5} = 0.007 = 0.7\%$

To get basic span

0.5	28
0.7	x
1.5	20

$\frac{x - 20}{28 - 20} = \frac{0.7 - 1.5}{0.5 - 1.5}$

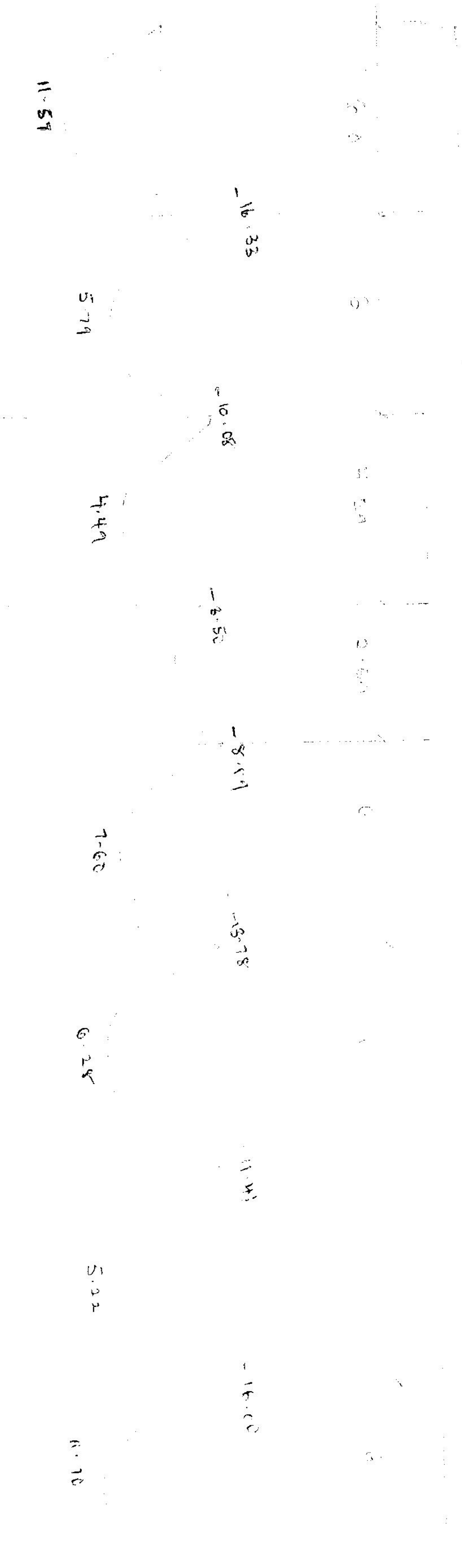
$\frac{x - 20}{8} = \frac{+0.8}{+1}$; $x - 20 = 6.4$

$x = 26.4$

Limiting Span ; m.f x basic span ratio $= 2 \times 26.4 = 52.8$

Actual span ; $l_x/d = \frac{6000}{247.5} = 24.24$

REFERENCE	CALCULATION	OUTPUT.



REFERENCE	CALCULATION	OUTPUT

DESIGN OF SUPPORT REINFORCEMENT.

M_{max} @ Support = 16.33 KN m $d = 300 - 30 - 25/_2 - 10$
$= 247.5mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{16.33 \times 10^6}{25 \times 225 \times 247.5^2} = 0.05$

$la = 0.5 + \sqrt{0.25 - K/_{1.134}} = 0.5 + \sqrt{0.25 - 0.05/_{1.134}} = 0.95$

Use $la = 0.95$

$Z = la\,d = 0.95 \times 247.5 = 235.13$

$AS_{req} = \frac{M}{0.87 f_{yk} Z} = \frac{16.33 \times 10^6}{.87 \times 410 \times 235.13} = 194.7mm^2$

$AS_{min} = 0.0013 \times 225 \times 300 = 87.75m^2$

Since $AS_{min} < AS_{req}$; PROVIDE!!!

Provide 2Y - 16mm $AS_{prov} = 402mm^2$

$AS_{prov} > AS_{min}$; Provision is O.K.

DESIGN OF SPAN REINFORCEMENT.

M_{max} @ Span = 11.70KN.m $d = 247.5mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{11.70 \times 10^6}{25 \times 225 \times 247.5^2} = 0.03$

$la = 0.5 + \sqrt{0.25 - K/_{1.134}} = 0.5 + \sqrt{.25 - 0.03/_{1.134}} = 0.97$

Use $la = 0.95$ Since $0.97 > 0.95$

$Z = la\,d = 0.95 \times 247.5 = 235.13$

$AS_{req} = \frac{M}{0.87 f_{yk} Z} = \frac{11.70 \times 10^6}{0.87 \times 410 \times 235.13} = 139.5mm^2$

$AS_{min} = 0.13\% = 0.0013 \times 225 \times 300$
$= 87.75mm^2$

Provide 2Y - 16mm $AS_{prov} = 402mm^2$

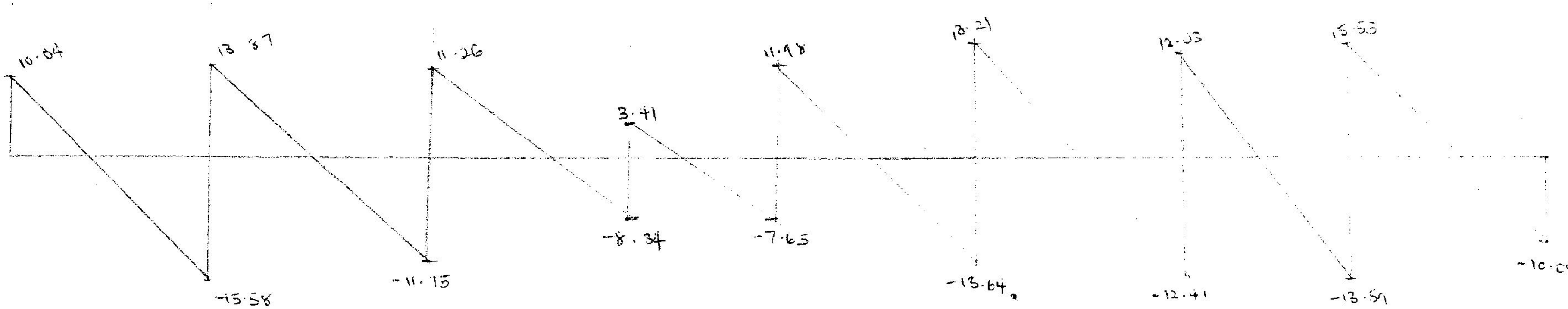
$AS_{prov} > AS_{min}$, PROVISION IS O.K.

REFERENCE	CALCULATION	OUTPUT

ROOF BEAM.

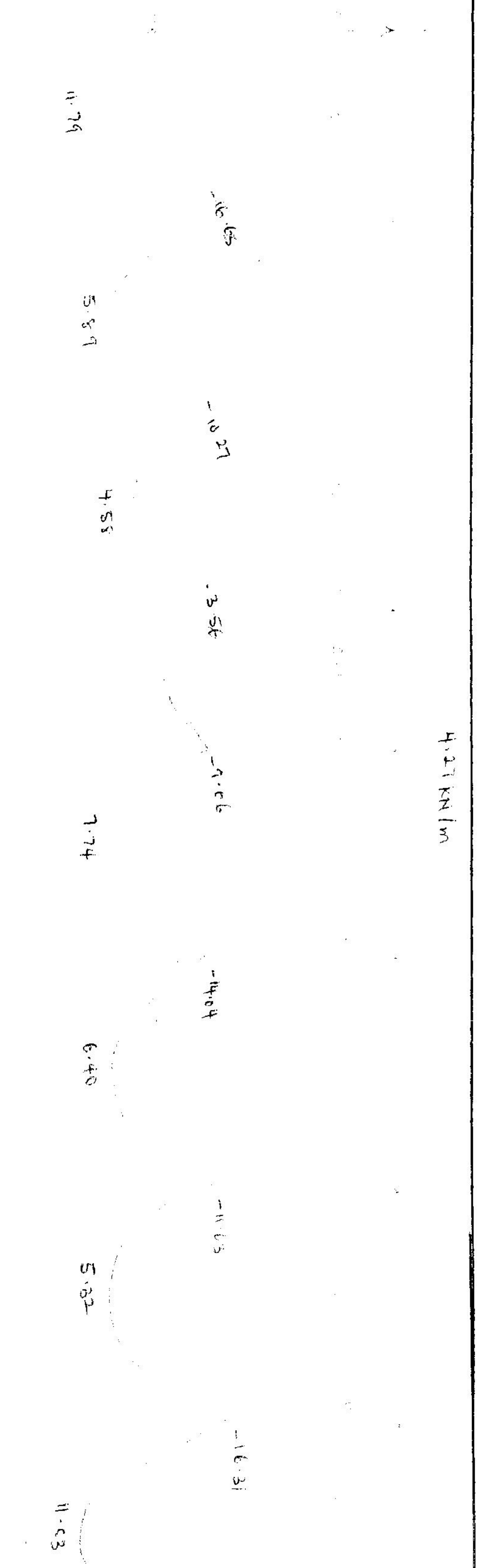
4.27 kN/m

S/w of the beam	S/w of beam	S/w of beam	S/w of beam	S/w of beam	S/w of beam	S/w of beam	S/w of beam
0.225x0.3x24x1.4 = 2.268 kN/m	2.268	2.268	2.268	2.268	2.268	2.268	2.268
Roof Load	**Roof Load**	**Roof Load**	**Roof Load**	**Roof Load**	**Roof Load**	**Roof Load**	**Roof Load.**
2 kN/m	2 kN/m	2 kN/m	2 kN/m	2 kN/m	2 kN/m	2 kN/m	2 kN/m
Total Load	**Total Load**	**Total Load**	**Total Load**	**Total Load**	**Total Load**	**Total Load**	**Total Load.**
2+2.27= 4.27 kN/m	4.27 kN/m	4.27 kN/m	4.27 kN/m	4.27 kN/m	4.27 kN/m	4.27 kN/m	4.27 kN/m



REFERENCE	CALCULATION	OUTPUT
	DESIGN OF SUPPORT REINFORCEMENT.	
	Mmax @ Support = 16.65 KNm.	
	$d = 300 - 30 - 25/2 - 10 = 247.5mm$	
	$K = \frac{M}{f_{cu} b d^2} = \frac{16.65 \times 10^6}{25 \times 225 \times 247.5^2} = 0.05$	
	$la = 0.5 + \sqrt{0.25 - K/0.9} = 0.5 + \sqrt{0.25 - 0.05/0.9} = 0.94$	
	Use la = 0.94 Since la < 0.95.	
	$Z = la\,d = 0.94 \times 247.5mm = 232.65$	
	$AS_{req} = \frac{M}{0.95 f_y Z} = \frac{16.65 \times 10^6}{0.95 \times 410 \times 232.65} = 183.7 mm^2$	
Table 3.25	$AS_{min} = 0.13\% bh = 0.0013 \times 225 \times 300 = 87.75 mm^2$	
	Since $AS_{min} < AS_{req}$; PROVIDE	
	PROVIDE 2T – 16mm $AS_{prov} = 402mm^2$	
	$AS_{prov} > AS_{min}$; Provision is O K.	
	DESIGN OF SPAN REINFORCEMENT.	
	Mmax @ Span = 11.93 KNm $d = 247.5mm$	
	$K = \frac{M}{f_{cu} b d^2} = \frac{11.93 \times 10^6}{25 \times 225 \times 247.5^2} = 0.03$	
	$la = 0.5 + \sqrt{0.25 - K/0.9} = 0.5 + \sqrt{0.25 - 0.03/0.9} = 0.96$	
	Since; 0.96 > 0.95, use la = 0.95	
	$Z = la\,d = 0.95 \times 247.5mm = 235.13 mm$	
	$AS_{req} = \frac{M}{0.95 f_y Z} = \frac{11.93 \times 10^6}{0.95 \times 410 \times 235.13} = 130.26 mm^2$	
Table 3.25	$AS_{min} = 0.13\% bh = 0.0013 \times 225 \times 300 = 87.75 mm^2$	
	Since $AS_{min} < AS_{req}$, Provide.	
	Provide 2T-16mm $AS_{prov} = 402 mm^2$	
	$AS_{prov} > AS_{min}$; PROVISION IS O.K.	

REFERENCE	CALCULATION	OUTPUT



REFERENCE.	CALCULATION	OUTPUT

DESIGN OF SHEAR REINFORCEMENT.

$V_{max} = 15.58\,kN.$

$$v = \frac{V}{bd} = \frac{15.8 \times 10^3}{225 \times 247.5} = 0.28$$

$$v_c = \frac{0.79 \times (400/d)^{1/4} (100As/bd)^{1/3}}{\lambda_m}$$

$$\frac{400}{d} = \frac{400}{247.5} = 1.6$$

$$\frac{100As}{bd} = \frac{100 \times 402}{225 \times 247.5} = 0.72$$

$$v_c = \frac{0.79 \times (0.72)^{1/3} (1.6)^{1/4}}{1.25} = \frac{0.79 \times 0.9 \times 1.12}{1.25}$$

$v_c = 0.64$

Since $v < 0.5 v_c$

$S = 0.75d = 0.75 \times 247.5 = 185.6mm$

Provide 2 leg - Y10mm @ 200mm c/c

DEFLECTION CHECK. (At Span)

S... ... ...

FLAT SLAB

DESIGN TO EUROCODE
& BS8110.

REFERENCE	CALCULATIONS	OUTPUT

FLAT SLAB

Effective diameter of column head,

$$h_c = \left(\frac{44}{\pi}\right)^{1/2} \leq 0.25 l_c$$

Depth of drops = 100mm

Column size = 300mm x 300mm

Column grid = 6m x 6m

Flat slab has, 2000mm x 2000mm drops at column

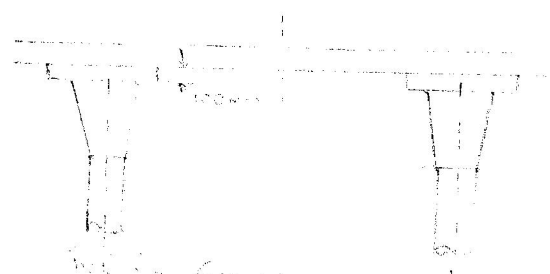
Assume flat slab thickness of 200mm.

Live load on floor = 4 kN/mm².

$$h_c = \left(\frac{4 \times 300 \times 300}{\pi}\right)^{1/2} \leq 0.25(6000)$$

$$= 338.51\text{mm} \leq 1500\text{mm} \quad \text{(O.K)}$$

Use h_c = 600mm



Dead load:

Weight of slab = $0.2 \times 24 \times 6.0^2$
= 172.8 kN

Weight of drop = $0.1 \times 24 \times 2^2$
= 9.6 kN

Total load = 172.8 + 9.6 = 186.40 kN

Live load:

Total live load = 4×6^2 = 144 kN

Ultimate load on floor, w

w = 1.35(186.40) + 1.5(144)
= 467.64 kN per panel

Equivalent distributed load, $n = \frac{467.64}{6^2}$
= 12.99 kN/m².

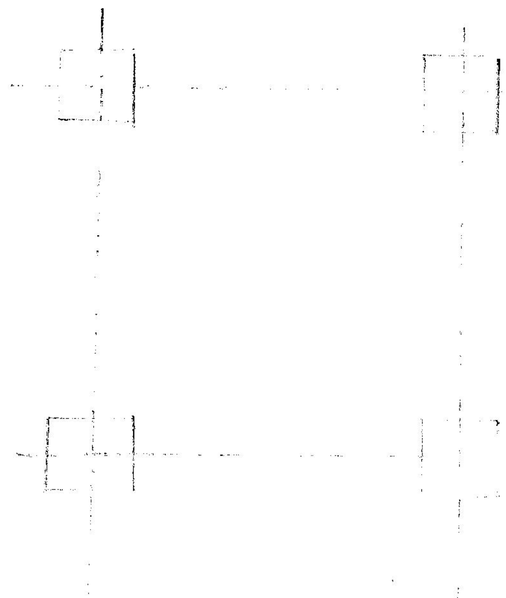
REFERENCE	CALCULATIONS	OUTPUT

The effective span, L = clear span $2h_c/3$.

$$= 6.0 - \frac{2 \times 0.6}{3}$$

$$= 5.6m$$

Since the drop dimension is greater than one-third of the panel dimension, therefore the column strip is taken as the width of the drop down panel (2.0m).



BENDING REINFORCEMENT

i) Centre of interior span

Positive moment = $0.071wl$

$= 0.071 \times 467.64 \times 5.6$

$= 185.93$ KNm

∴ Width of middle strip is $(6.0 - 20) = 4m$, which is greater than half of the panel dimension, therefore proportion of positive moment taken by the middle strip is given by;

$$\frac{45}{100} \times \frac{4}{6/2} = 0.6$$

Thus the middle strip moment = 0.6×185.93
$= 117.22$ kNm

The column strip positive moment =
$(1 - 0.6) \times 185.93$
$= 111.56$ KNm

REFERENCE	CALCULATIONS	OUTPUT

The column strip positive moment $= (1-0.6)\times 185.93$

$= 74.37$ kNm

a. for middle strip

$$K = \frac{M}{f_{cu}bd^2} = \frac{111.56\times10^6}{25\times4000\times151^2}$$

$= 0.05$

where $d = h - c - \phi - \phi/2$

$= 200 - 25 - 16 - 8$

$= 151$mm.

$$l_a = 0.5 + \sqrt{0.25 - K/1.134}$$

$= 0.95$

$Z = l_a d = 0.95\times151$

$= 143.45$ mm

$$As_{req} = \frac{M}{0.87 f_y Z} = \frac{111.56\times10^6}{0.87\times410\times143.45}$$

$= 2180.24$ mm^2

Provide 11 - Y16mm, $As_{prov} = 2212$ mm^2.

b) For column strip

$$K = \frac{M}{f_{cu}bd^2} = \frac{74.37\times10^6}{25\times2000\times155^2} = 0.06$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.06}{1.134}}$$

$= 0.94$

$Z = l_a d = 0.94\times155$

$= 145.7$mm

$$As_{req} = \frac{74.37\times10^6}{0.87\times410\times145.7} = 1430.99\text{mm}^2$$

Provide 8 - Y16mm, $A_{s\,prov} = 1610$ mm^2

2) Interior support

Negative moment $= -0.055 F_L$

$= -0.055\times467.64\times56$

$= 144.03$ KN

And this is divided into;

middle strip $= 0.25\times\frac{4}{6/2}\times144.03$

$= 48.01$ kNm

Column strip $= (1-0.33)\times144.03$

$= 96.50$ kNm

REFERENCE	CALCULATIONS	OUTPUT
	a) for middle strip	
	$K = \frac{M}{f_{ck}bd^2} = \frac{48.01 \times 10^6}{25 \times 4000 \times 151^2}$	
	$= 0.021$	
	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	
	$= 0.98$	
	Since l_a is greater than 0.95, we use $l_a = 0.95$	
	$Z = 0.95 \times 151$	
	$= 143.45mm$	
	$A_{s\,req} = \frac{48.01 \times 10^6}{0.87 \times 410 \times 143.45}$	
	$= 938.27mm^2$	
	Provide 9-Y12mm, $A_{s\,prov} = 1020mm^2$	
	b) For column strip	
	$d = 300 - 25 - 16 - 8$	
	$= 251mm$	
	$K = \frac{M}{f_{ck}bd^2} = \frac{96.50 \times 10^6}{25 \times 2000 \times 251^2}$	
	$= 0.031$	
	$l_a = 0.5 + \sqrt{0.25 - \frac{0.031}{1.134}}$	

Reference	Calculations	Output.
	Flat Slab	
	Effective diameter of column head,	
	$h_c = \left(\frac{4A}{\pi}\right)^{1/2} \leq 0.25 l_n$	
	Depth of Drops = 100mm.	
	Column Size = 300mm x 300mm	
	Column Grid = 6m x 6m	
	Flat Slab has 2000mm x 2000mm drops at Column.	
	Assume flat slab thickness of 200mm.	
	Live Load On floor: $4kN/m^2$	
	$h_c = \left(\frac{4 \times 300 \times 300}{\pi}\right)^{1/2} \leq 0.25(6000)$	
	$= 338.51mm \leq 1500mm.$ (O.K)	
	Use $h_c = 600mm$.	
	2.0m² drops	
	h=200mm	

REFERENCE	CALCULATIONS	OUTPUT.

Ultimate Load On floor, F

$= 1.4(186.45) + 1.6(144)$.

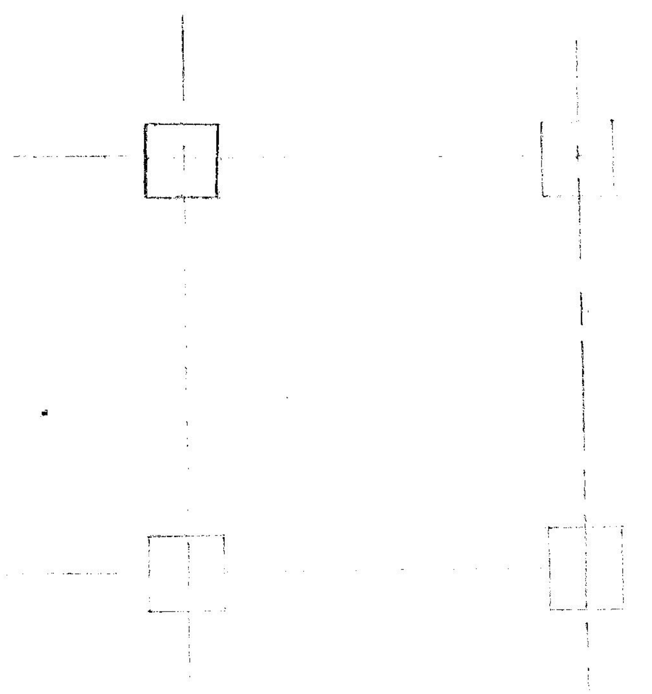
$= 491.36$ kN Per Panel.

Equivalent Distributed Load, $n = \frac{491.36}{6^2} = 13.65$ kN/m²

The Effective Span L = Clear Span $2h_c/3$.

$= 6.0 - \frac{2 \times 0.6}{3} = 5.6$m.

Since the drop dimension is greater than one-third of the panel dimension, therefore the column strip is taken as the width of the drop down panel (2.0m).



REFERENCE	CALCULATIONS	OUTPUT

BENDING REINFORCEMENT

1) Centre Of Interior Span

Positive moment $= 0.071FL$

$= 0.071 \times 491.36 \times 5.6 = 195.36$ kN.m.

∴ the Width of middle Strip is $(6.0-2.0) = 4m$, which is greater than half of the panel dimension, therefore Proportion of positive moment taken by the middle ~~Span~~ Strip is given by;

$$\frac{45}{100} \times \frac{4}{6/2} = 0.6$$

Thus the middle strip moment $= 0.6 \times 195.36$

$= 117.22$ kN.m.

The column strip Positive Moment $= (1-0.6) \times 195.36$

$= 78.14$ kN.m.

a) For middle strip.

$$K = \frac{M}{f_{cu}bd^2} = \frac{117.22 \times 10^6}{25 \times 4000 \times 151^2} = 0.05$$

Where here $d = h - c - \phi - \phi/2$.

$= 200 - 25 - 16 - 8$

$= 151$ mm.

$$l_a = 0.5 + \sqrt{0.25 - K/0.9}.$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.05}{0.9}} = 0.94$$

$Z = l_a d = 0.94 \times 151 = 141.94.$

$$A_{sreq} = \frac{M}{0.95 f_y Z} = \frac{117.22 \times 10^6}{0.95 \times 410 \times 141.94} = 2120.26 mm^2$$

Provide 11-Y16mm, $A_{sprov} = 2212 mm^2$

b) For Column Strip.

$$K = \frac{M}{f_{cu}bd^2} = \frac{78.14 \times 10^6}{25 \times 2000 \times 155^2} = 0.07$$

REFERENCE	CALCULATIONS	OUTPUT.

$l_a = 0.5 + \sqrt{0.25 - \frac{0.07}{0.9}} = 0.91$

$Z = l_a d = 0.91 \times 15\cancel{1} = \cancel{141.05}\ 137.41$

$A_{sreq} = \frac{78.14 \times 10^6}{0.95 \times 410 \times 137.41} = 1460.00 mm^2$

Provide 8-Y16mm, $A_{sprov} = 1610 mm^2$

2) Interior Support

Negative Moment $= -0.055 Fl$

$= -0.055 \times 491.36 \times 5.6$

$= 151.34$ KN.m.

And this is also divided into;

Middle Strip $= 0.25 \times \frac{4}{6.0/2} \times 151.34 = 50.45$ KN.m.

Column Strip $= (1 - 0.33) \times 151.34 = 100.89$ KN.m

a) for middle strip

$K = \frac{M}{f_{cu} b d^2} = \frac{50.45 \times 10^6}{25 \times 4000 \times 151^2} = 0.022$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.022}{0.9}} = 0.97$

Since l_a is greater than 0.95, Use $l_a = 0.95$

$Z = 0.95 \times 151 = 143.45$

$A_{sreq} = \frac{50.45 \times 10^6}{0.95 \times 410 \times 143.45} = 903 mm^2$

Provide 9-Y12mm, $A_{sprov} = 1020 mm^2$

b) For Column Strip

$d = 300 - 25 - 16 - 8 = 251 mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{100.89 \times 10^6}{25 \times 2000 \times 251^2} = 0.032$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.032}{0.9}} = 0.96$

Since l_a is greater than 0.96 Use $l_a = 0.95$

$Z = 0.95 \times 251 = \cancel{28}\ 238.45$

Reference	Calculations	Output

$$A_{sreq} = \frac{100.89 \times 10^6}{0.95 \times 410 \times 238.45} = 1086.30\ mm^2.$$

Provide 6-Y16mm, $A_{sprov} = 1210 mm^2/m$.

PUNCHING SHEAR:-

1) At column head,

Perimeter, $u = \pi d$
$= 3.142 \times 600$
$= 1885.2 mm$.

Shear force $V = F - \frac{\pi}{4} \times 0.6^2 \times n$

$$= 491.36 - \frac{3.142 \times 0.6^2}{4} \times 13.65$$

$= 487.5$ kN

To allow for the effects of moment transfer, V is increased by 15 per cent, thus

$$v = \frac{1.15V}{ud} = \frac{1.15 \times 487.5}{1885.2 \times 251} = 1.18\ N/mm^2$$

which is less than $0.8\sqrt{f_{cu}}$ or $5 N/mm^2$.

2) First Critical Perimeter is $1.5d = 1.5 \times 251$
$= 376.5 mm$.

Thus length of Perimeter, $u = 4(600 + 2 \times 376.5)$
$= 5412 mm$.

~~thus, Shear Stress v =~~

Ultimate Shear force, $= 491.36 - (0.6 + 2 \times 0.376)^2 \times 13.65$
$= 466.41$ kN.

Thus Shear Stress $v = \frac{1.15 \times 466.41 \times 10^3}{5412 \times 251}$

$= 0.39 N/mm^2$.

STRUCTURAL DRAWING IS SHOWN IN THE APPENDIX.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Results

The results of the design and analysis for the structural elements according to BS 8110-1997 and Eurocode2 as determined manually are as presented. To create a neutral base for comparison as regards bending moments and shear forces. Table 4.1 shows the input data used in generating total load on slab.

Table 4.1: Input data for both codes

Parameter	BS 8110	EUROCODE 2
Concrete unit weight	$24KN/m^3$	$24KN/m^3$
Overall depth, h	$175mm$	$175mm$
Width, b	$1000mm$	$1000mm$
Imposed load	$4KN/m^2$	$4KN/m^2$

Table 4.2: Percentage difference in area of steel required for slab

	A_s required(mm^2)		%
	BS 8110	EC 2	difference
Short span mid span	313.1	301.03	3.85
Short span continuous edge	421.1	404.8	3.87
Long span mid span	330	316.93	3.96
Long span continuous edge	435.9	423.35	2.89
			Average 3.64

Table 4.3: Span moment of beam

Span	Length (m)	Span Moment	
		BS 8110	EC 2
AB	6	315.26	301.74
BC	6	87.45	83.38
CD	6	87.45	83.38
DE	6	315.26	301.74

Table 4.4: Percentage difference in area of Steel required for maximum span support

Span	Length (m)	Maximum Span Moment		As required (mm^2)		% difference
		BS 8110	EC 2	BS 8110	EC 2	
AB & DE	6	315.26	301.74	1319.76	1382.67	4.29

Table 4.5: Percentage difference in area of Steel required for maximum support moment

Support	Maximum Support Moment		As required (mm^2)		% difference
	BS 8110	EC 2	BS 8110	EC 2	
B & D	382.96	366.49	1817.5	1812.97	4.31

Table 4.6: Percentage difference in total weight of steel required for slab

Type (mm)	Unit weight (kg/m)	Total length (m)		Total weight (kg)		% difference
		BS 8110	EC2	BS 8110	EC2	
Y12	0.8878	10541.35	10400.13	9358.61	9233.24	1.25

Table 4.7: Percentage difference in total weight of steel required for column

Type (mm)	Unit weight (kg/m)	Total length (m)		Total weight (kg)		% difference
		BS 8110	EC2	BS 8110	EC2	
Y16	1.5783		1519.3	0	2397.91	0
Y20	2.4662	1240.88	0	3035.44	0	0
Y25	3.8534	3032.6	2983.75	11685.82	11497.58	1.88
						Average 1.88

Table 4.8: Percentage difference in total weight of steel required for beam

Type (mm)	Unit weight (kg/m)	Total length (m)		Total weight (kg)		% difference
		BS 8110	EC2	BS 8110	EC2	
Y10	0.6185	54893.5	54893.5	33951.63	33951.33	0
Y16	1.5783	1054.7	1054.70	1664.63	1664.63	0
Y20	2.4662	1240.88	1240.88	3060.26	3060.26	0
Y25	3.8534	480	480	1849.63	1849.63	0

4.2 Discussion of Results

The percentage difference for area of reinforcement between the two codes was calculated with the BS8110 values as controls. For the combination of dead and imposed loads considered, the average percentage difference for the span moments of the BS8110 exceeds that of the Eurocode 2 by 4.29%, while the average support moments for the BS8110 exceeds those of the Eurocode2 by 4.31%.

In the case of slab, the average percentage difference between the areas of steel required for the BS8110 exceeds that of the Eurocode2 by 3.64% for both short and long span.

The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.25% for slabs.

The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.88% for columns.

CHAPTER FIVE

CONCLUSION

The results of the comparative study led to the following conclusions:

1. The BS8110 moments exceeds that of the Eurocode2 by an average of about 4.29% at spans and 4.31% at supports for beams.
2. The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.25% for slab.
3. The average percentage difference in total weight of steel required for the exceeds that of the Eurocode by 1.88% for column.
4. The Eurocode2 is more conservative in terms of the partial factors of safety for loadings.
5. Based on the results obtained, it can be concluded that buildings designed by the provisions of the Eurocodes are more economical with the required margin of safety.

REFERENCES

1991-1, E. (1994). *Basis of Design and Actions on Structures. Part 1.*

Ajis, N. A. (2012). *Study on the Improvement of Code of Practices in Structural Design and Material Specification.* Malaysia.

Al- Ansari, N., Al- Taie, E. and Knutsson, S. (2014). The Need to Develop a Building Code for Iraq. *Scientific Research,* 610-632.

Alnuaimi, A. S. and Patel, I. Y. (2013). Serviceability limit state, bar aanchorage and lap lengths in ACI318:08 and BS8110:97: A Comparative Study. *The Indian Concrete Journal.*

Arya, C. (2009). *Design of Structural Elements.* New York, U.S.A.: Taylor and Francis.

Building Code. (2013). Retrieved June 6, 2016 from http://www.co.ozaukee.wi.us/DocumentCenter/Home view 315

Council, I. C. (2005). *Setting the Standard for Building Safety.* International Code Council.

Galambos, T. V. (n.d.). Structural Design Codes: *The Bridge between Research and Practice*

Narayanan, R. S. and Webster, R. (1994). *Concrete Structures: Eurocode EC2 and BS8110 Compared.* 1st Edition. Harlow: Londman Scientific and Technical.

Nwofor , T.C., Sule, S. and Eme, D.B. (2015). A Comparative Study of BS8110 and Eurocode 2 Standards for Design of a Continuous Reinforced Concrete Beam. *Journal Impact Factor,* 76-84.

O'Brien, E. J. and Dixon, A.S. (1995). *Reinforced and Prestressed Concrete Design: The Complete Process.* United Kingdom: Longman Scientific and Technical.

Reynolds, C. E. and Steedman, J. C. (1999). *Reinforced Concrete Designer's Handbook.* Tenth Edition. London: Taylor and Francis Group.

Shafii, F., Omar, W., Mohammad, S. and Makhtar, A. M. (2001). Standardisation of Structural Design: A Shift From British Standards to Eurocodes. *Jurnal Teknologi*, 34(B): 21-30.

Taylor, H. P. and Burgoyne, C. J. (2008). Structural design and the Eurocodes- a historical review. IStructE.

The aims of the Eurocodes. (2008). Retrieved September 10, 2008 from http://www.eurocode2.info/main.asp

Yeager, M. (2013). *History of Building Codes*. Retrieved June 5, 2016, from Buyer's Choice Home Inspection: http://www.buyerschoiceinspections.com/history-of-building-codes

Yusoff, T. S. (2015). Advantage using Eurocode 3 for design of factor frame. *Journal of basic and applied scientific research*, 5(11)11-16.