CHARACTERISTICS OF EUROCODES AND BS8110 AS DESIGN TOOLS FOR REINFORCED CONCRETE STRUCTURES

By

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ABSTRACT

The structural design of most buildings worldwide is based on national and/or international codes of practice. These guide the engineer in the appraisal of the overall structural scheme, detailed analysis and design. Codes of practice are basically aids drawn up by experienced engineers and allied professionals, and they provide a framework for addressing issues of safety and serviceability in structural design.

Structural Euro codes are a set of European design standards which introduced a common technical language and a common technical culture in structural design. This facilitated the creation of an effective Internal Market within the European Union, by removing potential barriers to trade that could exist when Member States have different national design standards.

BS 8110 is a British Standard for the design and construction of reinforced and prestressed concrete structures. It 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).

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 results of the design and analysis for the structural elements according to BS 8110-1997 and Euro code 2 was determined manually and was as presented. To create a neutral base for comparison as regards the characteristic bending moments and shear forces. 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 Euro code 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 Eurocode2 exceeds that of the BS8110 by 3.64% for both short and long span.

CERTIFICATION

This is to certify that this project was written by OKEKE, Obinna Wilson (CVE/11/0372) under my supervision and is approved for its contribution to knowledge and literary presentation. All sources of information are specifically acknowledged by means of references, in partial requirements for the award of Bachelor of Engineering (B.Eng.) degree in civil Engineering, Federal university Oye-Ekiti, Ekiti, Nigeria.

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DEDICATION

This project is dedicated to the lord almighty.

ACKNOWLEDGMENT

The most profound gratitude to the Almighty God for His Benevolence and mercy in my studies and life who has despite the challenges and tests shown His faithfulness always.

My sincerest thanks to the entire academic and non-academic staff of the department of Civil engineering, Federal university Oye-Ekiti for their continuing efforts in imparting knowledge into us the student and equipping us with the necessary tools needed to thrive in this world. Special thanks to my supervisor for his advice, love and his candor and my profound gratitude to Olojo Adebowale Kosoko, Emate Victor, Josu Gigonu micheal (Agba Awo) for their support.

I am grateful to my parents whose foundation of hard work and honest lifestyle has helped build me into who I am, my siblings who spared nothing in helping me in every of my desire to succeed, my friends, who have been a huge positive influence in my life, a gift from God Himself and my colleagues who created a healthy competition to thrive and to push me to come out on top and to everyone who spared a second to give me good advice. Thank you and God bless you.

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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

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CHAPTER ONE

INTRODUCTION

1.0 Background

Codes are as old as antiquity. Religious traditions and civic cultures have codes as their foundations. The Mosaic Decalogue (Ten Commandments) is the keystone for Judaism, Islam and Christianity. Pericles made the Athenian code the underpinning of ancient Greek politics and culture. (Gilman) In each case codes carry general obligations and admonitions, but they are far more than that. They often capture a vision of excellence, of what individuals and societies should be striving for and what they can achieve. In this sense codes, which are often mistaken as part of law or general statements of mere aspiration, are some of the most important statements of civic expectation. When applied to certain classes of people – public servants, doctors – codes are the ultimate terms of reference. They are the framework upon which professions are built. Often codes are what professionals use to make the claim that they are "professionals" and are often the founding document for a profession, e.g. the Hippocratic Oath. Because the term code is often used in different contexts its meaning can be confused. For our purposes code is not synonymous with law. (Gilman) Laws can have codes within them. But legal systems are not codes (e.g. Hammurabi's Code) in the way the term "code" is used in this document. Laws, often referred to as legal codes, are a series of detailed proscriptions dealing with the "crime or offense" and the punishment. An example would be a city code forbidding spitting on the sidewalk that provides a 30 day jail sentence for violations.

Codes aim is to ensure obtainment of constructive product with adequate standard reliability. That is achieved by elaboration of unifying suitable methodologies relating to the whole design-construction process, from qualification of technical operators to acquisition of data on surrounding actions and structural elements and material to modelling for verification and finally to checking operations. The code is a starting point as regards to design and execution phases of realization process, conditioning final constructive product which has to be in conformity to it. On the contrary, it is an end-point as regards the process connected with forming of knowledge and technology inheritances being a consequence of them. (Jennings, 1996)

1.2 The Law and Code of Hammurabi

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. King Hammurabi, the famous law-making Babylonian ruler who reigned from approximately 1955 to 1913 B.C., is probably best remembered for the Code of Hammurabi, a statute primarily based on retaliation. (http://www.ct.upt.ro/users/AurelStratan/)

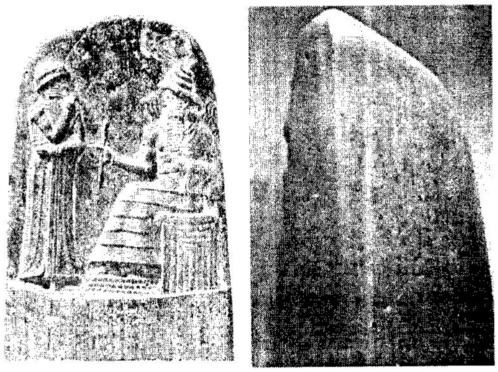


Fig1.1 statue of Hammurabi

The following decree is from the Code of Hammurabi: Only one example of the Code survives today, inscribed on a basalt stone stele. Originally, several stele 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 (http://www.ct.upt.ro/users/AurelStratan/)

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

- 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.
- 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.

Today, society no longer endorses Hammurabi's ancient law of retaliation but seeks, rather, to prevent accidents and loss of life and property. From these objectives have evolved the rules and regulations that represent today's codes and standards for the built environment (http://www.ct.upt.ro/users/AurelStratan/)

1.3 Building codes

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: principles of structural design guidance in evaluation of loads on structures 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. (http://www.ct.upt.ro/users/AurelStratan/)

Forms of building codes

Codes can often be classified as specifications type or performance type

Specifications type codes: Names specific materials for specific uses and specifies minimum or maximum dimensions, for example, "A brick wall may not be less than 40 cm thick".

Performance type codes: Specifies required performance of a construction but leaves materials, methods, and dimensions for the designers to choose.

Performance-type codes are generally preferred, because they give designers greater design freedom in meeting clients' needs, while satisfying the intent of the code.

Most codes are rather a mixture of specifications and performance type. The reason for this is that insufficient information is currently available for preparation of an entire enforceable performance code. (http://www.ct.upt.ro/users/AurelStratan/)

1.4 Codes of Ethics Today

Most professions regularly amend their codes of ethics. Many have undertaken drastic revisions more than once. But engineering seems to be unique in the number of competing codes proposed and adopted over the years. (Gilman)

Chief among the explanations often advanced for the number of codes is that engineering is simply too diverse for one code of ethics to apply to all. Some engineers are independent practitioners. Some are employees of large organizations. Some are managers. Many are closely supervised. Some, whether in large organizations or on their own, are more or less their own boss. Engineers (it is said) just do too many different things for the same standards to apply to all. In sum, engineering is not a single profession but a family of historically related professions. Though much rings true in this explanation of the number of codes of ethics, something rings false as well. If the divisions in engineering were like that, say, between medicine and dentistry, why would engineers establish "umbrella" organizations and devote so much time to trying to achieve one code for all engineers? Doctors and dentists have not made similar efforts to write a single code of ethics for their two professions. The three-quarters of a century engineers have tried to write a code for all engineers is like the existence of schools of engineering evidence that engineers all belong to one profession, however divided and diverse its membership. Indeed, we might think of the effort to write a single code as an attempt to preserve the unity of the profession. (Jennings, 1996)

The structural design of most buildings worldwide is based on national or international codes of practice. These guide the engineer in the general appraisal of the overall structural scheme, detailed analysis and design. Codes of practice are basically guides drawn up by experienced engineers and a team of professionals, and they provide a framework for addressing issues of safety and serviceability in structural engineering design. In the African continent, national codes of practice have been primarily derived from the British standard BS8110-1997 and its predecessors. 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 Euro codes (ECs). (Franklin, 2011)The Euro codes are a new set of European structural design codes for building and civil engineering works. The Euro codes have been introduced as part of the wider European harmonization process and not just simply to directly replace any national codes. (Franklin, 2011)

1.4.1 BS 811O

BS 8110 is a British Standard for the design and construction of reinforced and pre stressed concrete structures. It 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). In 2010 BS 8110 was superseded by EN 1992 (Euro code 2) although parts of the standard have been retained in the National Annex of the Euro code. 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 Euro codes (ECs). These

comprehensive set of harmonized ECs for the structural and geotechnical design of buildings and civil engineering works were first introduced as Euronorme Voluntaire (ENV) standards, intended for use in conjunction with national application documents (NADs) as an alternative to national codes such as BS8110-1997 for a limited number of years. Subsequently these have been largely superseded by Euronorme (EN) versions with each member state of the European community adding a National Annex (NA) containing nationally determined parameters with the object of implementing the ECs as a national standard. It should be stressed at this juncture however that the ECs have been introduced as part of the wider European harmonization process and not just simply to directly replace any national codes.

1.4.2 Euro Codes

The EN Euro codes are a set of European standards which provide common rules for the design of construction works, to check their strength and stability against live extreme loads such as fire and earthquakes. In line with the EU's strategy for smart, sustainable and inclusive growth (EU2020), standardization plays an important part in supporting the industrial policy for the globalization era. (1, 1994)

In 1975, the Commission of the European Community (presently the European Commission), decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was to eliminate technical obstacles to trade and the harmonization of technical specifications. Within this action programme, the Commission took the initiative to establish a set of harmonized technical rules for the design of construction works which, in a first would serve as an alternative to the national rules in force in the member states of the European Union (EU) and, ultimately, would replace them. For fifteen years, the Commission, with the help of a steering committee with representatives of the member states, conducted the development of the Euro codes programme, which led to the first generation of European codes in the 1980's. In 1989, the Commission and the member states of the EU and the European Free Trade Association (EFTA) decided, on the basis of an agreement between the Commission and to transfer the preparation and the publication of the Euro codes to the European Committee for Standardization (CEN) through a series of mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Euro codes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/1 06/EEC on construction products - CPD and Council Directives 93/37/EEC, 921S0lEEC and 89/440lEEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market). (British Standard Institution, 1985)

The Euro codes are pan-European structural design codes. There are ten Euro codes in a total of 58 parts covering: basis of design; actions on structures; design of structural elements in concrete, steel, composite steel and concrete, timber, masonry and aluminum; together with geotechnical and seismic design. They cover the design of bridges, buildings, silos, tanks, pipelines, towers, masts and more.

The purpose of the Euro codes is to provide:

- A means to prove compliance with the requirements for mechanical strength and stability and safety in case of fire established by European Union law.
- A basis for construction and engineering contract specifications.
- A framework for creating harmonized technical specifications for building products (CE mark).

By March 2010 the Euro codes are mandatory for the specification of European public works and are intended to become the de facto standard for the private sector. The Euro codes therefore replace the existing national building codes published by national standard bodies (e.g. BS 5950), although many countries had a period of co-existence. Additionally, each country is expected to issue a National Annex to the Euro codes which will need referencing for a particular country (e.g. The UK National Annex). At present take up of Euro codes is slow on private sector projects and existing national codes are still widely used by engineers. (British Standard Institution, 1985)

The verification procedure in the Euro codes is based on the limit state concept used in conjunction with partial safety factors. The Euro codes allow also for design based on probabilistic methods as well as for design assisted by testing, and provide guidance for the use of these methods.

The Euro codes suite

The Euro codes suite is made up by 10 European Standards for structural design

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

EN 1992 Eurocode 2: Design of concrete structures

EN 1993 Eurocode 3: Design of steel structures

EN 1994 Eurocode 4: Design of composite steel and concrete structures

EN 1995 Eurocode 5: Design of timber structures

EN 1996 Eurocode 6: Design of masonry structures

EN 1997 Eurocode 7: Geotechnical design

EN 1998 Eurocode 8: Design of structures for earthquake resistance

EN 1999 Eurocode 9: Design of aluminum structures

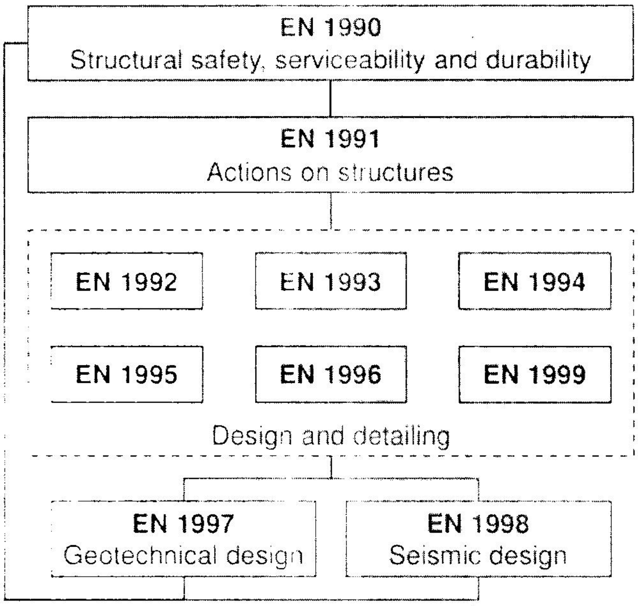


Fig 1.2: Links between the Euro codes

Differences between the Euro codes and the British Standards, Is there a different design philosophy?

The Euro codes are limit state codes like the British Standards, although are perhaps a little more explicitly based in reliability theory. Many of the Eurocode rules are based on the same theory as the British Standards, although the Euro codes embody the most up to date research on many aspects of structural behavior.

The Eurocode clauses are structured in a slightly different way in that they contain principles that must be satisfied and application rules that offer a way of satisfying the principles. This is intended to stimulate innovation. The Euro codes are also less prescriptive than the British Standards, with more aspects left open to the designer. (1, 1994)

Implementation of the Euro codes

When an EN Eurocode Part is made available by CEN (Date of Availability), National Authorities and National Standards Bodies should:

translate the Eurocode Part in authorised national languages

set the Nationally Determined Parameters to be applied on their territory

publish the National Standard transposing the EN Eurocode Part and the National Annex
adapt their National Provisions so that the EN Eurocode Part can be used on their territory
promote training on the Euro codes

- The implementation of an EN Eurocode Part has three phases:

Translation period (max 1 year). The National Standards Bodies may start the translation of a Eurocode Part in authorised national languages at the latest at the Date of Availability.

- National Calibration period (max 2 years). The Member States should fix the Nationally Determined Parameters. At the end of this period, the national version of the EN Eurocode Part with the National Annex will be published by the National Standards Bodies. Also, the Member States should adapt the National Provisions so that the Eurocode Part can be used on their territory.
- Coexistence period. During the coexistence period, which starts at the end of the National Calibration period, the Eurocode Part can be used, just as the presently existing national system can also be used. The coexistence period of a Eurocode Package will last up to a maximum time of three years after the national publication of the last Part of a Package. Member States shall make sure that all the Parts of the related Package can be used
without ambiguity on their territory by adapting their National Provisions as necessary.

All conflicting National Standards in a Package should be withdrawn a maximum of 5 years after the Date of Availability of the last available Part in the Package Following CEN rules, the Euro codes can be used in parallel with National Standards until 2010, when all conflicting National Standards will be withdrawn. (Future, 2006)

1.3 Statement of Problems

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 recognized 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.

1.5 Aim

The aim of this project is to compare the analysis of BS 811O and Euro code 2 as a method of design using a library complex.

1.6 Objectives

To analyze and compare the BS 8110 and EUROCODE as method of design using a library complex.

Production of arrangement and detail drawings

To make analysis of the results obtained by using the conventional BS code method and Euro code method

To make recommendations based on the outcome of the analysis

1.7 Scope

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.

1.8 Definition of Terms

ACI	American Concrete Institute
ACI 318-11	Building Code Requirements for Structural Concrete
ASCE 7-10	Minimum Design Loads for Buildings and Other Structures

BS 8110	Structural Use of Concrete
BS 6399	Loadings for Buildings
EC	Eurocode
EC2	Eurocode2 (Design of Concrete Structures)
UAC	Unified Arabic Code
CSA-A23.3-94	Canadian Code
EC0	Basis of structural design
EC1	Actions on structures

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

The Oxford Advanced Learners" Dictionary defines a building as a structure such as a house or school that has a roof and walls. The same dictionary also defines a structure as the ways in which the parts of something are connected together, arranged or organized. However, the definition of a structure from the Structural Engineer's point of view was given by Derek Seward (1998), *"as a system for transferring loads from one place to another."* The structural function of a building is therefore to transfer the loads of human beings, furniture, goods, wind, etc, including its own weight safely down to the foundations and subsequently into the ground. Hence, failure occurs when a building is not able to perform the above function. On the other hand, the purpose of structural design is to ensure that the building performs the above function effectively. It is imperative to note here that the structural function of a building mentioned above is directly related to other general function such as acting as cover from weather, burglars, etc.

The word "design" means different things to different people and professionals. For example, the Tailor sees design as the various plans and processes to ensure that clothes are made fitting to their users; the fine Artist sees design as any plan towards ensuring a beautiful image; the Electrical Engineer thinks of how current would flow safely through appropriate wires and cables to supply power at different points. These can go on and on. But our interest in this paper is the Structural Engineer's definition of design.

The aim of structural design according to the British Standard, BS8110: Structural Use of Concrete, Part 1 (1997), "is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an acceptable degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and resistance to the effects of misuse and fire."

Structural design refers the selection of materials, size, type and the suitable configuration that could carry loads in a safe and serviceable fashion. In general, it is the engineering of stationary objects such as bridges and buildings. The design of concrete structures such as slabs, beams, columns and foundations is generally done within the framework of codes giving specific requirements for materials, structural analysis, member proportioning, etc. These codes are often referred to as design codes. They are legal documents which represent the minimum requirements for obtaining safe structures and are written by responsible people with wide knowledge and experience of engineering.

Standard professional practice is adhered to, using relevant codes of practice. In Nigeria, the British standard codes are generally used or referred to. The entire process of design is achieved through a series of calculations. The purpose of these calculations according to R. Westbrook and D. Walker (1996) includes:

i. To show that the design is according to good structural practice, and where appropriate, comply with the current and relevant National Standards and Building Regulations.

ii. To demonstrate that the design is adequate in relation to stability, strength and serviceability requirements.

iii. To aid, instruct and assist the draughtsman preparing the general arrangement and detailed drawings.

iv. To provide a permanent record for future reference.

There are many structural design codes that are being used in different regions or countries across the globe, for example, Turkish standards (TS 500), Unified Arabic Code (UAC), Canadian Code (CSA-A23.3-94), Eurocode2 (EC), BS8110 and also American Code (ACI 318) among others. While some countries or regions have developed their own national or international codes, for example Eurocode used by countries across Europe and ACI318 in the USA, other countries (mostly developing) do not employ the use of specific design codes. Structural engineers in these countries often resort to consulting national codes from other countries. The structural design of most buildings worldwide is based on national or international codes of practice. These guide the engineer in the general appraisal of the overall structural scheme, detailed analysis and design. Codes of practice are basically guides drawn up by experienced engineers and a team of professionals, and they provide a framework for addressing issues of safety and serviceability in structural engineering design. In the African continent, national codes of practice have been primarily derived from the British standard BS8110-1997 and its predecessors. 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 Euro codes (ECs). The Euro codes are a new set of European structural design codes for building and civil engineering works. The Euro codes 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 Euro codes 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.

Concrete and reinforced concrete are the principal materials used in structural design and engineered construction (MacGregor, 1997; Wang and Salmon 1998; MacGinley and Choo 1990). They can be formed into various shapes and sizes (Mosley et al., 1999) which are only limited by the skills and technology in moulding. In Malaysia, the structural concrete design has been based on British Code BS 8110 (BSI, 1985) since its predecessor CP110 (BSI, 1972). Unfortunately, BS 8110 will be superseded by Eurocode 2 (CEN, 1992) by the year 2008 in the United Kingdom, with the accompanying document containing the Nationally Determined Parameters (NDP's). Therefore, the structural designers in Malaysia may have to implement Eurocode 2 gradually in the structural concrete design after the withdrawal of BS 8110 (Omar et al., 2001).

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.

Nowadays, Euro codes 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. The purpose of this work is to find out significant differences (if any) between the BS 8110 and the Eurocode2, taking the design of a reinforced concrete beam as a case study of the comparison. Structural design refers to the selection of materials, size, type and the suitable configuration that could carry loads in a safe and serviceable fashion. 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. Design is involved at all elements of the building such as slab, beam, column, foundation, roof etc. In the design of reinforced concrete Beams, considerations are made for bending moment, shear force, cracking and area of reinforcement.

Usually in Nigeria, the design of structures is guided by the use of British Standard, (BS 8110). 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). In this study, excel spreadsheet is used to compare the results of the two design codes. The algorithm is simple and it caters for the errors that may be incurred in the manual design.

Engineering is all about the design and construction of safe structures which meets all quality requirements at lowest possible cost. Even if a structure is safe, it may not necessarily be regarded as a successful engineering structure unless it is also economical i.e. in engineering safety and economy goes hand in hand. Comparative studies of these differences helps in better understanding and interpretation of these codes. It will also help the structural engineer to choose which code is more economical for the design of an intended structure

• Liew (2009) "British standard (BS 811 O) 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 811 O. 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.

• Alnuaimi and Patel (2013) "Serviceability, limit state, bar anchorage and lap lengths in ACI318:08 and BS8110:97: A comparative study" This paper presents a comparative calculation study of the deflection, bar anchorage, lap lengths and control of crack width of reinforced concrete beams using the BS 811 O and ACI318 codes. The deflections 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.

2.2 Comparison between Structural Euro Codes 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 Euro codes. 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 Europeen de Normalisation), the European Standards Organisation.

BS8110/EC2 flow chart

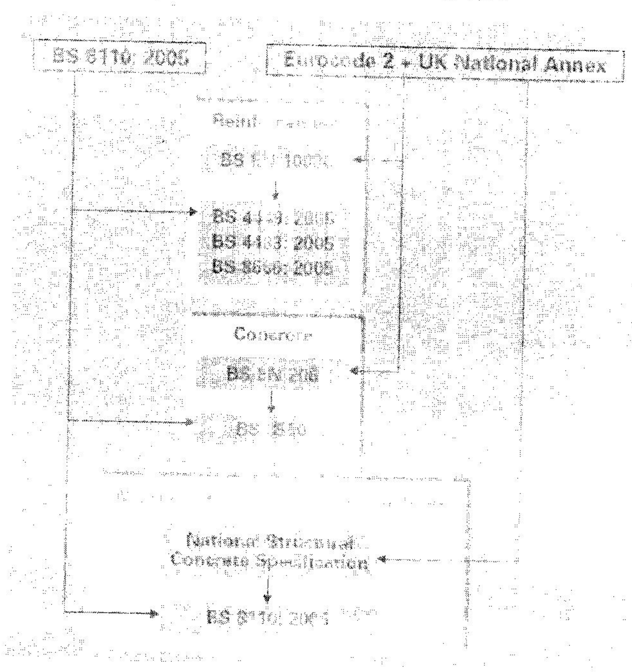


Fig 2.1 bs8110/EC2 flow chat

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 Euro codes 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 Euro codes to CEN, and EFTA secretariat agreed to support the CEN work. CEN Technical

Committee CEN/TC250 is responsible for all Structural Euro codes. The proposed Euro codes currently under preparation are as follows:

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

EN 1992 - EC 2: Design of Concrete Structures

EN 1993 - EC 3: Design of Steel Structures

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

EN 1995 - EC 5: Design of Timber Structures

EN 1996 - EC 6: Design of Masonry Structures

EN 1997 - EC 7: Geotechnical Design

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

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

The steps brought about by the developments of Euro codes 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 (Euro codes), and some introductory aspects particularly on design principles and the differences brought about by the harmonized codes.

2.3 Structural Euro Codes

Structural Euro codes are a set of European design standards which introduced a common technical language and a common technical culture in structural design. This facilitated the creation of an effective Internal Market within the European Union, by removing potential barriers to trade that could exist when Member States have different national design standards. The use of Structural Euro codes is expected to contribute towards widened and improved competitiveness of the European construction industry.

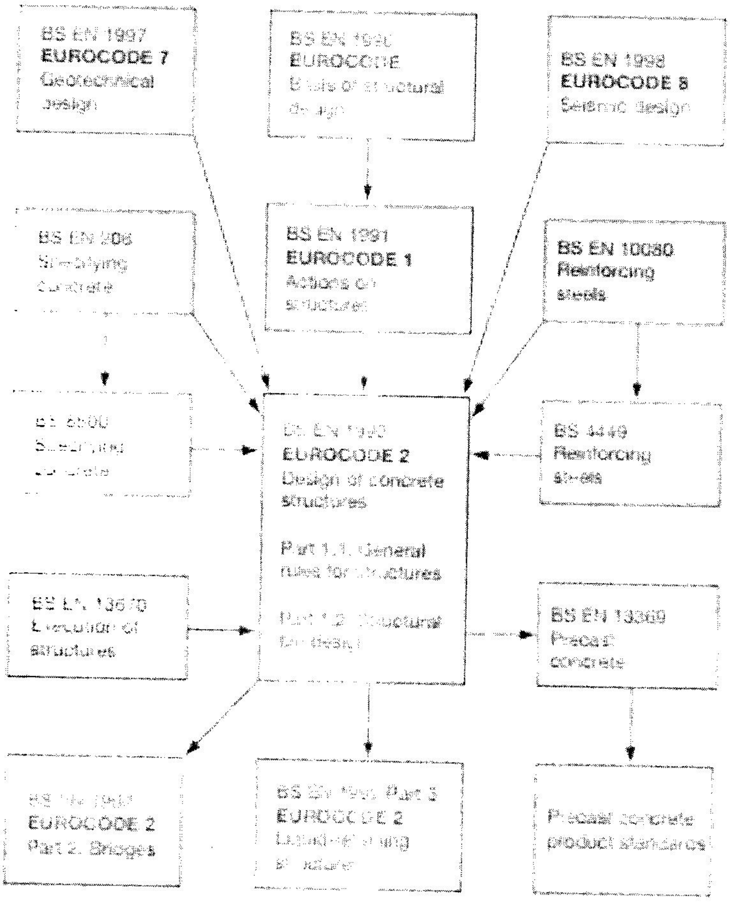


Fig 2.2 Relationship between euro code 2 and other European codes

2.3.1 British Standard

British Standards are the standards produced by BSI Group which is incorporated under a Royal Charter (and which is formally designated as the National Standards Body (NSB) for the UK). The BSI Group produces British Standards under the authority of the Charter, which lays down as one of the BSI's objectives to set up standards of quality for goods and services, and prepare and promote the general adoption of British Standards and schedules in connection therewith and from time to time to revise, alter and amend

such standards and schedules as experience and circumstances require. BSI Group began in 1901 as the Engineering Standards Committee, to
standardise the number and type of steel sections, in order to make British manufacturers more efficient and competitive. BSI Group currently has over 27,000 active standards. Products are commonly specified as meeting a particular British Standard, and in general this can be done without any certification or independent testing. The standard simply provides a shorthand way of claiming that certain specifications are met, while encouraging manufacturers to adhere to a common method for such a specification.

Table 2.1: Parts of BS 8110

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 is a British Standard for the design and construction of reinforced and prestressed concrete structures. It 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). In 2010, BS 8110 was superseded by EN 1992 (Eurocode 2) although parts of the standard have been retained in the National Annex of the Eurocode.

2.3.2 Eurocode 2 - Design for Concrete Structures

Eurocode 2 or in short 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, known as European Prestandards (ENV) are listed as follows:

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

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

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

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

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

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

ENV 1992-2: 1996 Concrete bridges

ENV 1992-3: 1998 Concrete foundations

ENV 1992-4: 1998 Liquid retaining and containment structures.

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 [2--3]. 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 I.

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 type

Table 2.2: Parts of Euro code

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

Typical of any Euro codes, the Principles stated in EC2 does not allow for alternatives and all designs should comply with them. Application Rules allows for alternative methods provided that it can be demonstrated that they comply with the principles. As stated earlier some of the terms used in Euro codes 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.

In BS 8110 *f*cu should not be taken as greater than 40 N/mm2. There is no limit on the concrete strength in EC2. The values for γm are 1.25 and 1.5 in BS 8110 and EC2, respectively. EC2 provides alternative in designing the shear links. It allows the use of the method as in BS 8110 which is based on 45° strut. EC2 also allows the use of variable strut inclination method leading to increased consumption in the requirement of shear links.

The differences between BS 8110 and EC2 could also be seen in the serviceability limit state design. For example, EC2 includes the provision to check the stress level in reinforced concrete, whilst this is not required by BS 8110. In contrast to BS 8110 which uses the characteristic loads for serviceability check, EC2 requires three levels of loading, depending on the nature of the particular check being carried out.

From the brief discussions above, certainly there need to be a clear understanding on the background of the new codes, particularly the differences, before a designer could use it effectively in practice

Principles of Eurocode 2

EN 1992 Eurocode 2: Design of concrete structures is of fundamental importance to civil engineers given the predominance of concrete in civil engineering construction.

Ultimately Eurocode 2 will become the one design code for all concrete structures. It will bring reinforced concrete design up-todate. The general basis for design of structures in reinforced and prestressed concrete made with normal and lightweight concrete together with specific rules given are mainly aimed at building structures as explained in the first section of the first part of Eurocode 2. The new code will thus be a more comprehensive document than its predecessor.

The scope of design in Euro codes is similar to many current national codes in Europe. The main chapters of the code deal with:

Basis of design

- Materials
- Durability
- Structural analysis
- Ultimate limit state
- Serviceability limit state
- Detailing of reinforcement
- Detailing of members
- Additional rules for precast elements and structures
- Lightweight aggregate concrete
- Plain concrete

It has been known that the design process will not change as a result of using Eurocode 2. But there is a change of emphasis as Eurocode 2 is laid out to deal with phenomena such as *flexure,* shear and deflection rather than beams, slabs, column and foundation which are dealt with in BS8110

Table 2.3: Relationship between Eurocode and British Standard

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 DESIGN METHODS

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 processes 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 judgment 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. For instance, after the infamous disaster at the Ronan Point block of flats in Newham, London, when a gas explosion caused a serious partial collapse, research work was carried out, and codes of practice were amended so that such structures could survive a gas explosion, with damage being confined to one level.

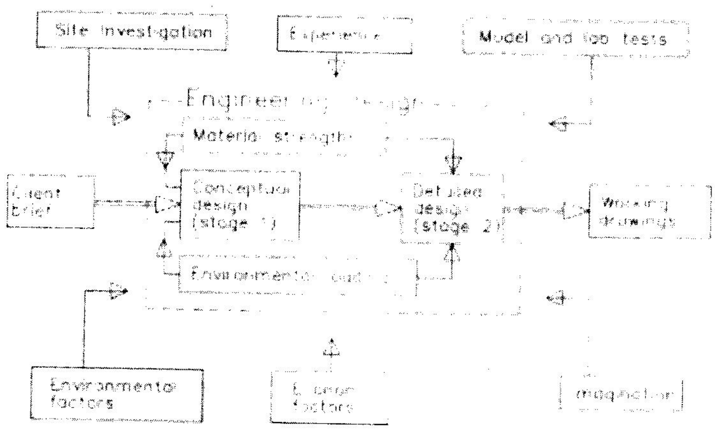


Fig.2.3 Inputs into the design process.

In design there exist within the structure a number of critical points (e.g. beam mid-spans) where the design process is concentrated. The normal distribution curve on the left of Fig. 2.2 represents the actual maximum material stresses at these critical points due to the loading. Because loading varies according to occupancy and environmental conditions, and because design is an imperfect process, the material stresses will vary about a modal value the peak of the curve. Similarly the normal distribution curve on the right represents material strengths at these critical points, which are also not constant due to the variability of manufacturing conditions.

The overlap between the two curves represents a possibility that failure may take place at one of the critical points, as stress due to loading exceeds the strength of the material. In order for the structure to be safe the overlapping area must be kept to a minimum. 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.

2.4.1 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.

2.4.2 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.

2.4.3 LIMIT STATE DESIGN

Originally formulated in the former Soviet Union in the 1930s and developed in Europe in the 1960s, limit state design can perhaps be seen as a compromise between the permissible and load factor methods. It is in fact a more comprehensive approach which takes into account both methods in appropriate ways. Most modern structural codes of practice are now based on the limit state 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 Eurocode 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.

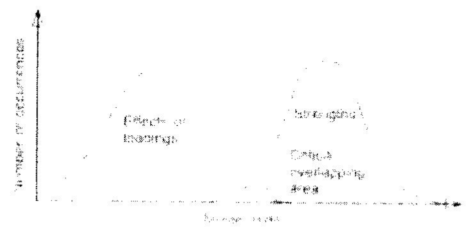


Fig. 2.4 Relationship between stress and strength.

In the absence of a national design code, the structural engineers in Nigeria use the BS811 O, Euro code2, ACI318 and quite a number of structural design codes to design structures. They find these codes useful for complying with the legal stipulations there. However, designers and project owners frequently compare the stipulations in these codes seeking points of similarities and differences. Although the main purpose of these design codes is to provide guidelines for the design of safe and economic structures, the principles, procedures and assumptions employed to achieve this may differ from one code to another. Also studies have shown that some codes are more economical than others

2.5 Design load

Design load divided into three parts:

i. Dead load (ga) is not changed much from value which is estimated. Among dead load feature is self-weight slab and finishing weight.

ii. Imposed load (q) was unsteady load and will changes depended on structure use.

iii. Wind loading *(Wk)* which depended location, form, dimension building, and wind velocity at that area.

2.5.1 Dead and Imposed Loads

In the beam design, commonly used parameters such as imposed loads and concrete grade were made uniform for both BS8110 and Eurocode2. Varied parameters were mainly those based on theories of the codes such as the equations governing flexure at the ultimate limit state and shear.

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 24KN/m3, while that for Eurocode2 is 25KN/m3. 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.

In this study, prismatic beam cross-sections were adopted because both Eurocode2 and

BS8110 show no substantial difference between preliminary span/effective depth ratios for beams.

Table 2.4: BS8110 and Eurocode2 basic span/effective depth ratios for rectangular beams

Support conditions	BS8110-1997[3]	Euro code 2[12]
cantilever	7	7
Simply supported	20	18
continuous	26	25
End span of continuous beam	-	23

2.5.2 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.

$$w = 1.4gk + 1.6qk \qquad (1)$$

$$n = 1.35gk + 1.5qk \qquad (2)$$

Where:

gk and qk 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.

2.6 The Design Process:

The design process of structural planning and design requires not only imagination and conceptual thinking but also sound knowledge of science of structural engineering besides the knowledge of practical aspects, such as recent design codes, bye laws, backed up by sample experience, initiation and judgment. The purpose of standards is to ensure and enhance the safety, keeping careful balance between economy and safety.

The process of design commences with planning of the structure. Primarily to meet its functional requirements. Initially, the requirements proposed by the client are taken into consideration. They may be vague, ambiguous or even unacceptable from engineering point of view because he is not aware of the various implications involved in the process of planning and design, about the limitation and intricacies of structural science.

It is emphasized that any structure to be constructed must satisfy the need efficiently for which it is intended and shall be durable for its desired life span.

Thus, the design of any structure is categorized into the following two main types:-

1) Functional design

2) Structural design.

The structure to be constructed should be primarily serve the basic purpose for which it is to be used and must have a pleasing look.

Euro code 2 is not widely different from bs8110 in terms of the design approach.it gives similar answers but is less prescriptive and more extensive than bs8110. It gives designers the opportunity to derive benefit from the consideration advances in concrete technology over recent years

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 1 and Table 2show the difference:

Table 2.5 Summary of the differences between BS 8110 and Eurocode 2

<table>
<tr><th>S/N</th><th>Parameters</th><th>BS 8110</th><th>Eurocode 2(CEN, 1992)</th></tr>
<tr><td>1.</td><td>Concrete strength</td><td>Cube strength, f_{cu}
$f_{cu} \approx f_{ck}/0.8$</td><td>Cylinder strength, f_{ck}
$f_{ck} \approx 0.8f_{cu}$</td></tr>
<tr><td>2.</td><td>Partial safety factor, γ_m</td><td>For concrete in bending = 1.5
For steel = 1.15</td><td>For concrete:
Fundamental = 1.5
Accidental = 1.3
For steel:
Fundamental = 1.5
Accidental = 1.0</td></tr>
<tr><td>3.</td><td>Yield strength of high yield steel</td><td>$f_y = 460\ MPa$
f_y/γ_m
$= 400\ MPa$</td><td>$f_{yk} = 460\ MPa$
$f_{yk}/\gamma_m = 400\ MPa$</td></tr>
<tr><td>4.</td><td>Yield strength of high mild steel</td><td>$f_y = 250\ MPa$
f_y/γ_m
$= 217\ MPa$</td><td>$f_{yk} = 250\ MPa$
$f_{yk}/\gamma_m = 217\ MPa$</td></tr>
<tr><td>5.</td><td>Ultimate strain of concrete, ε_{cu}</td><td>0.0035 for flexure</td><td>0.002 for axial load
0.0035 for flexure</td></tr>
<tr><td>6.</td><td>Maximum allowable neutral axis depth, x</td><td>0.5d (no redistribution)</td><td>$0.45d$ for $f_{ck} \leq 35\ MPa$
$0.35d$ for $f_{ck} > 35\ MPa$
$0.35d\ for\ plastic\ analysis$</td></tr>
<tr><td>7.</td><td>Concrete compression zone depth (simplified rectangular stress block)</td><td>$0.9x$</td><td>$0.8x$</td></tr>
<tr><td>8.</td><td>Ultimate moment of resistance, M_U</td><td>M_u
$= 0.156f_{cu}bd^2$</td><td>For $f_{ck} \leq 35$ MPa: $M_u =$
$0.167f_{ck}bd^2$
For $f_{ck} > 35$ MPa: $M_u =$
$0.128f_{ck}bd^2$</td></tr>
<tr><td>9</td><td>Shear</td><td>$V_R = v_c b_v d$</td><td>$V_{Rd1} = \tau_c b_w d$</td></tr>
<tr><td>10</td><td>Column</td><td></td><td></td></tr>
<tr><td></td><td>- Short
- Slender</td><td></td><td></td></tr>
<tr><td></td><td>Equations</td><td></td><td></td></tr>
<tr><td>11</td><td>Links</td><td>Longitudinal spacing $\leq$ 0.75d or 300mm
Transverse spacing $\leq$ d</td><td>Longitudinal spacing<table><tr><th>V_{Rd2}</th><th>Lesser of</th></tr><tr><td>0 -0.2</td><td>0.8d or 300mm</td></tr><tr><td>0.20-0.67</td><td>0.6d or 300mm</td></tr><tr><td>0.67 -1.0</td><td>0.3d or 200mm</td></tr></table></td></tr>
</table>

Euro code 2 is not widely different from bs8110 in terms of the design approach.it gives similar answers but is less prescriptive and more extensive than bs8110. It gives designers the opportunity to derive benefit from the consideration advances in concrete technology over recent years

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 1 and Table 2show the difference:

			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

Table 2.6: Differences Terminology between BS8110 and EC2

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

Table 2.7: Differences symbol between BS 8110 and EC2

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).	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.

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, while BS 8110 using 28 days concrete cube strength, . By estimation, the strength of the cylinder is 80% of the cube strength.

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).

Partial safety factor of materials

Same as BS 8110, EC 2 also use the factor of safety for concrete material 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 $500N/mm^2$.

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 which was prepared using Revit Architecture is as shown in the figures below. The ground floor consists of garage on one side and archive area on the other side. The first floor is the proposed Undergraduate library and consists of reading area and the bookshelf area while the second floor is the proposed Postgraduate library and consists of reading area and bookshelf area.

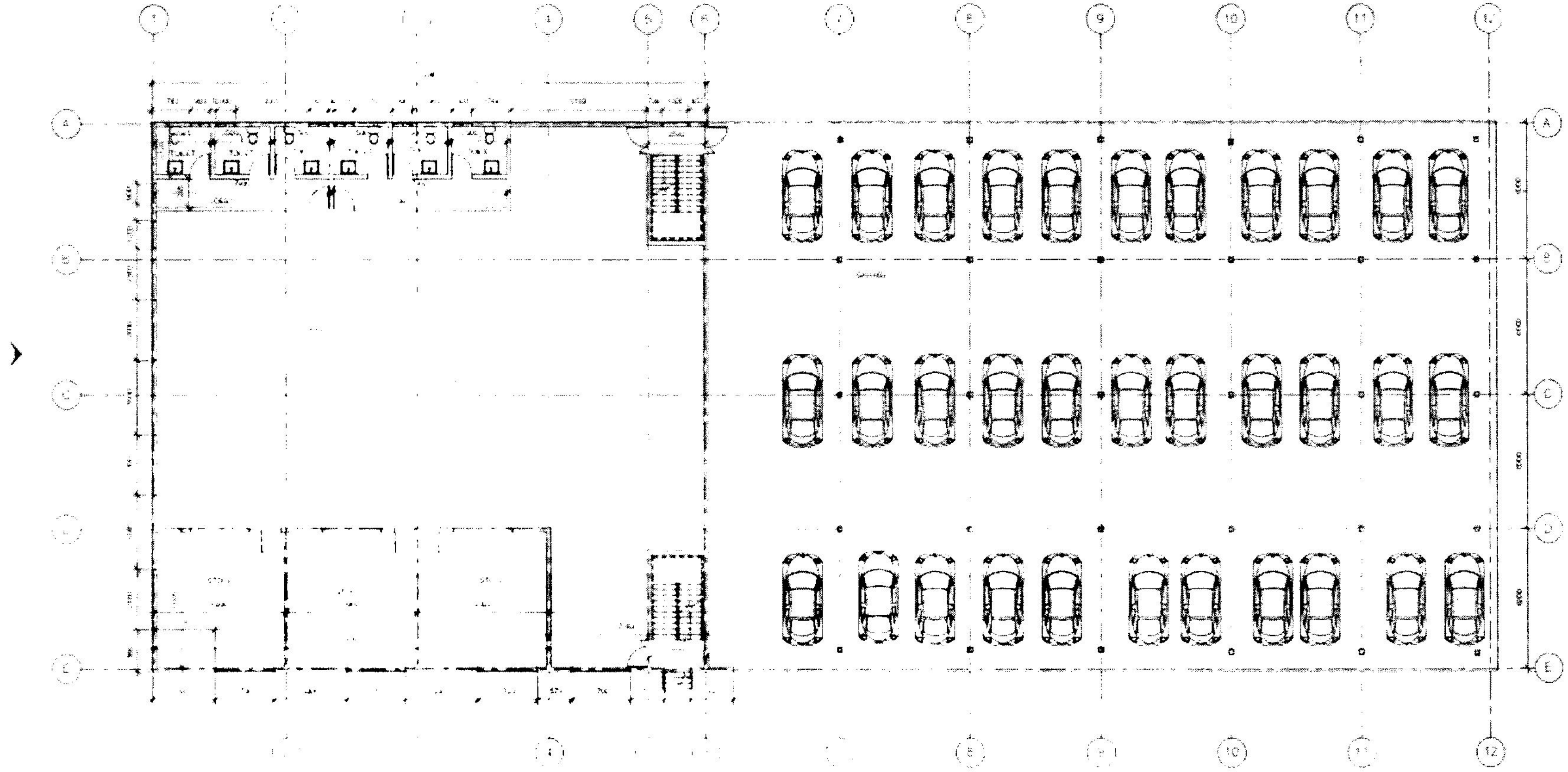


Plate 3.1: Ground floor plan

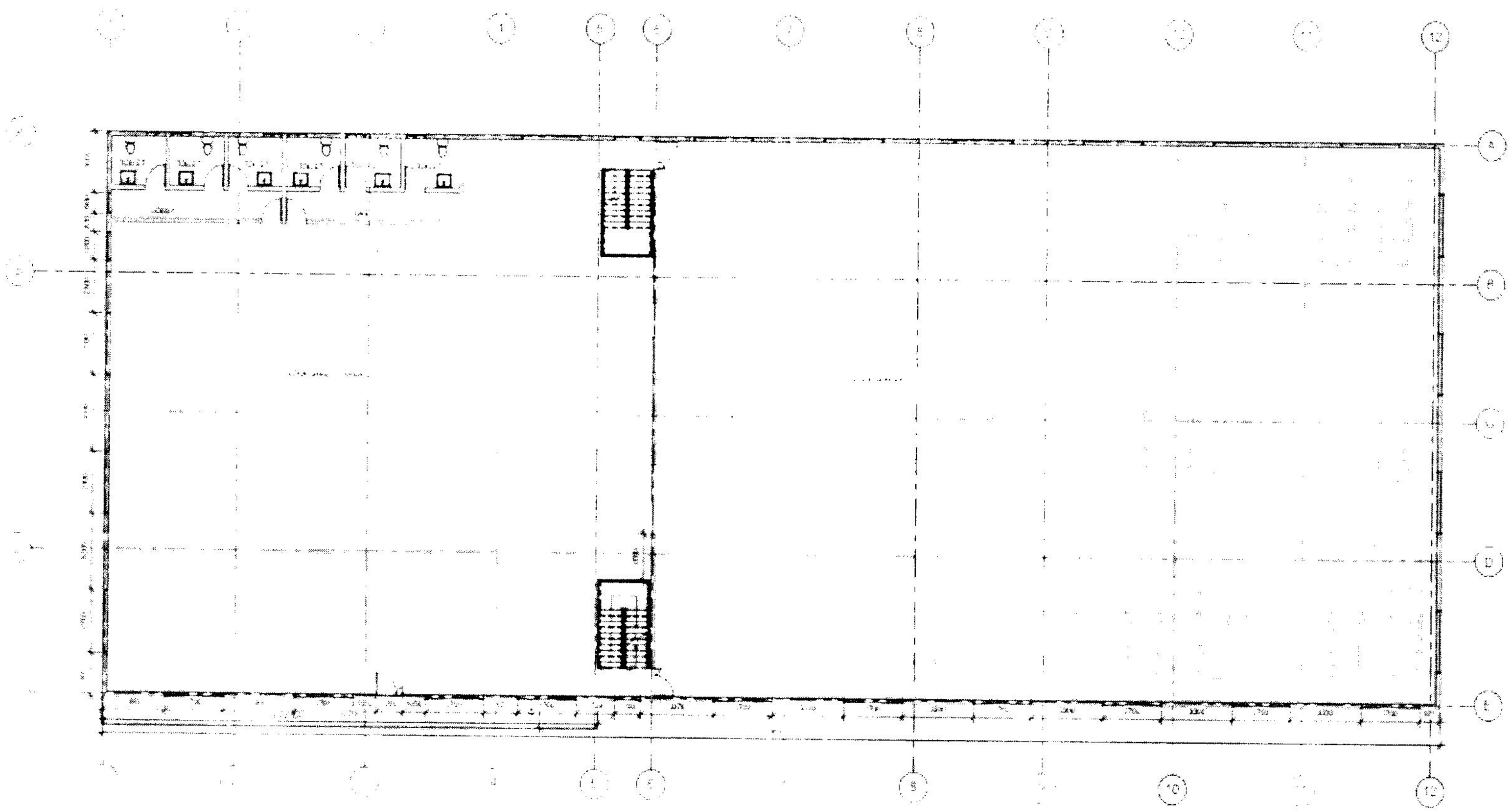
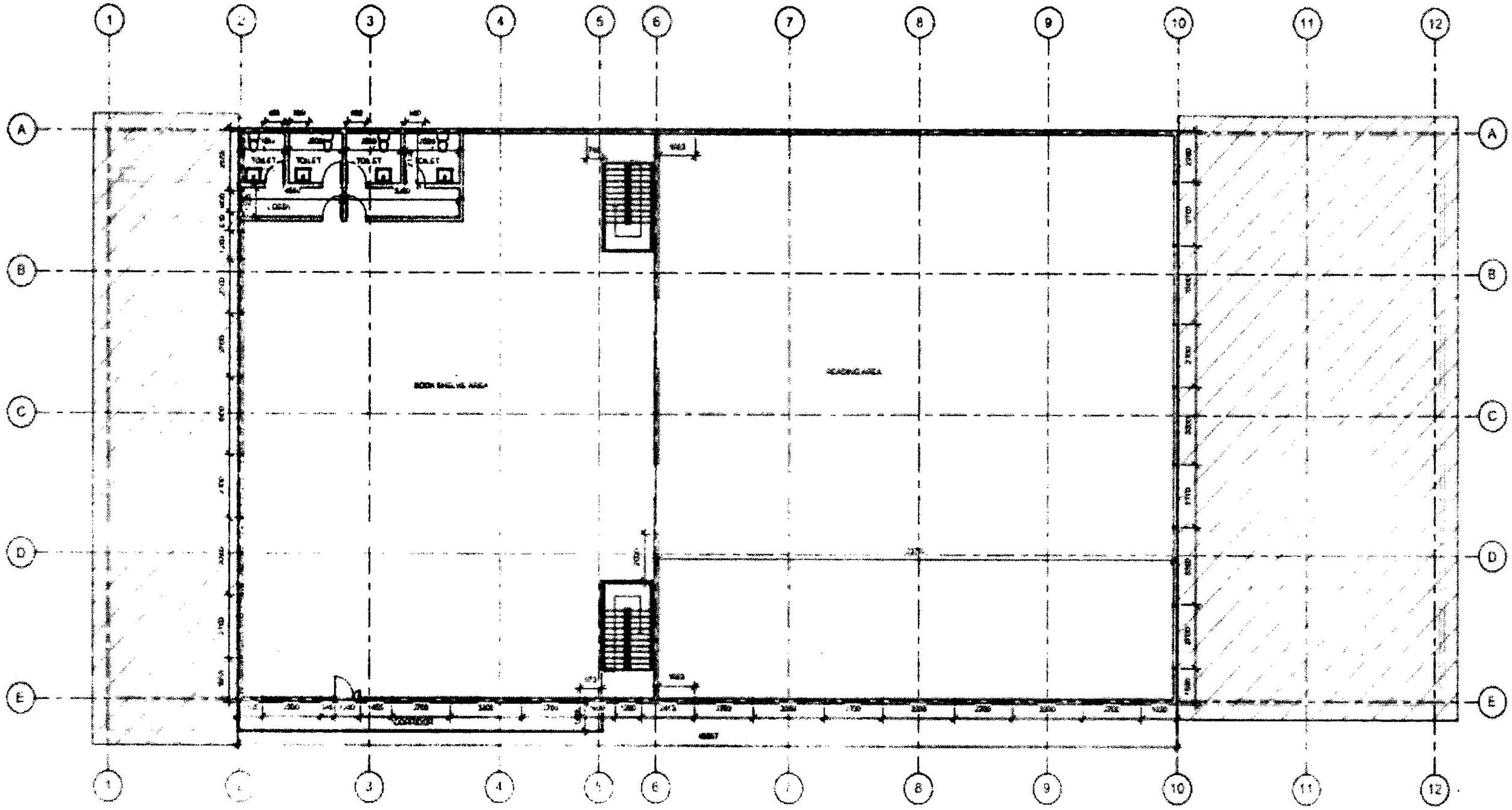


Plate 3.2: First floor plan



SECOND FLOOR PLAN

Plate 3.3: Second floor plan

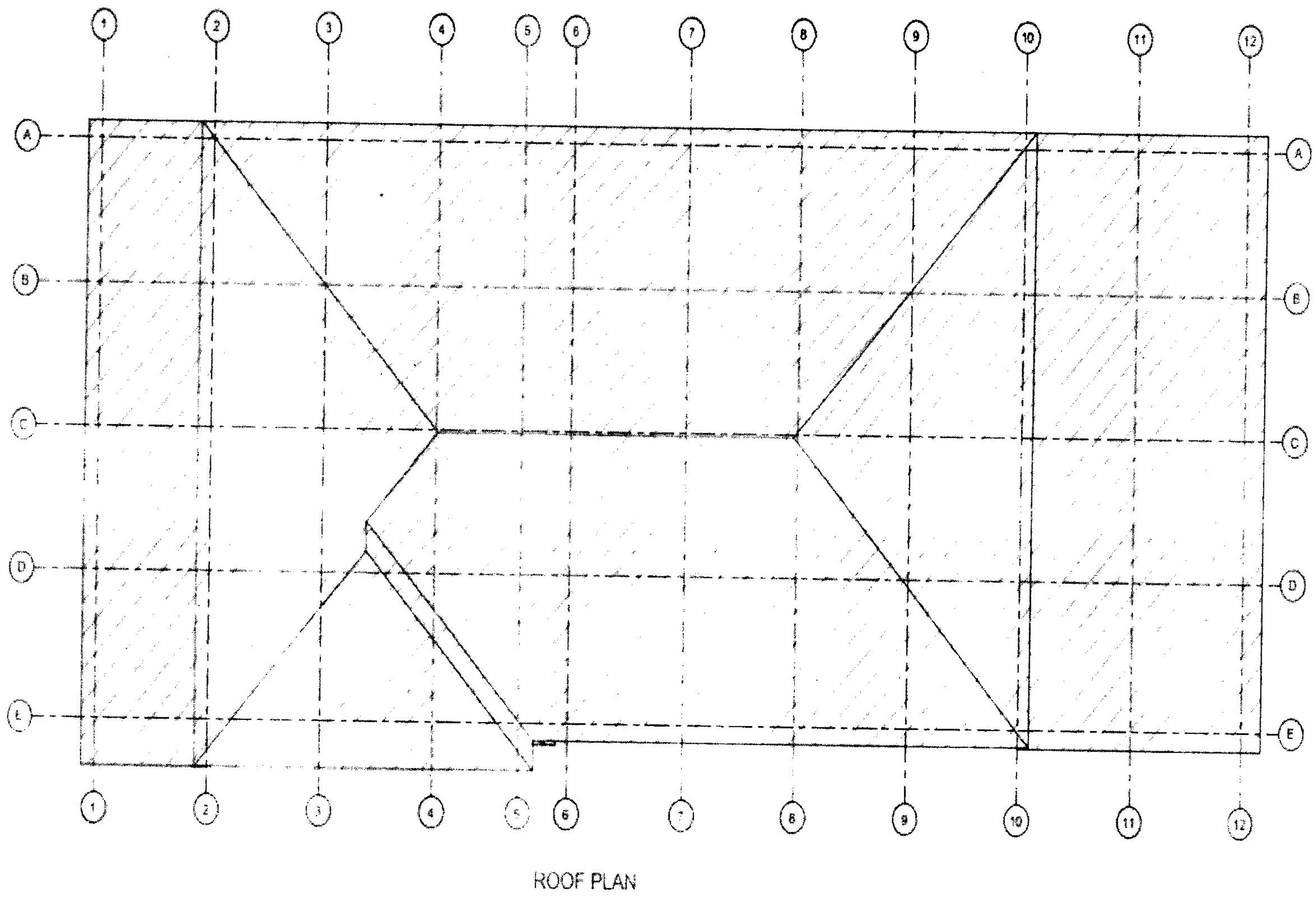


Plate 3.4: Roof plan

Plate 3.5: Approach view

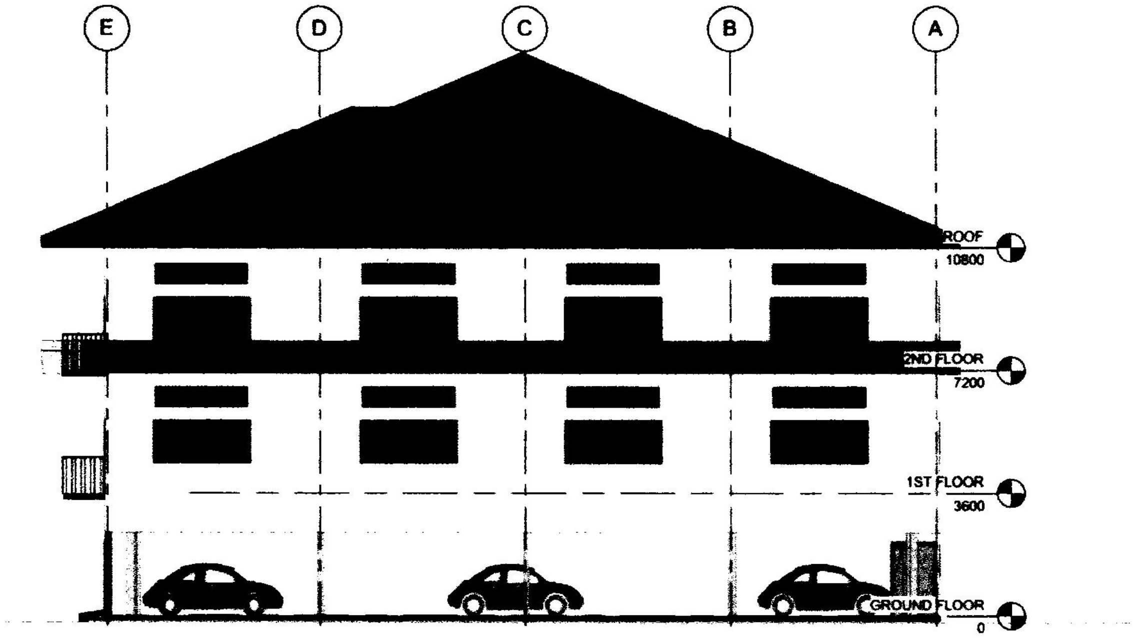


Plate 3.6: Right side view

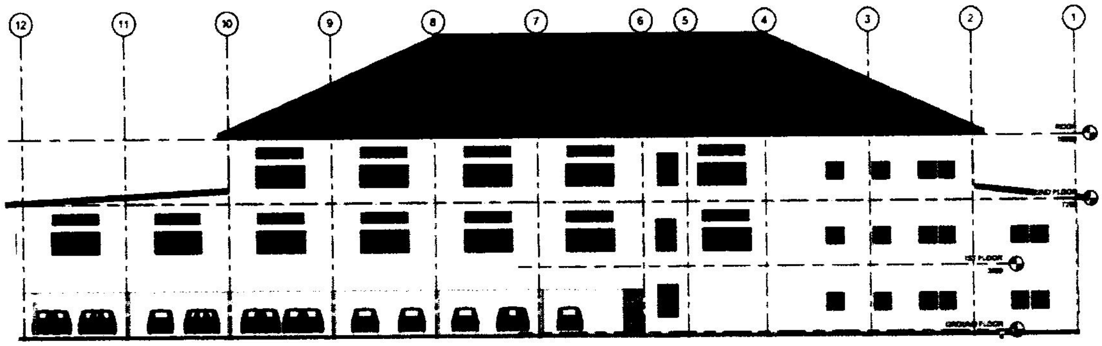


Plate 3.7: Rear view

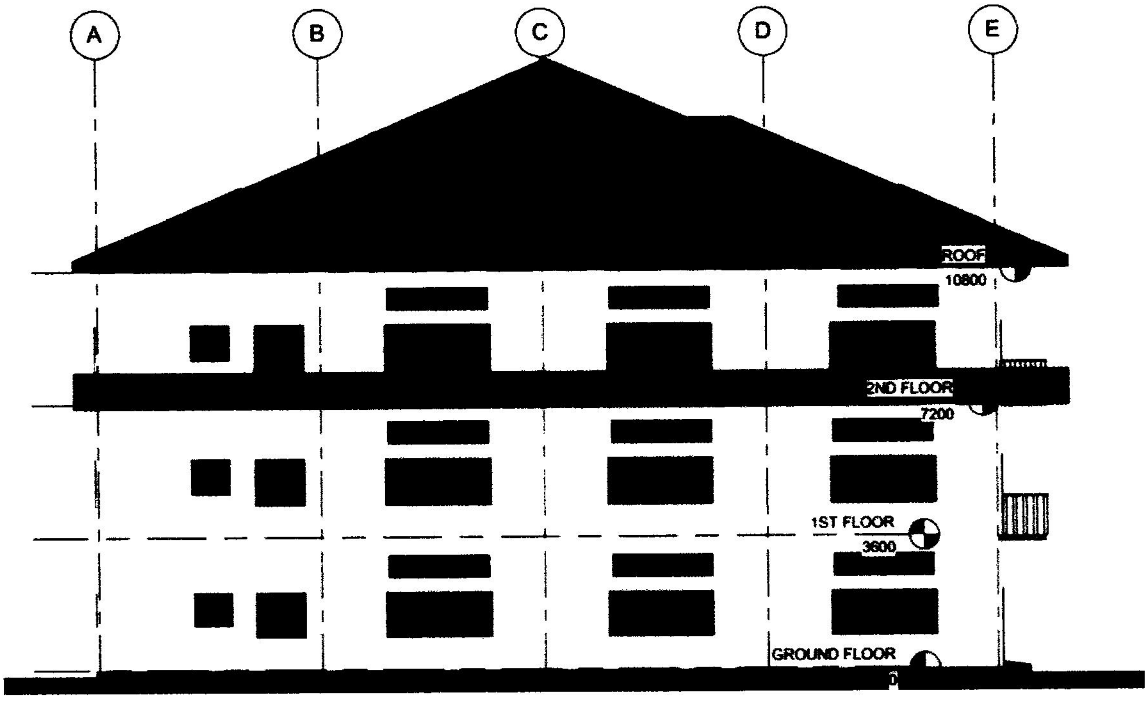


Plate 3.8: Left side view

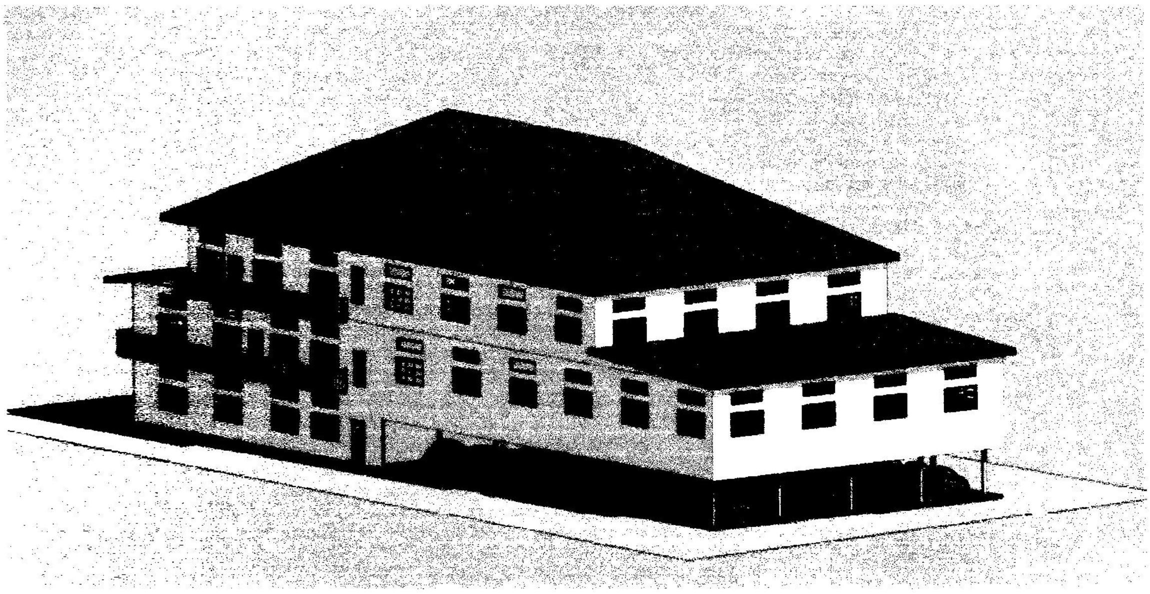


Plate 3.9: 3D view

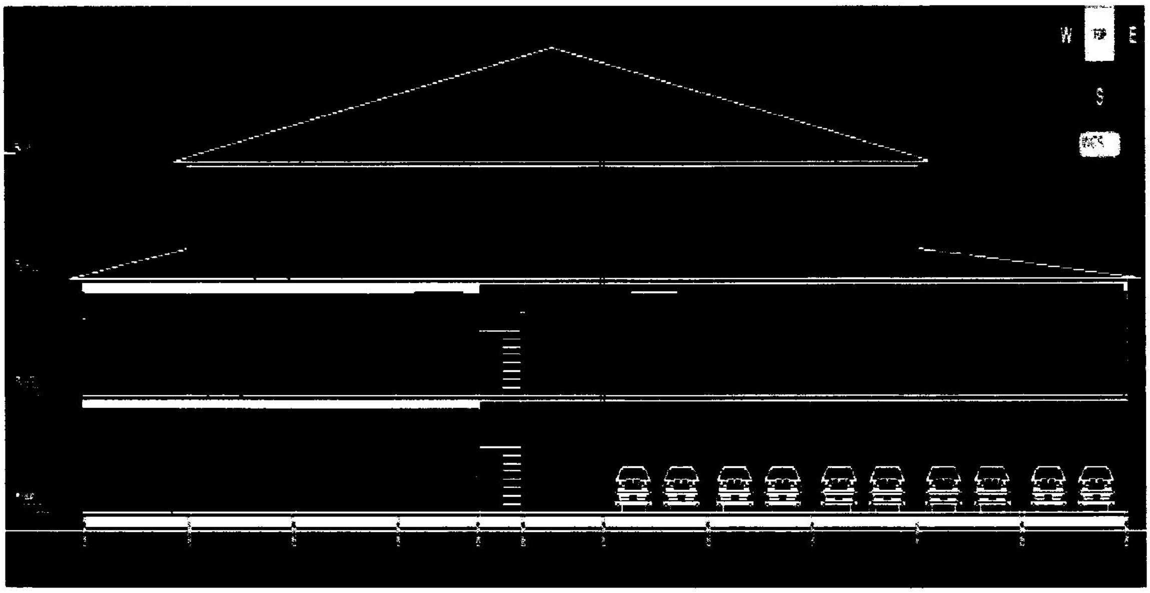


Plate 3.10: Longitudinal section

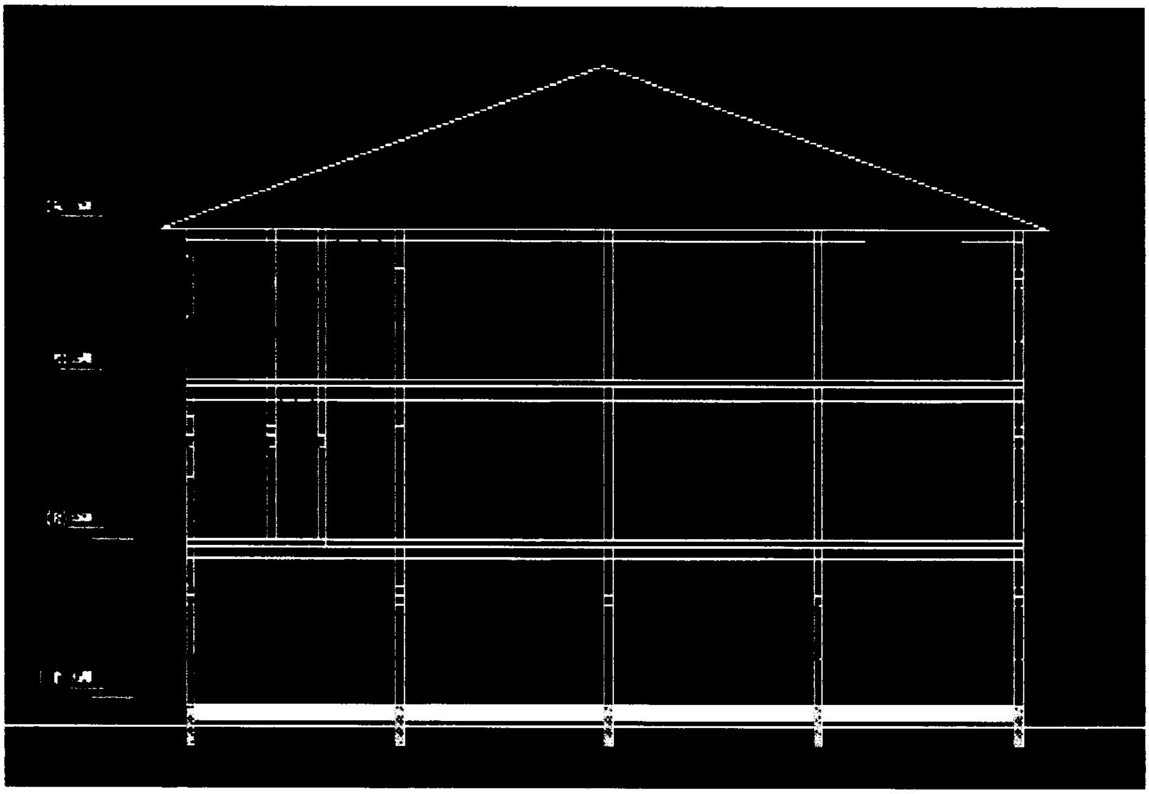


Plate 3.11: Transverse section

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:

Fitness for purpose

Safety and reliability

Durability

Good value for money

External appearance

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 $1.2kN/m^2$, while partition is taken as $4.0kN/m^2$.

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 $4.0kN/m^2$

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 (fk), strength of steel (gk) 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),

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), and

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

Step 11: Topping reinforcement

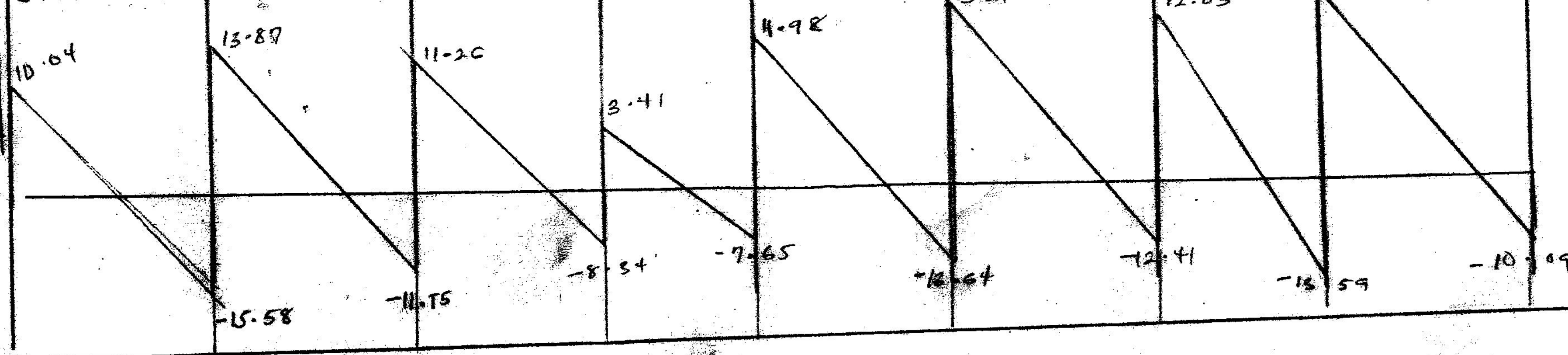
Minimum steel shall be provided in the topping = 0.13%bh

REFERENCE | CALCULATION | OUTPUT

Roof Beam

4.27kN/m

6	6	4.59	2.59	C	C	G	C
s/w of the beam	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
0.225×0.3×24× 1.4 = 2.268kN/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
2kN/m	2kN/m	2kN/m	2kN/m	2kN/m	2kN/m	2kN/m	2kN/m
Total load	Total load	Total load	Total load	Total load	Total load	Total load	Total load
2+2.27= 4.27kN	4.27kN/m	4.27kN/m	4.27kN/m	4.27kN/m	4.27kN/m	4.27kN/m	4.27kN/m



REFERENCE	CALCULATION	OUTPUT
	DESIGN OF SUPPORT REINFORCEMENT	
	Mmax @ support = 16.65kNm.	
	$d = 300 - 30 - \frac{25}{2} - 10$	
	$= 247.5mm$	
	$K = \frac{M}{Fcubd^2} = \frac{16.65 \times 10^6}{25 \times 225 \times 247.5^2} = 0.05$	
	$la = 0.5 + \sqrt{0.25 - K/0.9} = 0.5 + \sqrt{0.25 - \frac{0.05}{0.9}} = 0.94$	
	Use $la = 0.94$ since $la < 0.95$	
	$Z = lad = 0.94 \times 247.5mm = 232.65$	
	$As\,req = \frac{M}{0.95 fyZ} = \frac{16.65 \times 10^6}{0.95 \times 410 \times 232.65} = 183.7mm^2$	
Table 3.25	$As_{min} = 0.13\% bh = 0.0013 \times 225 \times 300$	
	$= 87.75mm^2$	
	Since $As_{min} < As\,req$: Provide	
	2Y-16mm As provid = $402mm^2$	
	As prov > As min; provision is OK.	
	DESIGN OF SPAN REINFORCEMENT	
	Mmax @ span = 11.93kNm d = 247.5mm	
	$K = \frac{M}{Fcubd^2} = \frac{11.93 \times 10^6}{25 \times 225 \times 247.5^2} = 0.03$	
	$la = 0.5 + \sqrt{0.25 - \frac{K}{0.9}} = 0.96$	
	Since 0.96 > 0.95, use la = 0.95	
	$Z = lad = 0.95 \times 247.5mm = 235.13mm$	
	$As\,req = \frac{M}{0.95 fyZ} = \frac{11.93 \times 10^6}{0.95 \times 410 \times 235.13}$	
	$= 130.26mm^2$	
Table 3.25	X	

REFERENCE	CALCULATION	OUTPUT

Since $As_{min} < As_{req}$, Provide.

Provide 2T-16mm - $As_{prov} = 402mm^2$

$As_{prov} > As_{min}$; Provision is OK.

DESIGN OF SHEAR REINFORCEMENT

$V_{max} = 15.58KN$

$v = \frac{V}{bd} = \frac{15.8 \times 10^3}{225 \times 247.5} = 0.28$

$$v_c = \frac{0.79 \times (400/d)^{1/4} (100As/bd)^{1/3}}{\lambda_m}$$

$$\frac{400}{d} = \frac{400}{247.5} = 1.6$$

$$\frac{100As}{bd} = \frac{100 \times 402}{225 \times 247.5} = 0.72$$

$$v_c = \frac{0.79 \times (0.72)^{1/3} (1.6)^{1/4}}{1.25} = 0.64$$

Since $v < 0.5v_c$

$S = 0.75d = 0.75 \times 247.5 = 185.6mm$

Provide 2-leg-Y10 @ 200mm c/c

DEFLECTION CHECK (At Span)

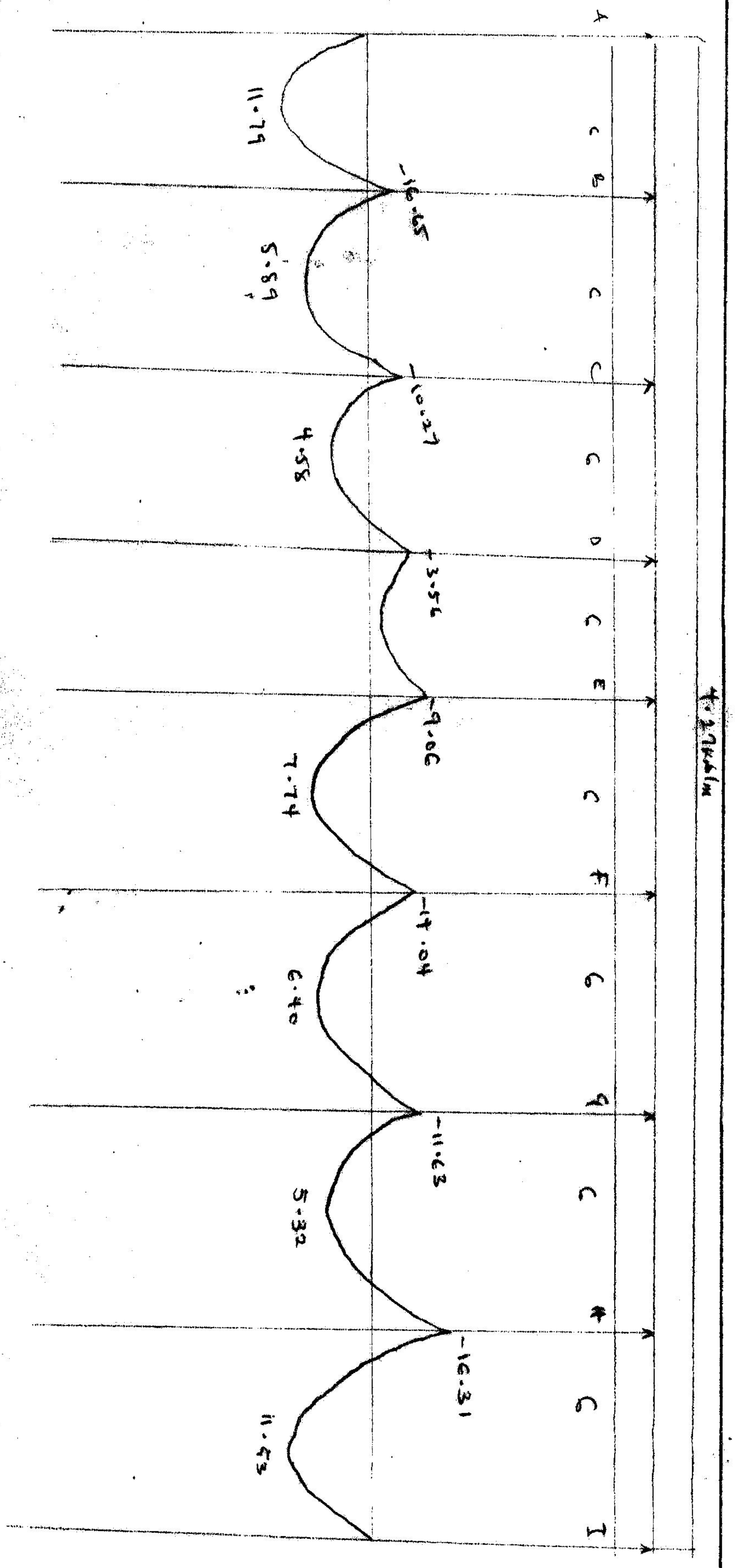
Service stress, $F_s = \frac{2}{3} \times f_y \times \frac{As_{req}}{As_{prov}} = \frac{2}{3} \times 410 \times \frac{130.26}{402}$

$= 88.57 N/mm^2$

$$M.F = 0.55 + \frac{477 - 88.57}{120\left(0.9 + \frac{11.93 \times 10^6}{225 \times 247.5^2}\right)} = 2.38$$

CALCULATION	OUTPUT
Actual span $= \frac{lx}{d} = \frac{6000}{247.5} = 24$	
Actual span < limiting span $\therefore$ Deflection is OK.	

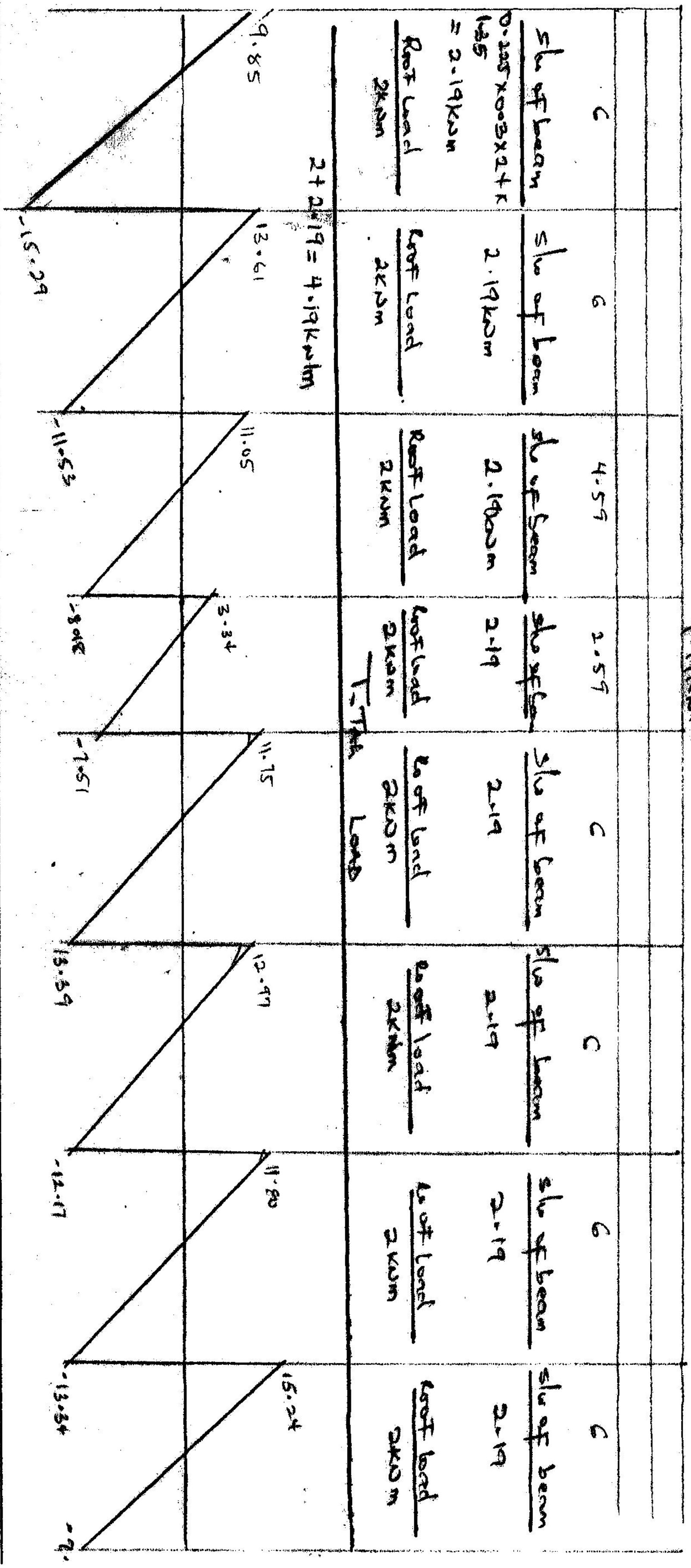
REFERENCE	CALCULATION	OUTPUT



ROOF BEAM DESIGN TO EUROCODE.

REFERENCES.	CALCULATION	OUTPUT

Roof Beam



Reference	Calculation	Output

Design of support Reinforcement

M_{max} @ support = 16.33kNm $d = 300 - 30 - 25/2 - 10$
$= 247.5mm$

$$K = \frac{M}{F_{ck}bd^2} = \frac{16.33 \times 10^6}{25 \times 225 \times 247.5^2} = 0.05$$

$$la = 0.5 + \sqrt{0.25 - K/1.134} = 0.5 + \sqrt{0.25 - \frac{0.05}{1.134}} = 0.95$$

Use $la = 0.95$

$z = lad = 0.95 \times 247.5 = 235.13$

$$As_{req} = \frac{M}{0.87F_{yk}z} = \frac{16.33 \times 10^6}{87 \times 410 \times 235.13} = 194.7mm^2$$

$As_{min} = 0.0013 \times 225 \times 300 = 87.75m^2$

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 = 11.70kNm $d = 247.5mm$

$$K = \frac{M}{F_{ck}bd^2} = \frac{11.70 \times 10^6}{25 \times 225 \times 247.5^2} = 0.03$$

$$la = 0.5 + \sqrt{0.25 - \frac{K}{1.134}} = 0.5 + \sqrt{0.25 - \frac{0.03}{1.134}}$$
$$= 0.97$$

Use $la = 0.95$ since $0.97 > 0.95$

$z = lad = 0.95 \times 247.5 = 235.13.$

$$As_{req} = \frac{M}{0.87F_{yk}z} = \frac{11.70 \times 10^6}{0.87 \times 410 \times 235.13} = 139.5mm^2$$

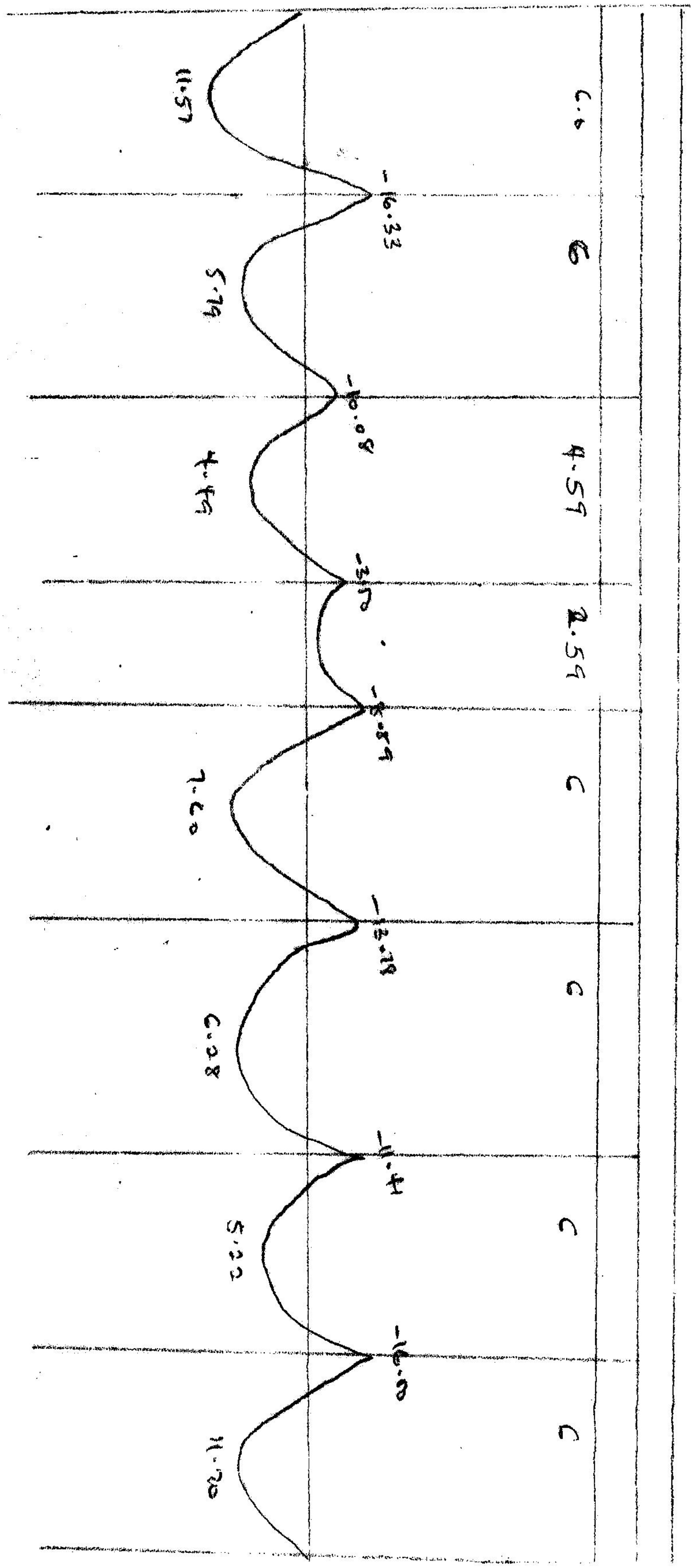
$As_{min} = 0.13\% = 0.0013 \times 225 \times 300 = 87.75mm^2$

Provide 2Y-16mm As_{prov} = 402mm^2

$As_{prov} > As_{min}$: Provision is o.k

RS-EC

REFERENCE	CALCULATION	OUTPUT



REFERENCES	CALCULATION	OUTPUT

DESIGN OF SHEAR REINFORCEMENT

$V_{max} = 15.29kN$ $\qquad V_{RD} = 0.3 V F_{cu} b d$

$= 0.3 \times 0.5975 \times 25 \times 225 \times 247.5$

$= 9606.09N$

$$A_{sv} = \frac{1.28\, s(V_{sd} - V_{cd})}{f_{ykd}}$$

$V_{cd} = 0.48$

$V_{cd} = 0.48 \times 225 \times 247.5 = 26730$

$$157 = \frac{1.28\, s(26730 - 15290)}{250 \times 247.5}$$

$S = 200.47mm$

Provide 2legs Y10mm at 200mm c/c.

DEFLECTION CHECK

Service stress $\sigma_s = \frac{5}{8} \times 410 \times \frac{139.7}{402}$

$= 88.92N/mm^2$

$M_F = \frac{310}{\sigma_s} = \frac{310}{88.92} = 3.5$; 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 $\qquad \frac{x - 20}{28 - 20} = \frac{0.7 - 1.5}{0.5 - 1.5}$

0.7 —— x

1.5 —— 20 $\qquad \frac{x - 20}{8} = \frac{-0.8}{-1}$, $x - 20 = 6.4$

$x = 26.4$

limiting span: mf x basic span ratio $= 2 \times 26.4$

$= 52.8$

Actual span: $\frac{lx}{d} = \frac{6000}{247.5} = 24.24$

... ok.

FLAT SLAB

DESIGN TO BS8110

& EUROCODE

BS 1

REFERENCE	CALCULATION	OUTPUT

FLAT SLAB

Effective diameter of column heads

$h_c = \left(\frac{4A}{\pi}\right)^{1/2} \leq 0.25 l_x$

Depth of Drops = 100mm.

Column size = 300mm x 300mm

Column Grid = 6m x 6m

Flat slab has 2000mm x 2000mm drops at column

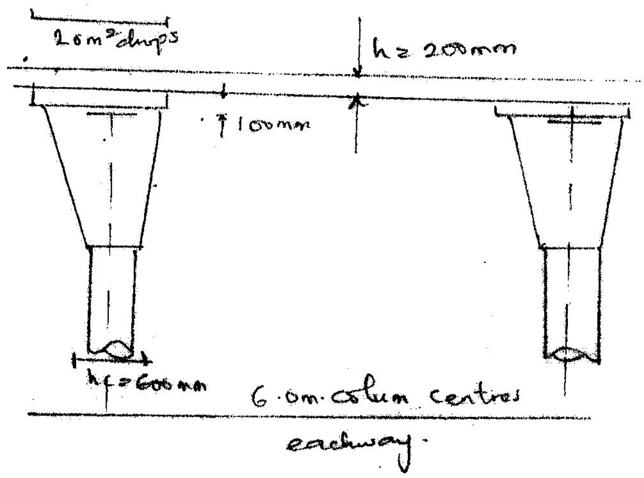
Assume flat slab thickness of 200mm

Live load on floor = $4 kN/m^2$.

$h_c = \left(\frac{4 \times 300 \times 300}{\pi}\right)^{1/2} \leq 0.25(6000)$

$= 338.51mm \leq 1500mm$. (O.K).

Use $h_c = 600mm$



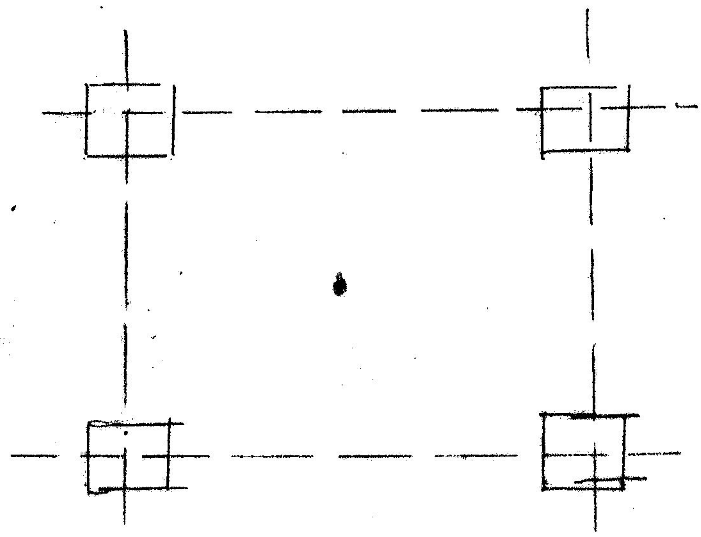
Dead load:

Weight of slab $= 0.2 \times 24 \times 6.0^2$
$= 172.8kN$

Weight of drop $= 0.1 \times 24 \times 2^2$
$= 9.6kN$

Total load $= 172.80 + 9.60 = 186.40kN$

REFERENCE	CALCULATION	OUTPUT
	Ultimate load on floor, F	
	$= 1.4(186.40) + 1.6(144)$	
	$= 491.36$ 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	CALCULATION	OUTPUT

BENDING REINFORCEMENT:-

1.) Centre of the 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 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 $= (1-0.6) \times 195.36$

$= 78.14 kN.m$

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

$= 200 - 25 - 16 - 8$

$= 151 mm$

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

$l_a = 0.5 + \sqrt{0.25 - 0.05/0.9} = 0.94$

$Z = l_a d = 0.94 \times 151 = 141.94$

$$As_{req} = \frac{M}{0.95 f_y Z} = \frac{117.22 \times 10^6}{0.95 \times 410 \times 141.94} = 2120.26 mm^2$$

Provide 11-Y16mm, $As_{prov} = 2212 mm^2$

(b) For Column Strip

REFERENCE	CALCULATION	OUTPUT
	$l_a = 0.5 + \sqrt{0.25 - \frac{0.07}{0.9}} = 0.91$	
	$Z = l_a d = 0.91 \times 151 = 137.41$	
	$As_{req} = \frac{78.14 \times 10^6}{0.95 \times 410 \times 137.41} = 1460.00 mm^2$	
	Provide 8-Y16mm, $As_{prov.} = 1610 mm^2$	
	2.) Interior Support	
	Negative Moment $= -0.055 F_l$	
	$= -0.055 \times 491.36 \times 5.6$	
	$= 151.34 kN.m$	
	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	
	$K = \frac{M}{f_{cu} b d^2} = \frac{50.45 \times 10^3}{25 \times 4000 \times 151^2} = 0.022$	
	$l_a = 0.5 + \sqrt{0.25 - \frac{0.022}{0.9}} = 0.97$	
	Since l_a is greater than 0.95, use $l_a = 0.95$.	
	$Z = 0.95 \times 151 = 143.45$	
	$As_{req} = \frac{50.45 \times 10^6}{0.95 \times 410 \times 143.45} = 903 mm^2$.	
	Provide 9-Y12mm, $As_{prov} = 1020 mm^2$.	
	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 $l_a = 0.95$	

REFERENCE	CALCULATION	OUTPUT

$As_{req} = \frac{100.89 \times 10^6}{0.95 \times 410 \times 23..45} = 1086.30mm^2$

Provide 6 - Y16mm , $As_{prov} = 1210mm^2/m$

PUNCHING SHEAR

1) At column head;

Perimeter, $u = \pi d$

$= 3.142 \times 600$

$= 1885.2mm$

Shear force, $V = F - \frac{\pi}{4} \times 0.6^2 \times n$

$= 491.36 - \frac{3.142 \times 0.6^2}{4} \times 13.65$

$= 487.5KN$

To allow for the effects of moment transfer, V is increased by its percent, i.e

$v = \frac{1.15V}{ud} = \frac{1.15 \times 487.5}{1885.2 \times 251} = 1.18 N/mm^2$

which is less than $0.8\sqrt{f_{cu}}$ or $5N/mm^2$

2) First Critical Perimeter is $1.5d = 1.5 \times 251$

$= 376.5mm$

thus, length of perimeter, $u = 4(600 + 2 \times 376.5)$

$= 5412mm$

Ultimate Shear force,

$= 491.36 - (0.6 + 2 \times 0.376)^2 \times 13.65$

$= 466.41KN$

thus Shear Stress $v = \frac{1.15 \times 466.41 \times 10^3}{5412 \times 251}$

$= 0.39N/mm^2$

$V_c = 0.79 \times \left(\frac{100As}{bd}\right)^{1/3}$

REFERENCE	CALCULATION	OUTPUT

FLAT SLAB

Effective diameter of column head

$h_c = \left(\frac{4A}{\pi}\right)^{1/2} \leq 0.25 l_x$

Depth of drops = 100mm

Column size = 300mm X 300mm

Column grid = 6m X 6m

Flat slab has 2000mm x 2000mm drops at column

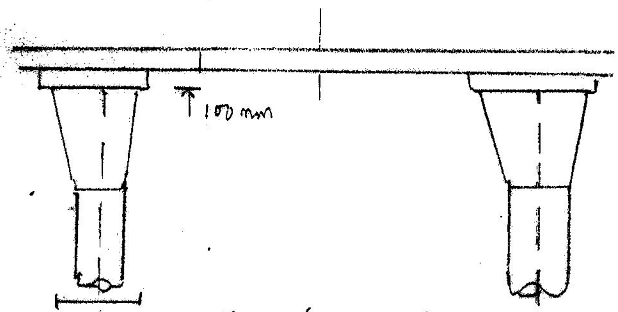
Assume flat slab thickness of 200mm

Live load on floor = 4kN/mm²

$h_c = \left(\frac{4 \times 300 \times 300}{\pi}\right)^{1/2} \leq 0.25(6000)$

$= 338.51mm \leq 1500mm$ (O.K)

Use $h_c = 600mm$



$h_c \ngtr 600mm$ 6m column centre eachway.

Dead Load

weight of slab $= 0.2 \times 24 \times 6.0^2$
$= 172.8kN$

weight of drop $= 0.1 \times 24 \times 2.0^2$
$= 9.6kN$

Total load $= 172.8 + 9.6 = 186.40kN$

Live load

Total live load $= 4 \times 6^2 = 144kN$

Ultimate load on floor, w

$w = 1.35(186.40) + 1.5(144)$

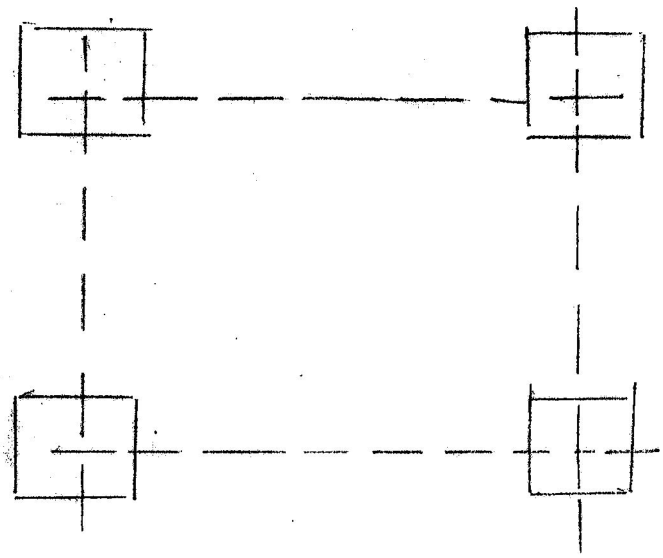
REFERENCE	CALCULATION	OUTPUT

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 = $0.071wL$

$= 0.071 \times 467.64 \times 5.6$

$= 185.93 kN \cdot m$

∴ width of middle strip is

$(6.0 - 2.0)m = 4m$, 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 $= 0.6 \times 185.93$

$= 117.22 kN \cdot m$

The column strip positive moment $= (1 - 0.6) \times 185.93$

REFERENCE	CALCULATION	OUTPUT

The column strip positive moments $= (1-0.6) \times 185.93$
$= 74.37\,kN.m$

a) for middle strip

$$k = \frac{M}{f_{ck}bd^2} = \frac{111.56 \times 10^6}{25 \times 4000 \times 151^2} = 0.05$$

where $d = h - c - \phi - \phi/2$
$= 200 - 25 - 16 - 8$
$= 151\,mm$

$$l_a = 0.5 + \sqrt{0.25 - k/1.134} = 0.95$$

$Z = l_a d = 0.95 \times 151$
$= 143.45\,mm$

$$As_{req} = \frac{M}{0.87 f_y Z} = \frac{111.56 \times 10^6}{0.87 \times 410 \times 143.45} = 2180.24\,mm^2$$

Provide 11 - Y16mm, $As_{prov} = 2212\,mm^2$

b) For column strip

$$k = \frac{M}{f_{ck}bd^2} = \frac{74.37 \times 10^6}{25 \times 2000 \times 155^2} = 0.06$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.06}{1.134}} = 0.94$$

$Z = l_a d = 0.94 \times 155$
$= 145.7\,mm$

$$As_{req} = \frac{74.37 \times 10^6}{0.87 \times 410 \times 145.7} = 1430.99\,mm^2$$

Provide 8 - Y16mm, $As_{prov} = 1610\,mm^2$

2) Interior support

Negative ~~moment~~ $= -0.055 F l$
$= 0.055 \times 467.64 \times 5.6$
$= 144.03\,kN$

And this is divided into:

middle strip $= 0.25 \times \frac{1}{6/3} \times 144.03$

REFERENCE	CALCULATION	OUTPUT

a) for middle strip

$$k = \frac{M}{f_{ck}bd^2} = \frac{48.01 \times 10^6}{25 \times 4000 \times 151^2} = 0.021$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{k}{1.134}} = 0.98$$

Since l_a is greater than 0.95, we use $l_a = 0.95$

$Z = 0.95 \times 151$

$= 143.45mm$

$$As_{req} = \frac{48.01 \times 10^6}{0.87 \times 460 \times 143.45}$$

$= 938.27mm^2$

Provide 9-Y12mm, $As_{prov} = 1020mm^2$

b) for Column strip

$d = 300 - 25 - 16 - 8$

$d = 251mm$

$$k = \frac{M}{f_{ck}bd^2} = \frac{96.50 \times 10^6}{25 \times 2000 \times 251^2} = 0.031$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.031}{1.134}}$$

$l_a = 0.97$

Since l_a is greater than 0.95, we use $l_a = 0.95$

$Z = 0.95 \times 251 = 238.45mm$

$$As_{req} = \frac{M}{0.87 f_y Z} = \frac{96.50 \times 10^6}{0.87 \times 410 \times 238.45}$$

$= 1134.56mm^2$

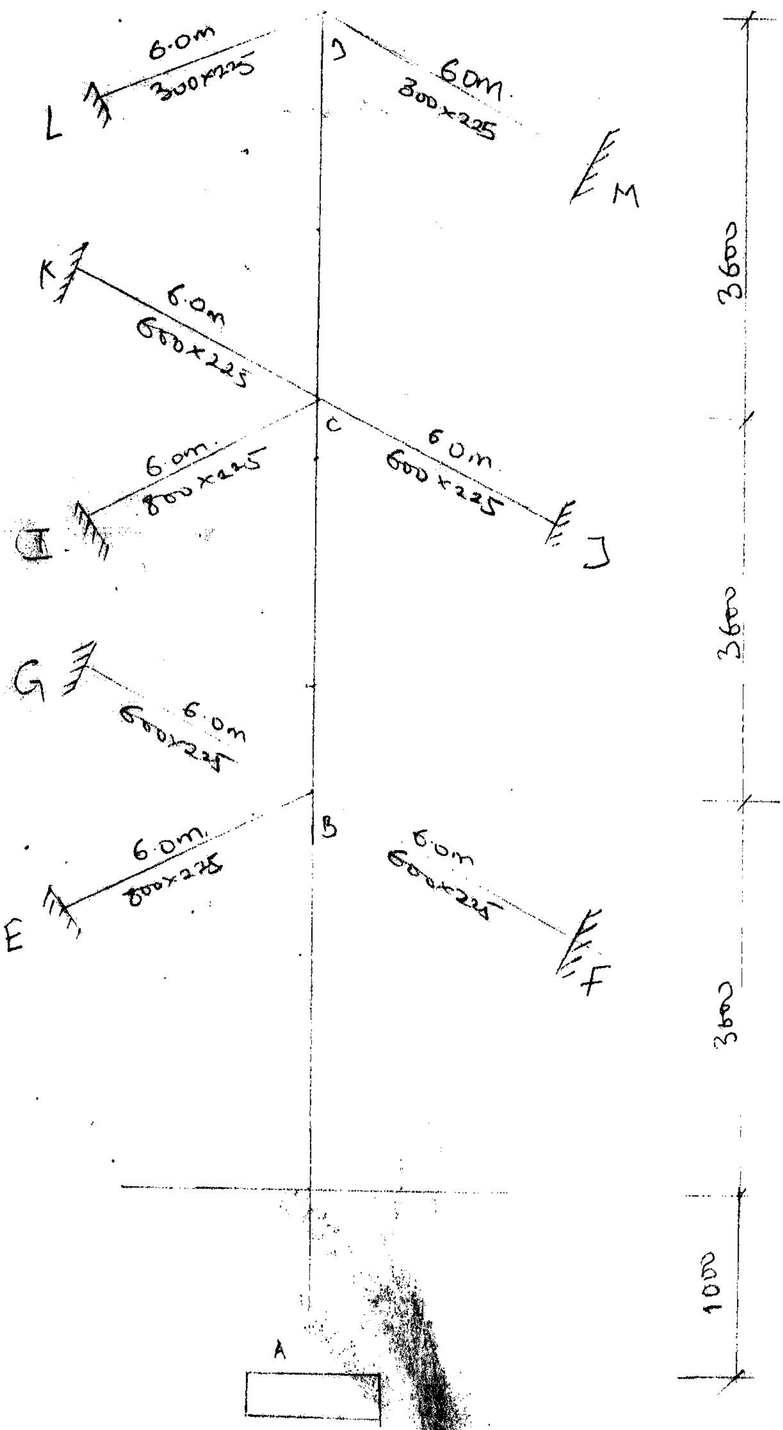
Provide 6-Y16mm, $A_{sprov} = 1210mm^2/m$

COLUMN

DESIGN TO EUROCODE

& BS8110

Reference	Calculations	Output
	Column Design	



Reference	Calculations	Output

SLENDERNESS CALCULATIONS.

Consider of Plane ... DM

Roof

300x225

M

3600

$\beta = 0.75$

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

$l_e/h = \frac{2700}{225} \approx 12 < 15$
(short column)

51.53 kN/m

47.90 kN/m

K

J

$\beta = 0.75$

$l_e = 0.75 \times 3600$
[illegible]

l_e/h [illegible]

51.53 kN/m

G

$\beta = 0.90$

$l_e = 0.90 \times 3600$
[illegible]

l_e/h [illegible]
(short column)

Kcolumn [illegible]

[illegible]

Beam, K_{BM} $\frac{1}{2}$ [illegible]

Beam, K_{BJ} $\frac{1}{2}$ [illegible]

[illegible]

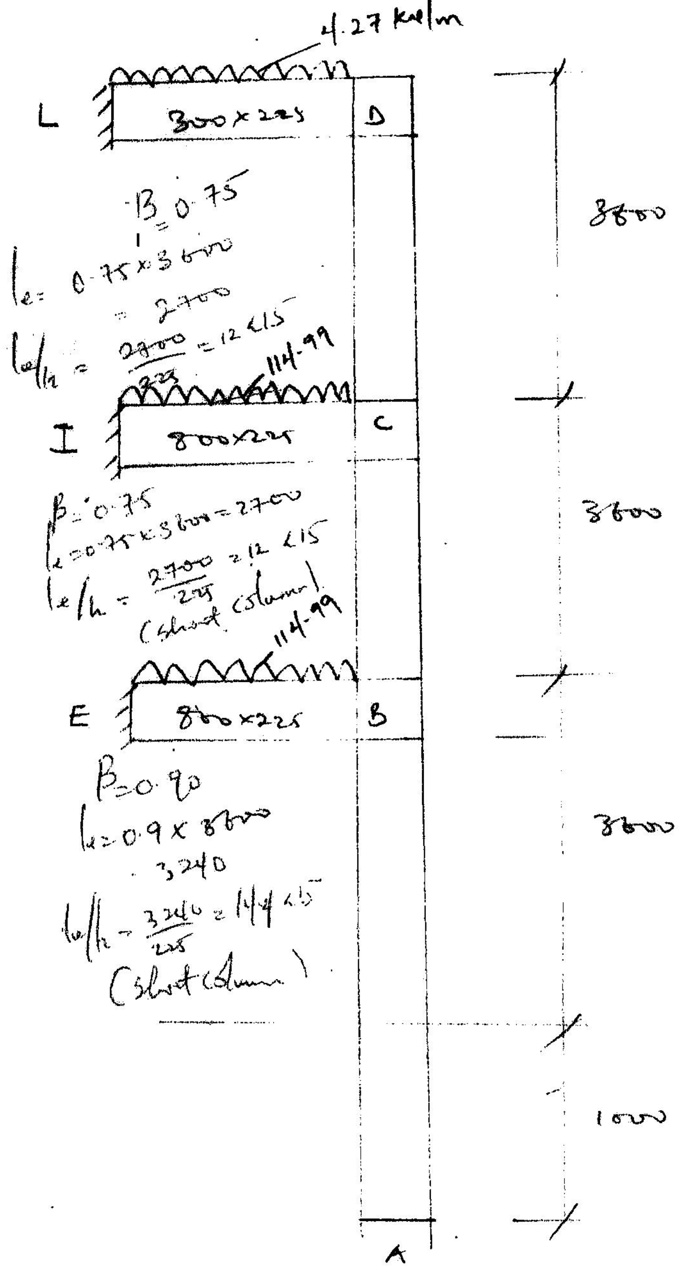
Reference	Calculations	Output

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

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

$$Beam,\ K_{DM} = \frac{1}{2}\left(\frac{225 \times 300^3}{12 \times 6000}\right) = 0.84 \times 10^5$$

PLANE ABECIDL;



Reference	Calculations	Output

$K_{col_{BA}} = \frac{0.75I}{L} = \frac{0.75 \times 225 \times 225^3}{12 \times 4600} = 0.35 \times 10^5$

Beam, $K_{BE} = \frac{I}{L} = \frac{1}{2}\left(\frac{225 \times 800^3}{12 \times 6000}\right) = 80 \times 10^5$

Beam, $K_{CI} = \frac{1}{2}\left(\frac{225 \times 800^3}{12 \times 6000}\right) = 8.0 \times 10^5$

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

$K_{col_{CD}} = \frac{225 \times 225^3}{12 \times 3600} = 0.59 \times 10^5$

Beam, $K_{DL} = \frac{1}{2}\left(\frac{225 \times 300^3}{12 \times 6000}\right) = 0.42 \times 10^5$

Moment Of Members On Plane ABECIDL

$F.E.M._{DL} = \frac{wl^2}{12} = \frac{4.27 \times 6^2}{12} = 12.81$ kN.m

$Moment_{DC} = \frac{12.81 \times 0.59}{0.42 + 0.59} = 7.48$ kN.m.

$F.E.M_{CI} = \frac{114.99 \times 6^2}{12} = 344.97$

$Moment_{CD} = \frac{344.97 \times 0.59}{0.59 + 8 + 0.59} = 22.17$ kN.m.

$Moment_{CD} = Moment_{CB} = 22.17$ kN/m (Since they have the same stiffness).

Joint B; $Moment_{BE} = \frac{wl^2}{12} = \frac{114.99 \times 6^2}{12} = 344.97$

$Moment_{BC} = \frac{344.97 \times 0.59}{0.59 + 8 + 0.35} = 22.77$ kN.m.

$Moment_{BA} = \frac{344.97 \times 0.35}{0.59 + 8 + 0.35} = 13.51$ kN.m.

REFERENCE	CALCULATIONS	OUTPUT.

Moment of Members On plane AGBFKCJDM:-

$F.E.M_{DM} = \frac{4.[illegible] \times 6^2}{12}$ [illegible] KN/m.

M[illegible] $= \frac{[illegible]}{[illegible]}$ = 7.52 KN/m

[illegible]

[illegible] 5.4 [illegible]

Joint C;

$51.33 \times 0.39 =$ [illegible] KN/m.

$F.E.M_{CK} = \frac{wl^2}{12} = \frac{20.01 \times 6^2}{12} = 60.03$ KN.m.

$F.E.M_{CJ} = \frac{47.7 \times 6^2}{12} = 143.1$ KN m.

Out of balance Moment $= 143.1 - 60.03 = 83.07$ KN.m

$M_{CD} = \frac{83.07 \times 0.39}{0.39 + 3.37 + 3.37} = 6.69$ KN.m

$M_{CD} = M_{CB} = 6.69$ KN.m

Joint B;

$51.33 \times 0.39 = 20.01$

$F.E.M_{BG} = \frac{20.01 \times 6^2}{12} = 60.03$ KN.m

$F.E.M_{BF} = \frac{47.7 \times 6^2}{12} = 143.1$ KN.m

Out of balance Moment $= 83.07$ KN.m

$M_{BC} = \frac{83.07 \times 0.39}{(0.39 + 3.37 + 3.37 + 0.35)} = 6.3$ KN.m.

$M_{BA} = \frac{83.07 \times 0.35}{} = 3.79$ KN.m.

Reference	Calculations	Output
	a) At roof level; from DM = 0.5×4.27×6 = 12.81 kN.	
	" DL = 0.5×4.27×6 = 12.81 kN	
	25.62 kN.	
	Self Weight 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	
	S/w 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	
	S/w of column 6.12	
	1329.32 kN.	

CALCULATIONS	OUTPUT.

DESIGNING THE MOST LOADED COLUMN:-

$M_x = 13.51 \text{kN.m}$.

$M_y = 3.79 \text{kN.m}$.

$N = 1329.32 \text{kN}$.

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

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

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

$\beta = 0.3$.

Since $M_x/h' \geq M_y/b'$ $\therefore$ $M_x' = M_x + \beta \frac{h'}{b'} M_y$

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

$$= 13.51 + 1.137 = 14.65 \text{kN.m}.$$

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

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

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

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

$\therefore$ Provide 4 - Y25 mm + 2 Y20 mm

$A_{s\,prov} = 2591.82 \text{mm}^2$

	Calculations	Output.

Designing the Most Loaded Column:-

$M_z = 13.51 \, kN.m.$

$M_y = 3.79.$

$N = 1329.32 \, kN.$

$$e_z = \frac{M_z}{N_{ED}} = \frac{1351 \times 10^6}{1329.32 \times 10^3} = 10.16.$$

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

thus

$$e_{y}/_{x} \quad \frac{e_z}{h} \Big/ \frac{e_y}{b} = \frac{10.16}{225} \Big/ \frac{2.85}{225} = 3.56 > 0.2.$$

$\therefore$ both columns can be designed as biaxially Loaded column.

$$\frac{M_z}{h'} = \frac{1351 \times 10}{177.5} = 0.08$$

$$\frac{M_y}{b'} = \frac{379}{177.5} = 0.02.$$

Since $M_z/h' > M_y/b'$ $\therefore$

$$M_z' = M_z + \beta \frac{b'}{h'} M_y.$$

$$\frac{N_{ED}}{bhf_{ck}} = \frac{1329.32 \times 10^3}{225 \times 225 \times 25} = 1.05. \geq 0.7 \therefore \beta = 0.3.$$

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

$= 14.65 \, kN.m.$

$$\therefore \frac{M_{ED}}{bh^2 f_{ck}} = \frac{14.65 \times 10^6}{225 \times 225^2 \times 25} = 0.05.$$

V4.

Reference	Calculations	Output
	From Design chart;	
	$\frac{A_s f_{yk}}{bh f_{ck}} \approx 0.65$	
	$A_s = \frac{0.65 \times bh f_{ck}}{f_{yk}}$	
	$= \frac{0.65 \times 225 \times 225 \times 25}{410} = 2006.48$	
	$= 2007 \ mm^2$.	
	Provide 4-Y25mm + 2Y16mm.	
	$A_{s\,prov} = 2365.62 mm^2$	

BEAM 3 AND BEAM 6

DESIGN TO BS8110

BEAM 3

85.98kN/m	80.44kN/m	80.44kN/m	91.50kN/m
A 6 B	6 C	6 D	6 E
Selfweight of beam $= 0.225 \times 0.4 \times 24 \times 1.4 = 3.024kN/m$	S/w of beam $= 3.024kN/m$	S/w of beam $= 3.024kN/m$	Selfweight of beam $= 3.024kN/m$
Load from panel 6 $= 33.18kN/m$	Load from Panel 7 $= 30.4kN/m$	Load from Panel 8 $= 30.4kN/m$	Load from panel 8 $= 35.94kN/m$
Load from panel 10 $= 33.18kN/m$	Load from panel 11 $= 30.4kN/m$	Load from Panel 12 $= 16.6kN/m$	Load from panel 13 $= 35.94kN/m$
Partition Load $= 3.47 \times 3.42 \times 1.4 = 16.6kN/m$	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

$$FEM_{AB} = \frac{WL^2}{8} = \frac{85.98 \times 6^2}{8} = 386.76KNm$$

SPAN B-C

$$FEM_{BC} = \frac{WL^2}{12} = \frac{80.44 \times 6^2}{12} = 241.32KNm$$

SPAN C-D

$$FEM_{CD} = \frac{WL^2}{12} = \frac{80.44 \times 6^2}{12} = 241.32KNm$$

SPAN D-E

$$FEM_{DE} = \frac{WL^2}{8} = \frac{91.5 \times 7.183^2}{8} = 590.12KNm$$

STIFFNESS, K

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

$= .167$

$K_{DE} = \frac{0.75}{L} = \frac{0.75}{7.183} = 0.104.$

DISTRIBUTION FACTOR

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

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

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

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

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

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

MOMENT DISTRIBUTION.

A	B	B	C	C	D	D	E
85.98		80.44		80.44		91.50	
6		6		6		6	
0	0.43	0.57	0.5	0.5	0.62	0.38	0
	−386.26	+241.32	−241.52	+241.52	−241.52	+590.12	
	+62.54	+82.90	0	0	−216.24	−132.54	
		0	+41.45	−108.13	0		
			+33.34	+33.64			
		+16.67			+16.66		
	−7.17	−9.50			−10.34	−6.33	
0	−331.39	+331.39	−166.5	+166.5	451.2	+451.25	0

SHEAR FORCE.

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

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

$= 202.71 \text{ KN}.$

REFERENCE	CALCULATIONS	OUTPUT

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

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

$$= 313.17KN.$$

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

$$= \frac{80.44 \times 6}{2} + \frac{331.39 - 166.5}{6}$$

$$= 268.8KN.$$

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

$$= \frac{80.44 \times 6}{2} + \frac{166.5 - 331.39}{6}$$

$$= 213.84KN$$

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

$$= \frac{80.44 \times 6}{2} + \frac{166.5 - 451.25}{6}$$

$$= 193.86KN.$$

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

$$= \frac{80.44 \times 6}{2} + \frac{451.25 - 166.5}{6}$$

$$= 288.78KN$$

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

$$= \frac{91.5 \times 7.183}{2} + \frac{451.25 - 0}{7.183}$$

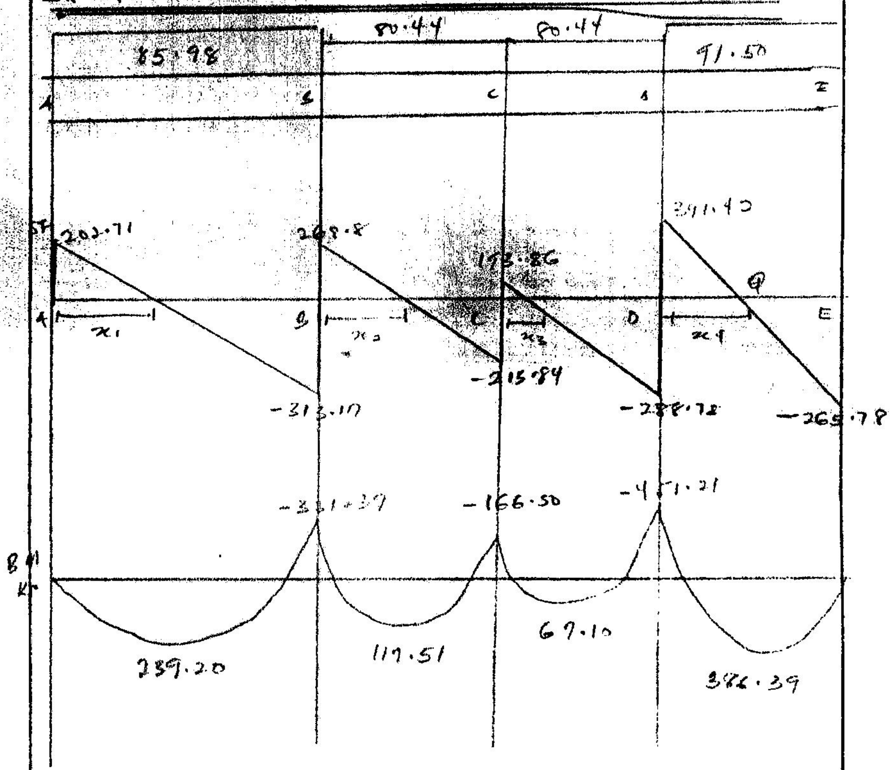
$$= 328.6 + 62.82$$

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

$$= 265.78KN.$$

REFERENCE.	CALCULATIONS	OUTPUT

SHEAR FORCE AND BENDING MOMENT DIAGRAM



Span moment A-B = Area of triangle AOQ

$= 1/2 \times x_1 \times 202.71$

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

$x_1 = 2.36m$

Moment @ span A-B $= 1/2 \times 2.36 \times 202.71$

$= 239.2kNm$

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

$= 1/2 \times x_2 \times 268.8 - 331.39$

$x_2: 268.8 - 80.44x_2 = 0$

$x_2 = 3.34m$

Moment @ span B-C $= 1/2 \times 3.34 \times 268.8 - 331.39$

$= 117.51kNm$

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

$= 1/2 \times x_3 \times 193.86 - 166.50$

$x_3; 193.86 - 80.44x_3 = 0$

$x_2 = 2.41m$

Moment at span B-C $= 1/2 \times 2.41 \times 193.86 - 166.50$

$= 67.10kNm$

Span moment D-E = Area of triangle − support moment @ DOQ

$= 1/2 \times x_4 \times 391.42 - 451.25$

x_4; $391.42 - 91.5x_4 = 0$

$x_4 = 4.28m$

Moment @ span D-E = $1/2 \times 4.28 \times 391.42 - 451.25$

$= 386.39kNm$

DESIGN OF SUPPORT REINFORCEMENT

M_{max} @ support $= 451.25kNm$

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

$= 700 - 30 - \frac{32}{2} - 10 = 644mm$

$K = \frac{M}{f_{cu}bd^2} = \frac{451.25 \times 10^6}{25 \times 300 \times 644^2} = 0.145$

Since $k < 0.156$, compression reinforcement is not required

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

$= 0.5 + \sqrt{0.25 - 0.145/0.9}$

$= 0.5 + 0.8$

$= 0.8$

$\therefore z = l_a d = 0.80 \times 644$

$= 515.2mm$

$As_{req} = \frac{M}{0.95 f_y z} = \frac{451.25 \times 10^6}{0.95 \times 410 \times 515.2}$

$= 2248.7mm^2$

Provide 5-Y25mm

$As_{prov} = 2450mm^2$

$b_f = b_{eff} + \frac{l_z}{5}$

$b_f = 300 + \frac{0.85 \times 6000}{5} = 1000mm$

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 OK

DESIGN OF SPAN REINFORCEMENT

$M_{max} = 386.39 kNm$

$K = \frac{M}{b_f f_{cu} d^2} = \frac{386.39 \times 10^6}{1000 \times 644^2 \times 25} = 0.036$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.036}{0.9}} = 0.95$

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

$Z = l_a d = 0.95 \times 644 = 611.8mm$

$A_{s\,req} = \frac{386.39 \times 10^6}{0.95 \times 410 \times 611.8} = 1621.47mm^2$

Provide 6Y20mm, A_s prov $= 1890mm^2$

$b_w/b_f = \frac{300}{1000} = 0.29$

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

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

$= 378mm^2$

Since As req is greater than As min, provision is ok | Provision is ok

DESIGN OF SHEAR REINFORCEMENT.

$V_{max} = 391.42kN$

$V = \frac{N_{max}}{bd} = \frac{391.42 \times 10^3}{300 \times 644} = 2.03N/mm^2$

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

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

$V_c = \frac{0.79 \left(^{100A_s}/_{bd}\right)^{1/3} \left(^{400}/_d\right)^{1/4}}{\gamma_m}$

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

$= 0.61 N/mm^2$

$(v_c + 0.4) < 0.8\sqrt{f_{cu}}$ or $5N/mm^2 \rightarrow$ satisfies the Condition, therefore $S_v = \frac{A_{sv}\, 0.95 f_{yv}}{0.4 b_v}$

$S_v = \frac{157 \times 0.95 \times 250}{0.4 \times 300} = 310.73mm$

REFERENCES	CALCULATIONS	OUTPUT
	DEFLECTION CHECK.	

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

$= \frac{2}{3} \times 410 \times \frac{1601.47}{1890}$

$= 234.50 N/mm^2$

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

$\frac{M}{bd^2} = \frac{386.39 \times 10^6}{1020 \times 644^2}$

$M_F = 0.55 + \frac{477 - 234.50}{120(0.9 + 0.91)} = 1.67$

Since $\frac{b_w}{b_f} = 0.29 < 0.3$, basic span ratio $= 20.8$

limiting span $= 1.67 \times 20.8$

depth ratio $= 34.74$

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 OK.

CALCULATION | OUTPUT

DESIGN OF SPAN REINFORCEMENT

$M_{max} = 386.39kNm$

$$k = \frac{M}{b_f f_{cu} d^2} = \frac{386.39 \times 10^6}{1000 \times 644^2 \times 25} = 0.036$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.036}{0.9}} = 0.95$$

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

$Z = l_a d = 0.95 \times 644 = 611.8mm$

$$A_{s\,req} = \frac{386.39 \times 10^6}{0.95 \times 410 \times 611.8} = 1621.47mm^2$$

Provide 6Y20mm, A_s prov = $1890mm^2$

$$b_w/b_f = \frac{300}{1000} = 0.29$$

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

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

$= 378mm^2$

Since As req is greater than As min, provision is ok | Provision is ok

DESIGN OF SHEAR REINFORCEMENT.

$V_{max} = 391.42kN$

$$V = \frac{V_{max}}{bd} = \frac{391.42 \times 10^3}{300 \times 644} = 2.03N/mm^2$$

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

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

$$V_c = \frac{0.79 \left(^{100A_s}/_{bd}\right)^{1/3} \left(^{400}/_d\right)^{1/4}}{\gamma_m}$$

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

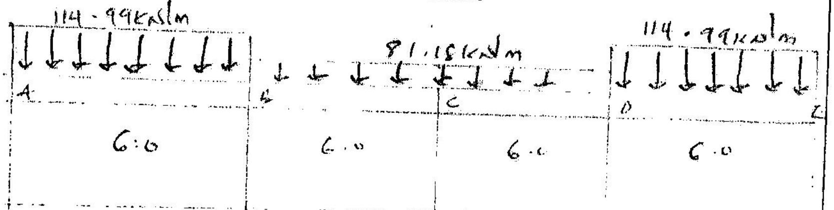
$= 0.61N/mm^2$

$(v_c + 0.4) < 0.8\sqrt{f_{cu}}$ or $5N/mm^2$ → satisfies the Condition, therefore $S_v = \frac{A_{sv}\, 0.95 f_{yv}}{0.4 b_v}$

$$S_v = \frac{157 \times 0.95 \times 250}{0.4 \times 300} = 310.73mm$$

REFERENCES	CALCULATION	Output

BEAM 6 (SIX)



2↑

S/w of beam = 0.225×0.4×2.4 ×1.4 = 3.024kN/m	S/w of beam = 3.024kN/m	S/w of beam = 3.024kN/m	S/w of beam = 3.024kN/m
Load from panel 1 = 52.2kN/m	Load panel 5 = 33.18kN/m	Load panel 5 = 33.18kN/m	Load from panel 1 = 52.2kN/m
Load from panel 2 = 45.2kN/m	Load from panel 6 = 30.41kN/m	Load from panel 6 = 30.41kN/m	Load from panel 2 = 45.2kN/m
Partition load = 3.47×3.0×1.4 = 14.57kN/m	Partition load = 3.47×3.0×1.4 = 14.57kN/m	Partition load = 3.47×3.0 ×1.4 =	Partition load = 14.57kN/m
Total load = 11.99kN/m	Total load = 81.18kN/m	Total load = 81.18kN/m	Total load = 114.99kN/m

FIXED END MOMENT

$$f.E.m_{A-B} = \frac{wl^2}{8} = \frac{114.99 \times 6^2}{8} = 517.46\,kN.m$$

$$f.E.m_{BC} = \frac{wl^2}{12} = \frac{81.18 \times 6^2}{12} = 243.54\,kN.m$$

$$f.E.m_{BC} = f.E.m_{CD} = 243.54\,kN.m$$

$$f.E.m_{DE} = \frac{wl^2}{8} = \frac{114.99 \times 6^2}{8} = 517.46\,kN.m$$

STIFFNESS (K):

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

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

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

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

G.L.

REFERENCE	CALCULATION	OUTPUT

DISTRIBUTION FACTOR (D.F.)

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

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

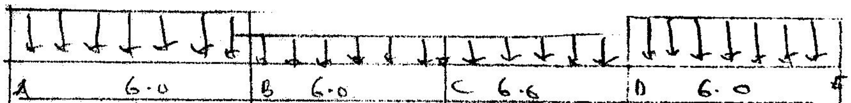
$$D \cdot f_{CB} = \frac{K_{BC}}{K_{BC} + K_{CD}} = \frac{0.167}{0.167 + 0.167} = 0.50$$

$$D \cdot f_{CB} = D \cdot f_{CD} = 0.5$$

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

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

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				
		78.67	78.67	0	
0	0	0	0	0	0
+	−	+	−	+	−
399.67	399.67	165.47	165.47	399.67	399.67

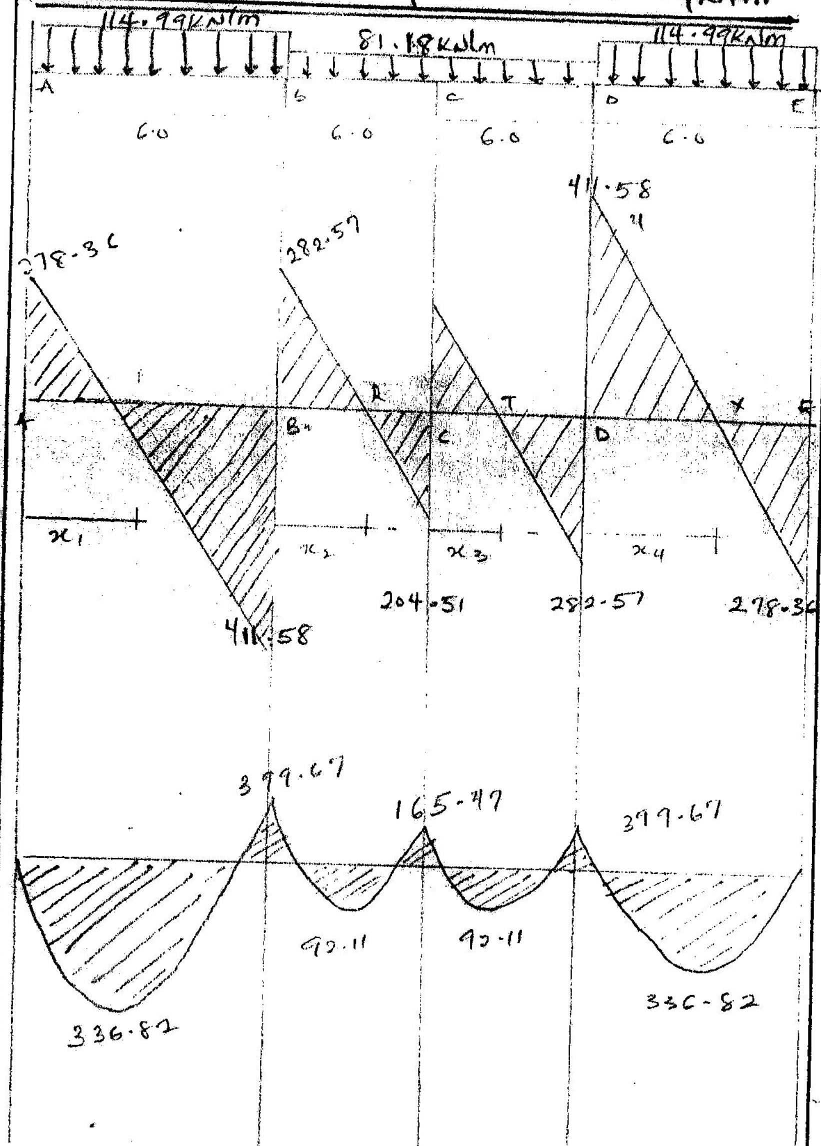
SHEAR FORCE.

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

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

REFERENCE	CALCULATION	OUTPUT
	Real Shear force$_{BA}$ = $\dfrac{W_{AB}L_{AB}}{2} + \left(\dfrac{M_B - M_A}{L_{AB}}\right)$	
	$= \dfrac{114.99 \times 6}{2} + \left(\dfrac{399.67 - 0}{6.0}\right)$	
	$= 411.58\,kN$	
	Real Shear force$_{BC}$ = $\dfrac{W_{BC}L_{BC}}{2} + \left(\dfrac{M_B - M_C}{L_{BC}}\right)$	
	$= \dfrac{81.18 \times 6}{2} + \left(\dfrac{165.47 - 399.67}{6}\right)$	
	$= 204.51\,kN$	
	Real Shear force$_{CD}$ = $\dfrac{W_{CD}L_{CD}}{2} + \left(\dfrac{M_C - M_D}{L_{CD}}\right)$	
	$= \dfrac{81.18 \times 6}{2} + \left(\dfrac{165.47 - 399.67}{6}\right)$	
	$= 204.51\,kN$	
	Real Shear force$_{DC}$ = $\dfrac{W_{CD}L_{CD}}{2} + \left(\dfrac{M_D - M_C}{L_{CD}}\right)$	
	$= \dfrac{81.18 \times 6}{2} + \left(\dfrac{399.67 - 165.47}{6}\right)$	
	$= 282.57\,kN$	
	Real Shear force$_{DE}$ = $\dfrac{W_{DE}L_{DE}}{2} + \left(\dfrac{M_D - M_E}{L_{DE}}\right)$	
	$= \dfrac{114.99 \times 6}{2} + \left(\dfrac{399.67 - 0}{6}\right)$	
	$= 411.58\,kN$	
	Real Shear force$_{ED}$ = Real Shear force$_{AB}$ = 278.36 kN	

SHEAR FORCE / BENDING MOMENT DIAGRAM



Span Moment $_{A-B}$ = Area of triangle ABP

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

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

$x_1 = \frac{278.36}{114.99} = 2.42m$

∴ Span Moment $_{A-B} = \frac{1}{2} \times 2.42 \times 278.36 = 336.82 KN/m$

Span Moment $_{AB}$ = 336.82 KN·m

Span Moment $_{BC}$ = Area of triangle BQR − Support moment B

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

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

$x_2 = \frac{282.57}{}\ = 3.48m$

REFERENCE	CALCULATION	OUTPUT
	Span moment$_{B-C}$ $= \left(\frac{1}{2} \times 3.48 \times 282.57\right) - 377.67$	
	$= 12.11 kN.m$	
	Span moment$_{C-D}$ = (Area of triangle CS 1) − (Support moment)	
	$= \left(\frac{1}{2} \times 204.51 \times x_3\right) - 165.47$	
	x_3; $204.51 - 81.18(x_3) = 0$	
	$\therefore x_3 = \frac{204.51}{81.18} = 2.52$	
	Span Moment$_{C-D}$ $= \left(\frac{1}{2} \times 204.51 \times 2.52\right) - 165.47$	
	$= 92.11 kN.m$	
	Span Moment$_{D-E}$ $= \left(\frac{1}{2} \times x_4 \times 411.58\right) - 377.67$	
	x_4; $411.58 - 114.77(x_4) = 0$	
	$\therefore x_4 = \frac{411.58}{114.77} = 3.58$	
	$= \left(\frac{1}{2} \times 3.58 \times 411.58\right) - 377.67 = 336.82 kN.m$	
	DESIGN SUPPORT REINFORCEMENT	
	$M_{max} = 377.67 kN.m$ $h = 800mm$	
	$d = h - c - \phi/2 - \phi_{links} = 800 - 30 - 32/2 - 10 = 744mm$	
	$K = \frac{M}{f_{cu} b d^2} = \frac{377.67 \times 10^6}{25 \times 225 \times 744^2} = 0.128$	
	$l_a = 0.5 + \sqrt{0.25 - K/0.9} = 0.5 + \sqrt{0.25 - \left(\frac{0.128}{0.9}\right)}$	
	$l_a = 0.83$	
	$Z = l_a d = 0.83 \times 744 = 617.52$	
	$As_{req} = \frac{M}{0.95 f_y Z} = \frac{377.67 \times 10^6}{0.95 \times 410 \times 617.52} = 1661.66 mm^2$	
		G.O

REFERENCE	CALCULATIONS	OUTPUT

$b_f = b_{eff} + l_0/5$

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

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

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

$A_{smin} = 0.26\% \, bt = \frac{0.26}{100} \times 225 \times 800 = 468 mm^2$

Since Asreq is greater than Asmin,

Provision is OK!!

DESIGN SPAN REINFORCEMENT

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

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

$l_a = 0.5 + \sqrt{0.25 - \left(\frac{0.02}{0.9}\right)} = 0.97$

Since $l_a = 0.97$ is greater than 0.95, Z/se

$l_a = 0.95$

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

$A_{sreq} = \frac{336.82 \times 10^6}{0.95 \times 410 \times 706.8} = 1223.47 mm^2$

Provide 4 - Y20 mm bar, $A_{sprov} = 1260 mm^2$

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

$A_{smin} = 0.18\% \, bh = \frac{0.18}{100} \times 225 \times 800 = 324 mm^2$

Since Asreq is greater than Asmin, Provision is OK!!

DESIGN SHEAR REINFORCEMENT

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

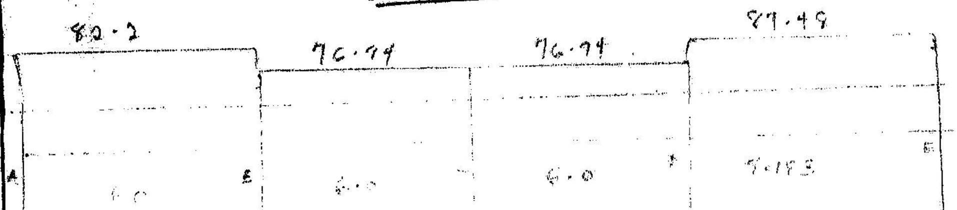
$V = 411.58 \times 10^3$

REFERENCE	CALCULATION	OUTPUT.
	$\frac{100As}{bd} = \frac{100 \times 1960}{225 \times 744} = 1.17$	
	$\frac{400}{d} = \frac{400}{744} = 0.54$	
	$V_c = \frac{0.79 \left(\frac{100As}{bd}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4}}{\lambda m} = \frac{0.79 \times (1.17)^{1/3} \times (0.54)^{1/4}}{1.25}$	
	$V_c = 0.571 N/mm^2$	
	$(V_c + 0.4) \leq V < 0.8\sqrt{f_{cu}}$ or $5N/mm^2$ $\Rightarrow$ satisfies this condition, therefore $S_v = \frac{A_{sv}\, 0.95 f_y}{0.4 b_v}$	
	$S_v = \frac{157 \times 0.95 \times 250}{0.4 \times 225} = 414.31mm$	
	Provides 2legs R10mm @ 400mm dc	

BEAM 3 AND BEAM 6

DESIGN TO EUROCODE

BEAM 3



Selfweight of beam $0.225 \times 0.4 \times 24 \times 1.35 = 2.92$ kN/m	Selweight of beam $= 2.92$ kN/m	Self weight of beam $= 2.92$ kN/m	Self weight of beam $= 2.92$ kN/m
Load from panel 6 $= 31.62$ kN/m	Load from panel 7 $= 28.99$ kN/m	Load from panel 8 $= 28.99$ kN/m	Load from panel 9 $= 34.26$ kN/m
Load from panel 10 $= 31.62$ kN/m	Load from panel 11 $= 28.99$ kN/m	Load from panel 12 $= 28.99$ kN/m	Load from panel 13 $= 34.26$ kN/m
Partition Load $= 3.47 \times 3.425 \times 1.35 = 16.04$ kN/m	Partition Load $= 16.04$ kN/m	Partition load $= 16.04$ kN/m	Partition Load $= 16.04$ kN/m
Total Load $= 82.2$ kN/m	Total Load $= 76.94$ kN/m	Total Load 76.94 kN/m	Total Load $= 87.48$ kN/m

FIXED END MOMENT

SPAN A-B

$$FEM_{AB} = \frac{wL^2}{8} = \frac{82.2 \times 6^2}{8} = 369.9 \text{ kNm}$$

SPAN B-C

$$FEM_{BC} = \frac{wL^2}{12} = \frac{76.94 \times 6^2}{12} = 230.82 \text{ kNm}$$

SPAN C-D

$$FEM_{CD} = \frac{wL^2}{12} = \frac{76.94 \times 6^2}{12} = 230.82 \text{ kNm}$$

SPAN D-E

$$FEM_{DE} = \frac{wL^2}{8} = \frac{87.48 \times 7.183^2}{8} = 564.20 \text{ kNm}$$

STIFFNESS, K

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

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

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

$K_{..}$ 0.75 0.75 0.1..

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

82.2		76.79		76.94		87.48	
A	6 B		6 C		6 D	7.183	E
0	0.43	0.57	0.5	0.5	0.62	0.38	0
	−	+	−	+	−	+	
	369.9	230.82	230.82	230.82	230.82	564.26	
	+	+			−	−	
	59.86	79.28	0	0	206.7	126.68	
			+	−			
		0	39.64	103.35	0		
			+	+			
		+	31.86	31.86	+		
	−6.85	15.93			15.93		
					−9.86	−6.05	
0	−316.95	+316.95	−159.32	+159.32	−431.47	+421.47	0

SHEAR FORCE

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

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

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

$= 277.43$ kN

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

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

$= 257.07$ kN

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

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

= [illegible]55 kN

1 in 2

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

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

$$= 185.46\,kN$$

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

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

$$= 276.18\,kN$$

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

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

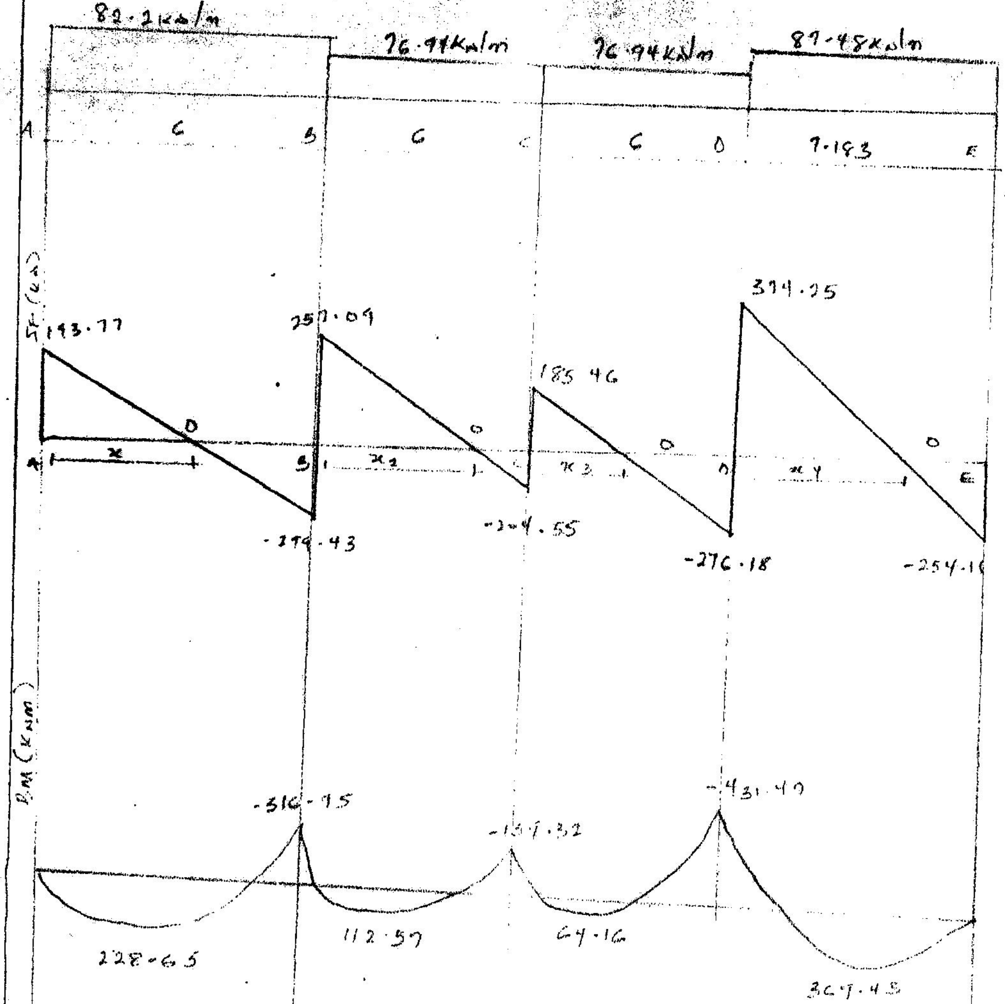
$$= 374.25\,kN$$

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

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

$$= 254.11\,kN$$

SHEAR FORCE AND BENDING MOMENT DIAGRAM



REFERENCE	CALCULATION	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, $257.09 - 76.94x_2 = 0$
$x_2 = 3.34m$

Span moment B-C $= \frac{1}{2} \times 3.34 \times 257.09 - 316.95$
$= 112.59$ 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; $185.46 - 76.94x_3 = 0$
$x_3 = 2.41m$

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
$= \frac{1}{2} \times x_4 \times 374.25 - 481.47$

x_4; $374.25 - 87.48x_4 = 0$
$x_4 = 4.28m$

Span moment D-E $= \frac{1}{2} \times 4.28 \times 374.25 - 431.47$
$= 369.43$ kNm

DESIGN OF SUPPORT REINFORCEMENT

Mmax @ support = 431.47 kNm

$d = h - c - \phi_{link} - \phi/2$
$= 700 - 30 - 32/2 - 10$
$= 644mm$

$K = \frac{M}{f_{ck} bd^2} = \frac{431.47 \times 10^6}{25 \times 300 \times 644^2}$
$= 0.139$

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	CALCULATION	OUTPUT
	$Z = l_a d = 0.85 \times 644$	
	$= 547.4mm$	
	$As_{req} = \frac{M}{0.87 f_y K Z} = \frac{431.77 \times 10^6}{0.87 \times 410 \times 547.4}$	
	$= 2207.75mm^2$	
	Provide 5-Y25mm	
	$As_{prov} = 2450mm^2$	
	$b_f = b_w + 0.17l$	
	$= 300 + 0.17(7.183) = 1521.11mm$	
	$b_w/b_f = \frac{300}{1521.11} = 0.2$	
	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	$M_{max} = 369.43KNm$	
	$K = \frac{M}{b_f f_{ck} d^2} = \frac{369.43 \times 10^6}{1521 \times 644^2 \times 25}$	
	$= 0.023$	
	$l_a = 0.5 + \sqrt{0.25 - \frac{0.023}{1.134}}$	
	$= 0.96$	
	Since $l_a > 0.95$, we use $l_a = 0.96$	
	$Z = l_a d = 0.95 \times 644$	
	$= 611.8mm$	
	$As_{req} = \frac{369.43 \times 10^6}{0.87 \times 410 \times 611.8}$	
	$= 1692.85mm^2$	
	Provide 6Y20mm ; $As_{prov} = 1890mm^2$	

REFERENCE	CALCULATION	OUTPUT
	DESIGN OF SHEAR REINFORCEMENT	
	$V_{max} = 374.25kN$	
	$V_{Rd} = 0.5 v f_{ck} b w d$; $V = 0.575$	
	$= 0.5 \times 0.675 \times 25 \times 300 \times 644$	
	$= 978.08kN$	
	$V_{max} < V_{Rd}$ (Section is OK to cater for shear)	
	$A_{sw} = \dfrac{1.28s(V_{sd} - V_{Rds})}{f_{yk}\, d}$	
	$V_{sd} = V_{max}$	
	$\dfrac{100 A_s}{bd} = \dfrac{100 \times 1897}{300 \times 644}$	
	$= 0.98$	
Table 5.16	$V_{Rd1} = 0.48$	
	$V_{Rd1} = V_{Rd1} b w d$	
	$= 0.48 \times 300 \times 644$	
	$= 92736$	
	$A_{sw} = \dfrac{1.28s(V_{sd} - V_{Rds})}{f_{yk} d}$	
	$157 = \dfrac{1.28s(374250 - 92736)}{250 \times 644}$	
	$s = 70.15mm$	
	Provide 2 legs of R10mm @ 150mm c/c	

REFERENCE	CALCULATION	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 = MF × basic span ratio

1.35 × 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 OK.

DEFLECTION IS SATISFACTORY

REFERENCE	CALCULATION	OUTPUT

BEAM 6

110.07KN/m   77.58KN/m   77.58KN/m   110.07KN/m

A – B	B – C	C – D	D – E
6	6	6	6
Slab of beam $= 0.225 \times 0.4 \times 24 \times 1.35$ $= 2.92KN/m$	Slab of beam $= 2.92KN/m$	Slab of beam $= 2.92KN/m$	Slab 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 $= 3.47 \times 3 \times 1.35$ $= 14.05KN/m$	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

$$F.E.M_{AB} = \frac{wL^2}{8} = \frac{110.07 \times 6^2}{8} = 498.15KNm$$

$$F.E.M_{BC} = \frac{wL^2}{12} = \frac{77.58 \times 6^2}{12} = 232.74KNm$$

$$F.E.M_{BC} = FEM_{CD} = 232.74KNm$$

$$F.E.M_{DE} = FEM_{AB} = 498.15KNm$$

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)

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

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

REFERENCE	CALCULATION	OUTPUT

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

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

$= 194.92 kN$

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

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

$= 270.56 kN$

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

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

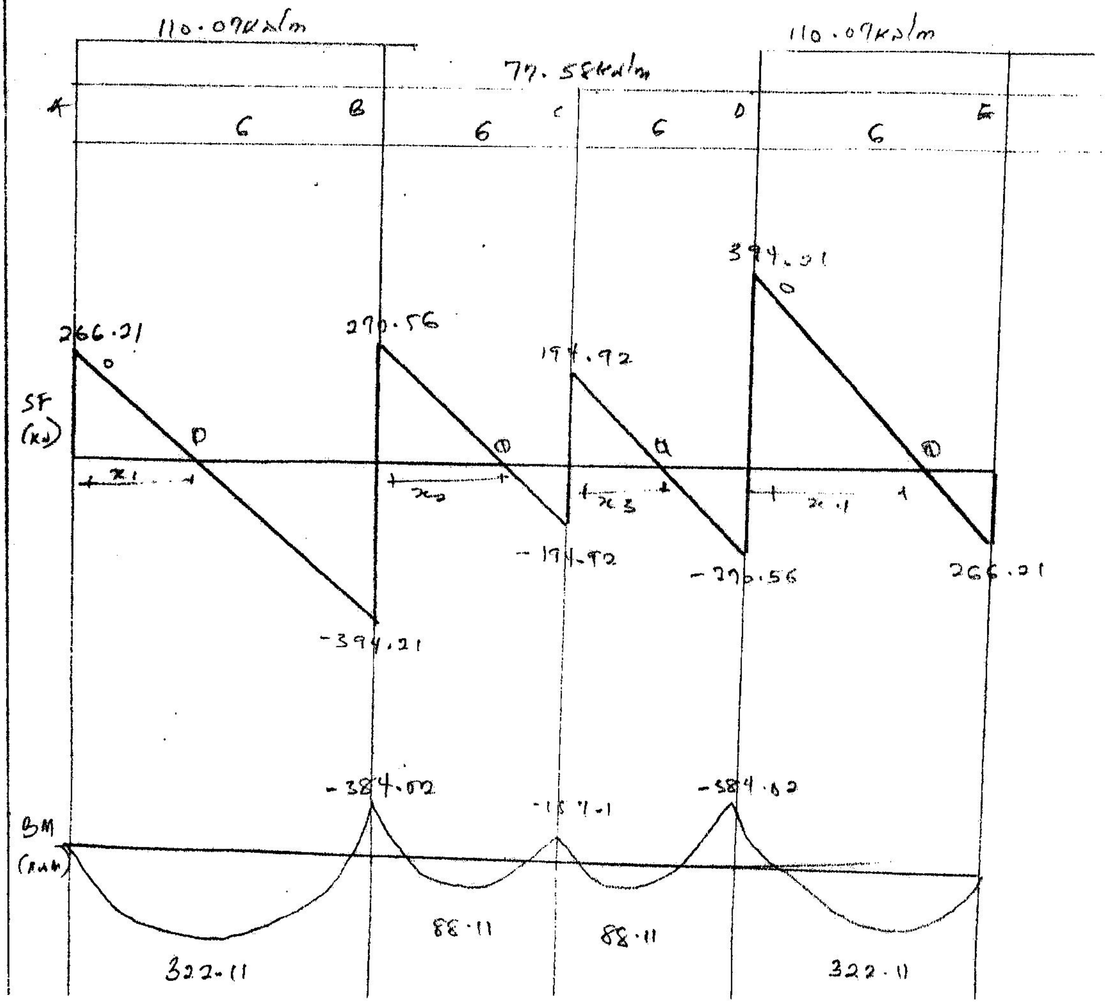
$= 394.21 kN$

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

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

$= 266.21 kN$

SHEAR FORCE AND BENDING MOMENT DIAGRAM



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

$M_{max} = 384.02 KNm$

$h = 800mm$

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

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

$$l_a = 0.5 + \sqrt{0.25 - \frac{K}{1.134}}$$
$= 0.88$

$Z = l_a d$
$= 0.88 \times 744 = 654.72mm$

$$As_{req} = \frac{M}{0.87 f_{yk} Z} = \frac{384.02 \times 10^6}{0.87 \times 410 \times 654.72}$$
$$= 1644.35 mm^2$$

Provide 4 Y 25mm Top

$As_{prov} = 1960 mm^2$

DESIGN OF SPAN REINFORCEMENT

M_{max} @ span = 322.11 KNm

Table 8.9 $b_f = b_w + 0.17 l$ (end Span)

$= 225 + 0.17 \times 6000$

$= 1245mm$

$$K = \frac{M}{f_{ck} b d^2} = \frac{322.11 \times 10^6}{25 \times 1245 \times 744^2}$$
$$= 0.02$$

$$l_a = 0.5 + \sqrt{0.25 - \frac{0.02}{1.134}}$$

$= 0.98$

Since $l_a > 0.98$, we use $l_a = 0.95$

$\therefore Z = l_a d$

$= 706.8mm$

$$As_{req} = \frac{M}{0.87 f_{yk} Z} = \frac{322.11 \times 10^6}{0.87 \times 410 \times 706.8}$$
$$= 1277.63 mm^2$$

Provide 5 - Y 20mm Top, $As_{prov} = 1570 mm^2$

$As_{min} = 0.13\% bh$

$= 0.0013 \times 300 \times 800$

$= 312 mm^2$

DESIGN OF SHEAR REINFORCEMENT

$V_{max} = 394.21 KN$

$V_{Rd,2} = 0.3 v f_{ck} b_w d$, $v = 0.575$

$= 0.3 \times 0.575 \times 25 \times 225 \times 744$

$= 721912.5 N$

$V_{max} < V_{Rd,2}$ (section is O.K to cater for Shear)

$$A_{sw} = \frac{1.28 s (V_{sd} - V_{rd1})}{f_{ywk} d} \qquad V_{sd} = V_{max}$$

$V_{Rd1} = ?$ $\frac{100 A_s}{bd} = \frac{100 \times 1960}{225 \times 744} = 1.17$

$V_{Rd1} = 0.48$

$V_{Rd1} = V_{Rd1} b_w d = 0.48 \times 225 \times 744$

$= 80.352 KN$

REFERENCE	CALCULATION	OUTPUT
	$157 = \frac{1.28s(394.210 - 80352)}{250 \times 744}$	
	$s = 72.689$	
	Provide 2legs R10mm @100mm c/c	
	DEFLECTION CHECK	
	Service Stress; $\sigma_s = \frac{5}{8} \times f_y \times \frac{As\ req}{As\ prov}$	
	$= \frac{5}{8} \times 410 \times \frac{1247.63}{1570}$	
	$= 208.53 N/mm^2$	
	Modification factor $= \frac{310}{\sigma_s} = \frac{310}{208.53}$	
	$= 1.49$	
Table 5.0	$\frac{As}{bd} = \frac{1570}{1245 \times 744} = 0.0017 = 0.17\%$	
	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 $= lx/d = \frac{6000}{744}$	
	$= 8.06$	
	Actual < limiting : deflecting is OK	Deflection is O.K

Solid Slab Design to BS8110

Reference	Calculation	Output
	Panel 8 :-	
	$l_y/l_x = \frac{6000}{6000} = 1.0$	
	Since $l_y/l_x < 2.0$, the Slab is a two Way Spanning Slab.	
	$C = 25mm$, $h = 200mm$.	
	$b = 1000mm$.	
	Additional Partition Load:-	
	$= 3.47 \times 1.4 \times 3.425 \times (6.0+6.0)$ kN.	
	$= 199.60$ kN.	
	This can be assumed to distributed over the Entire area of $6.0 \times 6.0m$.	
	Hence, the U.D.L $= \frac{199.06}{6 \times 6}$ kN/m²	
	$= 5.55$ kN/m²	
	Total U.D.L $= 5.55 + 16.2 = 21.75$ kN/m²	

Short Span

Reference	Continuous Edge	Mid-Span	Output
BS 8110 Table 3.14	$\beta_{sx}^{-} = 0.047$	$\beta_{sx}^{+} = 0.036$	
Clause 3.5.3.4 Equation 14	$M_{sx}^{-} = \beta_{sx}^{-} n l_x^2$	$M_{sx}^{+} = \beta_{sx}^{+} n l_x^2$	
	$= 0.047 \times 21.75 \times 6^2$	$= 0.036 \times 21.75 \times 6^2$	
	$= 36.80$ kN.m	$= 28.19$ kN.m	
	$d = h - c - \phi/2$	$d = h - c - \phi/2$	
	$= 200 - 25 - 12/2$	$= 200 - 25 - 12/2$	
	$= 169mm$	$= 169mm$	
Clause 3.4.4.4	$K = \frac{M}{f_{cu} b d^2} = \frac{36.8 \times 10^6}{25 \times 1000 \times 169^2}$	$K = \frac{28.19 \times 10^6}{25 \times 1000 \times 169^2}$	
	$= 0.052$	$= 0.039$	
Clause 3.4.4.4	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$	
	$= 0.5 + \sqrt{0.25 - 0.052/0.9}$	$= 0.5 + \sqrt{0.25 - \frac{0.039}{0.9}}$	

Short Span

Continous Edge

Since la is less than 0.95 la is o.k.

Clause 3.4.4.4

$Z = l_a d = 0.94 \times 169$

$= 158.86$

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

$= \dfrac{36.8 \times 10^6}{0.95 \times 410 \times 158.86}$

$= 594.74\ mm^2/m$.

Provide Y12mm @ 175mm c/c.

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

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

$= 0.0013 \times 1000 \times 200$

$= 260\ mm^2/m$.

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

Mid-Span

Since la is less than 0.95, la is O.K.

$Z = l_a d = 0.95 \times 169$

$= 160.55$

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

$= \dfrac{28.19 \times 10^6}{0.95 \times 410 \times 160.55}$

$= 450.79\ mm^2/m$.

Provide Y12mm @ 200mm c/c

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

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

$= 0.0013 \times 1000 \times 200$

$= 260\ mm^2/m$.

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

Provision is o.k.

Long Span.

Continuous Edge

$\beta_{sy}^{-} = 0.045$

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

$= 0.045 \times 21.75 \times 6^2$

$= 35.24\ kN.m$

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

$= 200 - 25 - 12 - 6$

$= 157mm$

$k = \dfrac{35.24 \times 10^6}{25 \times 1000 \times 157^2}$

$= 0.057$

Mid-span.

$\beta_{sy}^{+} = 0.034$.

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

$= 0.034 \times 21.75 \times 6^2$

$= 26.62\ kN.m$.

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

$= 200 - 25 - 12 - 6$

$= 157mm$.

$k = \dfrac{26.62 \times 10^6}{25 \times 1000 \times 157^2}$

$= 0.043$.

Reference	Calculation	Output

Long Span

Continuous Edge	Mid-Span
$l_a = 0.93$	$l_a = 0.95$
Since l_a is less than 0.95 l_a is O.K	Since l_a is less than 0.95 l_a is O.K.
$Z = l_a d = 0.93 \times 157$	$Z = l_a d = 0.95 \times 157$
$= 146.01$	$= 149.15$
$A_{sreq} = \frac{35.24 \times 10^6}{0.95 \times 410 \times 146.01}$	$A_{sreq} = \frac{26.62 \times 10^6}{0.95 \times 410 \times 149.15}$
$= 619.65 \, mm^2/m.$	$= 458.22 \, mm^2/m.$
Provide Y_{12mm} @ 175 mm c/c	Provide Y_{12mm} @ 200 mm c/c
$A_{sprov} = 646 \, mm^2/m$	$A_{sprov} = 566 \, mm^2/m.$
$A_{smin} = 0.13\% bh$	$A_{smin} = 0.13\% bh.$
$= 0.0013 \times 1000 \times 200$	$= 0.0013 \times 1000 \times 200$
$= 260 \, mm^2/m$	$= 260 \, mm^2/m.$
$A_{sprov} > A_{smin}$	$A_{sprov} > A_{smin}.$

Provision is O.K.

Deflection Check

We check for deflection at Short Span — Mid-Span

Table 3.10 equation 8

Service Stress; $f_s = \frac{2}{3} \times f_y \times \frac{A_{sreq}}{A_{sprov}}$

$= 2/3 \times 410 \times \frac{450.79}{566} = 217.70 \, N/mm^2.$

Table 3.10 equation 7

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 $= 1.69 \times 23 = 38.95.$

Actual Span $= \frac{l_x}{d} = \frac{6000}{169} = 35.50$

Since Actual Span is less than Limiting Span, Deflection is OK!!

Deflection is Satisfactory.

REFERENCE	CALCULATIONS	OUTPUTS

PANEL 2 (TWO):

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

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

One short edge discontinous.

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

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

Additional Partition Load:-

$= 3.47 \times 1.4 \times 3.425 \times (6.0 + 6.0) KN$

$= 199.66 KN$

This can be assumed to be distributed over the entire area of 6.0 x 6.0 m.

Hence, the Udl $= \frac{199.66}{6 \times 6} KN/m^2$

$= 5.546 KN/m^2$

Total U.D.L $= 5.55 + 15.36$

$= 20.91 KN/m^2$

SHORT SPAN.

Continous Edge	Mid-Span
$B^-_{sx} = 0.039$	$B^+_{sx} = 0.030$
$M_{sx} = B^-_{sx} n l_x^2$	$M^+_{sx} = B^+_{sx} n l_x^2$
$= 0.039 \times 20.91 \times 6^2$	$= 0.030 \times 20.91 \times 6^2$
$= 29.36 KNm$	$= 22.58 KNm$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144mm$
$K = \frac{M}{f_{cu} b d^2}$	$K = \frac{M}{f_{cu} b d^2}$

REFERENCE	CALCULATIONS		OUTPUT
	SHORT SPAN.		
	Continuous Edge	Mid-Span.	
	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$	
	$= 0.5 + \sqrt{0.25 - \frac{0.061}{0.9}}$	$= 0.5 + \sqrt{0.25 - 0.044/0.9}$	
	$= 0.93$	$= 0.95$	
	$Z = l_a d = 0.93d$	$Z = l_a d = 0.95d$	
	$= 0.93 \times 144 = 133.92$	$= 0.95 \times 144 = 136.8$	
	$As_{req} = \frac{M}{0.95 f_y Z}$	$As_{req} = \frac{M}{0.95 f_y Z}$	
	$= \frac{29.36 \times 10^6}{0.95 \times 410 \times 133.92}$	$= \frac{22.58 \times 10^6}{0.95 \times 410 \times 136.8}$	
	$= 562.86\ mm^2/m$	$= 423.77\ mm^2/m$	
	Provide Y12mm @ 175mm c/c	Provide Y12mm @ 200mm c/c	
	$As_{prov} = 646\ mm^2/m$	$As_{prov} = 566\ mm^2/m$	
	$As_{min} = 0.13\%\ bh$	$As_{min} = 0.13\%\ bh$	
	$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$	
	$= 227.5\ mm^2/m$	$= 227.5\ mm^2/m$	
	Since $As_{prov} > As_{min}$	Since $As_{prov} > As_{min}$	PROVISION IS O.K.

REFERENCE	CALCULATION	OUTPUTS

LONG SPAN

Continous Edge	Mid-Span
$\beta_{sy}^{-} = 0.037$	$\beta_{sy}^{+} = 0.028$
$M_{sy}^{-} = \beta_{sy}^{-} n l_x^2$	$M_{sy}^{+} = \beta_{sy}^{+} n l_x^2$
$= 0.037 \times 20.91 \times 6^2$	$= 0.028 \times 20.91 \times 6^2$
$= 27.85$ kNm	$= 21.07$ kNm
$d = h - c - \phi - \phi/2$	$d = h - c - \phi - \phi/2$
$= 175 - 25 - 12 - 12/6$	$= 175 - 25 - 12 - 12/6$
$= 132$ mm	$= 132$ mm
$K = \frac{27.85 \times 10^6}{25 \times 1000 \times 132^2}$	$K = \frac{21.07 \times 10^6}{25 \times 1000 \times 132^2}$
$= 0.064$	$= 0.048$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.064}{0.9}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.048}{0.9}}$
$= 0.92$	$= 0.94$
$Z = l_a d = 0.92 \times 132$	$Z = l_a d = 0.94 \times 132$
$= 121.44$	$= 124.08$
$As_{req} = \frac{27.85 \times 10^6}{0.95 \times 410 \times 121.44}$	$As_{req} = \frac{21.07 \times 10^6}{0.95 \times 410 \times 124.08}$
$= 588.78$ mm²/m	$= 435.97$ mm²/m
Provide Y12 mm @ 175 mm c/c	Provide Y12 mm @ 200 mm c/c
$As_{prov} = 646$ mm²/m	$As_{prov} = 566$ mm²/m
$As_{min} = 0.13\%\, bh$	$As_{min} = 0.13\%\, bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5$ mm²/m	$= 227.5$ mm²/m
$As_{prov} > As_{min}$	$As_{prov} > As_{min}$

PROVISION IS O.K.

Reference	Calculations	Outputs

Deflection Check.

Deflection is checked for at short-span - mid span.

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

$$f_s = \frac{2}{3} \times 410 \times \frac{423.77}{566} = \frac{347491.4}{1698}$$

$$= 204.65 N/mm^2$$

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

$$mf = 0.55 + \frac{477 - 204.65}{120(0.9 + \frac{22.58 \times 10^6}{1000 \times 144^2})}$$

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

$$m.f = 1.69$$

Limiting Span $= 1.69 \times 26 = 43.94$

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

Since Actual Span is less than Limiting Span, Deflection is O.K.

Deflection O.K.

CALCULATION	OUTPUT

PANEL 3 (THREE).

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

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

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

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

Additional Partition load

$= 3.47 \times 1.4 \times 3.425 \times (6.0 + 4.593) KN$

$= 176.25 KN$.

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

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

$= 6.39 KN/m^2$

Total U.D.L $= 6.4 + 15.36 = 21.76 KN/m^2$

SHORT SPAN.

Continous Edge	Mid-Span
$\beta_{sx}^- = 0.052$	$\beta_x^+ = 0.039$
$M_{sx}^- = \beta_{sx}^- n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.052 \times 21.76 \times 4.593^2$	$= 0.039 \times 21.76 \times 4.593^2$
$= 23.87 KN.m$	$= 17.90 KN.m$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144mm$
$K = \frac{M}{f_{cu} b d^2} = \frac{23.87 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{M}{f_{cu} b d^2} = \frac{17.90 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.05$	$= 0.03$

CALCULATIONS	OUTPUTS

SHORT SPAN

Continuous Edge	Mid Span
We use $l_a = 0.94$	Since $0.96 > 0.95$ we adopt 0.95
$Z = l_a d = 0.94 \times 144 = 135.36$	$Z = l_a d = 0.95 \times 144 = 136.8$
$As_{req} = \frac{M}{0.95 f_y Z}$	$As_{req} = \frac{M}{0.95 f_y Z}$
$= \frac{23.87 \times 10^6}{0.95 \times 410 \times 135.36}$	$= \frac{17.90 \times 10^6}{0.95 \times 410 \times 136.8}$
$= 452.75 mm^2/m$	$= 335.94 mm^2/m$
Provide Y12mm @ 200mm c/c	Provide Y12mm @ 250mm c/c
$As_{prov} = 566 mm^2/m$	$As_{prov} = 452 mm^2/m$
$As_{min} = 0.13\% bh = 0.0013 \times 1000 \times 175 = 227.5 mm^2/m$	$As_{min} = 0.13\% bh = 0.0013 \times 1000 \times 175 = 227.5 mm^2/m$
$As_{prov} > As_{min}$	$As_{prov} > As_{min}$

PROVISION IS O.K.

LONG SPAN

Continuous Edge	Mid-span
$\beta_{sy}^- = 0.037$	$\beta_{sy}^+ = 0.028$
$M_{sy}^- = \beta_{sy}^- n l_x^2$	$M_{sy}^+ = \beta_{sy}^+ n l_x^2$
$= 0.037 \times 21.76 \times 4.593^2$	$= 0.028 \times 21.76 \times 4.593^2$
$= 16.98 KN.m$	$= 12.85 KN.m$
$d = h - c - \phi - \phi/2$	$d = h - c - \phi - \phi/2$
$= 175 - 25 - 12 - 6$	$= 175 - 25 - 12 - 6$
$= 132 mm$	$= 132 mm$
$K = \frac{16.98 \times 10^6}{25 \times 1000 \times 132^2} = 0.04$	$K = \frac{12.85 \times 10^6}{25 \times 1000 \times 132^2} = 0.03$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.04}{0.9}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.03}{0.9}}$

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge

We use $l_a = 0.95$

$Z = l_a d = 0.95 \times 132$

$= 125.4$

$$AS_{req} = \frac{16.98 \times 10^6}{0.95 \times 410 \times 125.4}$$

$= 347.6\,mm^2/m$

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

Since $0.96 > 0.95$

We adopt 0.95

$Z = l_a d = 0.95 \times 132$

$= 125.4$

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

$= 263.08\,mm^2/m$

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 OK.

DEFLECTION CHECK:- We check for deflection on the Short-span mid-span.

Service Stress, $f_s = \frac{2}{3} \times f_y \times \frac{AS_{req}}{AS_{prov}} = \frac{2}{3} \times 410 \times \frac{335.94}{452}$

$= 203.15\,N/mm^2$

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

$$\frac{M}{bd^2} = \frac{17.90 \times 10^6}{1000 \times 144^2} = 0.86$$

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

in 5

REFERENCE	CALCULATION	OUTPUT
	Limiting span $= 1{\cdot}85 \times 26 = 48{\cdot}1$	
	Actual span $= {}^{lx}/_{d} = \frac{4593}{144} = 31{\cdot}9$	
	Since Actual span is less than Limiting span, Deflection is OK.	DEFLECTION IS OK.

REFERENCE	CALCULATION	OUTPUT

PANEL 4(FOUR)

$$l_y/l_x = \frac{2590}{1050} = 2.5$$

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

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

$n = 15.36 kN/m^2$ $b = 1000mm$

SUPPORT MOMENT

Moment @ Support $= \frac{9wl^2}{128} = 1.19 kNm$

$d = 175-25-12/6 = 144$

$$K = \frac{M}{f_{cu}bd^2} = \frac{1.19\times10^6}{25\times1000\times144^2} = 0.003$$

$$la = 0.5 + \sqrt{0.25 - \frac{0.003}{0.9}} = 0.99$$

Use $la = 0.95$

$Z = lad = 0.95\times144 = 136.8$

$$As_{req} = \frac{M}{0.95f_yZ} = \frac{1.19\times10^6}{0.95\times410\times136.8} = 22.3mm^2/m$$

$As_{min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5mm^2/m$

Since $As_{min} > As_{req}$, We use As_{min}

Provide Y10mm @ 300mm c/c

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

PROVISION IS O.K.

REFERENCE	CALCULATION	OUTPUT

SPAN MOMENT

Span Moment $= \frac{wl^2}{12} = \frac{15.36 \times 1.05^2}{12} = 1.4\,kN.m$

$d = 175 - 25 - 12/2 = 144$

$K = \frac{M}{f_{cu} b d^2} = \frac{1.4 \times 10^6}{25 \times 1000 \times 144^2} = 0.003$

$la = 0.5 + \sqrt{0.25 - \frac{0.003}{0.9}} = 0.99$

Since $la > 0.95$ Use $la = 0.95$

$Z = la\,d = 0.95 \times 144$

$= 136.8$

$AS_{req} = \frac{1.4 \times 10^6}{0.95 \times 410 \times 136.8} = 26.27\,mm^2/m$

$AS_{min} = 0.0013bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5\,mm^2/m$

Since $AS_{min} > AS_{req}$, We adopt AS_{min}

Provide Y12mm @ 300c/c

$AS_{prov} = 377\,mm^2/m$ — PROVISION O.K.

DEFLECTION CHECK: We check for deflection at the Mid-Span.

Service Stress $f_s = \frac{2}{3} f_y \frac{As_{req}}{As_{prov}} = \frac{2}{3} \times 410 \times \frac{227.5}{377}$

$= 164.9\,N/mm^2$

$m.f = 0.55 + \frac{477 - 164.9}{120\left(0.9 + \frac{1.4 \times 10^6}{1000 \times 144^2}\right)}$ $\left[0.55 + \frac{477 - f_s}{120(0.9 + M/bd^2)}\right]$

$m.f = 3.2$; Since $m.f > 2.0$, we use 2.

Limiting Span $= 2 \times 26 = 52$

Actual Span $= l/d = 1050 / 144 = 7.3$ — DEFLECTION

PANEL 5(F to E)

$l_y/l_{sx} = \frac{6000}{6000} = 1$

Since $l_y/l_{sx} < 2$, the slab will be designed as a two-way spanning slab.

$n = 15.36 kN/m^2$ $h = 175mm$

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

SHORT SPAN

Continous Edge	Mid-Span
$\beta_{sx}^- = 0.039$	$\beta_{sx}^+ = 0.029$
$M_{sx}^- = \beta_{sx}^- n l_{sx}^2$	$M_{sx}^+ = \beta_{sx}^+ n l_{sx}^2$
$= 0.039 \times 15.36 \times 6^2$	$= 0.029 \times 15.36 \times 6^2$
$= 21.57 kN.m$	$= 16.04 kN.m$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144m$
$K = \frac{M}{f_{cu}bd^2} = \frac{21.57 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{M}{f_{cu}bd^2} = \frac{16.04 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.04$	$= 0.03$
$Z = l_a d$	$Z = l_a d$
$l_a = \sqrt{0.25 - K/0.9} + 0.5$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.03}{0.9}}$
$= 0.5 + \sqrt{0.25 - \frac{0.04}{0.9}}$	$= 0.96$
$= 0.95$	Use $l_a = 0.95$ Since $0.96 > 0.95$
$Z = l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8$	$= 136.8$
$As_{req} = \frac{M}{0.95 f_y Z}$	$As_{req} = \frac{M}{0.95 f_y Z}$
21.57×10^6	$= 16.04 \times 10^6$

REFERENCE	CALCULATION	OUTPUT

SHORT SPAN

Continuous Edge	Mid - Span
$As_{min} = 0.0013 \times 1000 \times 175$ $= 227.5 mm^2/m$	$As_{min} = 0.0013 \times 1000 \times 175$ $= 227.5 mm^2/m$
Since $As_{min} < As_{req}$; We use As_{req} to provide	Since $As_{min} < As_{req}$; We use As_{req} to provide
Provide Y12mm @ 250mm c/c $As_{prov} = 452 mm^2/m$	Provide Y12mm @ 300mm c/c $As_{prov} = 377 mm^2/m$

PROVISION IS OK.

LONG SPAN.

Continous Edge	Mid - Span.
$\beta_{sy}^{-} = 0.037$	$\beta_{sy}^{+} = 0.028$
$M_{sy}^{-} = \beta_{sy}^{-} n l x^2$ $= 0.037 \times 15.36 \times 6^2$ $= 20.46 kN.m$	$M_{sy}^{+} = \beta_{sy}^{+} n l x^2$ $= 0.028 \times 15.36 \times 6^2$ $= 15.48 kN.m$
$d = h - c - \phi/_2 - \phi$ $= 175 - 25 - 6 - 12$ $= 132 mm$	$d = h - c - \phi/_2$ $= 175 - 25 - 6 - 12$ $= 132 mm$
$K = \frac{20.46 \times 10^6}{25 \times 1000 \times 132^2} = 0.05$	$K = \frac{15.48 \times 10^6}{25 \times 1000 \times 132^2} = 0.04$
$la = 0.5 + \sqrt{0.25 - 0.05/0.9}$ $= 0.94$	$la = 0.5 + \sqrt{0.25 - \frac{0.04}{0.9}}$ $= 0.95$
$Z = la\, d = 0.94 \times 132$ $= 124.08$	$Z = la\, d = 0.95 \times 132$ $= 125.4$
$A_s = 20.46 \times 10^6$ ...	$A_s = \frac{15.48 \times 10^6}{\ldots} = 316.63$

[illegible]

REFERENCE	CALCULATION	OUTPUT.

LONG SPAN

Continuous Edge	Mid-Span
$AS_{min} = 0.0013bh$	$AS_{min} = 0.0013bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5 mm^2/m$	$= 227.5 mm^2/m$
Since $AS_{min} < AS_{req}$ Use AS_{req} for Provision,	Since $AS_{min} < AS_{req}$ Use AS_{req} for Provision.
Provide Y12mm @ 200mm c/c	Provide Y12mm @ 300mm c/c
$AS_{prov} = 566 mm^2/m$	$AS_{prov} = 377 mm^2/m$

Provision :: O.K.

DEFLECTION CHECK :- We check for deflection at ~~the short~~ span Mid span.

Service Stress $f_s = \frac{2}{3} \times f_y \times \frac{As_{req}}{As_{prov}} = \frac{2}{3} \times 410 \times \frac{301.03}{377}$

$= 213.46 N/mm^2$

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

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

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

Limiting Span $= 1.87 \times 26 = 48.62$

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

Since Limiting Span > Actual Span

Deflection is OK.

Deflection ...

PANEL 6 (SIX)

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

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

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

$b = 1000mm$ $n = 15.36 kN/m^2$

SHORT SPAN

Continous Edge	Mid - Span
$\beta_{sx}^- = 0.031$	$\beta_{sx}^+ = 0.024$
$M_{sx}^- = \beta_{sx}^- n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.031 \times 15.36 \times 6^2$	$= 0.024 \times 15.36 \times 6^2$
$= 17.14 kN.m$	$= 13.27 kN.m$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144mm$
$K = \frac{M}{f_{cu} b d^2} = \frac{17.14 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{M}{f_{cu} b d^2} = \frac{13.27 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.03$	$= 0.03$
$l_a = 0.5 + \sqrt{0.25 - K/0.9}$	$l_a = 0.5 + \sqrt{0.25 - K/0.9}$
$l_a = 0.5 + \sqrt{0.25 - 0.03/0.9}$	$l_a = 0.5 + \sqrt{0.25 - 0.03/0.9}$
$= 0.96$	$= 0.96$
Use 0.95 as la since 0.96 > 0.95	Use 0.95 as la since 0.96 > 0.95
$Z = l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8$	$= 136.8$
$As_{req} = \frac{M}{0.95 f_y Z}$	$As_{req} = \frac{M}{0.95 f_y Z}$
17.14×10^6	13.27×10^6

CALCULATION

SHORT SPAN.

Continous Edge	Mid-Span
$As_{req} = 321.67 mm^2/m$	$As_{req} = 249.04 mm^2/m$
Provide Y12mm @ 300mm c/c	Provide Y12mm @ 300mm c/c
$As_{prov} = 377 mm^2/m$	$As_{prov} = 377 mm^2/m$
$As_{min} = 0.13\% bh$	$As_{min} = 0.13\% bh$
$= 0.0013 \times 1000 \times 175$	$= 0.0013 \times 1000 \times 175$
$= 227.5 mm^2/m$	$= 227.5 mm^2/m$
$As_{prov} > As_{min}$	$As_{prov} > As_{min}$

Provision is O.K.

LONG SPAN.

Continous Edge	Mid-Span
$\beta_{sy}^- = 0.032$	$\beta_{sy}^+ = 0.024$
$M_{sy}^- = \beta_{sy}^- n l_x^2$	$M_{sy}^+ = \beta_{sy}^+ n l_x^2$
$= 0.032 \times 15.36 \times 36$	$= 0.024 \times 15.36 \times 36$
$= 17.69 kN.m$	$= 13.27 kN.m$
$d = h - c - \phi - \phi/2$	$d = h - c - \phi - \phi/2$
$= 175 - 25 - 12 - 6$	$= 175 - 25 - 12 - 6$
$= 132mm$	$= 132m$
$K = \frac{17.69 \times 10^6}{25 \times 1000 \times 132^2}$	$K = \frac{13.27 \times 10^6}{25 \times 1000 \times 132^2}$
$= 0.04$	$= 0.03$
$la = 0.5 + \sqrt{0.25 - K/0.9}$	$la = 0.5 + \sqrt{0.25 - K/0.9}$
$= 0.95$	$= 0.96$
Use $la = 0.95$	Use $la = 0.95$ since $0.96 > 0.95$
$Z = lad = 0.95 \times 132$	$Z = lad = 0.95 \times 132$
$= 125.4$	$= 125.4$

REFERENCE	CALCULATION	OUTPUT
	LONG SPAN	

Continuous Edges	Mid - Span
$As_{req} = \frac{17.69 \times 10^6}{0.95 \times 410 \times 125.4}$	$As_{req} = \frac{13.27 \times 10^6}{0.95 \times 410 \times 125.4}$
$= 362.18 mm^2/m$	$= 271.69 mm^2/m$
Provide Y12 @ 250mm c/c	Provide Y12m @ 300mm c/c
$As_{prov} = 452 mm^2/m$	$As_{prov} = 377$
$As_{min} = 227.5 mm^2/m$ (0.13% bh)	$As_{min} = 227.5 mm^2/m$ (0.13% bh)
$As_{prov} > As_{min}$	$As_{prov} > As_{min}$

PROVISION IS OK.

DEFLECTION CHECK. - We check for deflection at the short span - mid-span.

Service stress $f_s = \frac{2}{3} \times f_y \times \frac{As_{req}}{As_{prov}} = \frac{2}{3} \times 410 \times \frac{247.09}{377}$

$= 180.6 N/mm^2$

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

$\frac{M}{bd^2} = \frac{13.27 \times 10^6}{1000 \times 144^2} = 0.64$

$m.f = 0.55 + \frac{477 - 180.6}{120(0.9 + 0.64)} = 2.15$

$m.f = 2$

Limiting Span $= 2 \times 26 = 52$

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

Since Actual Span is less than limiting Span Deflection is O K.

DEFLECTION ..

REFERENCE	CALCULATION	OUTPUT

SHORT SPAN.

Continous Edge

$As_{min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

$As_{prov} > As_{min}$

Mid-Span

$As_{min} = 0.13\% bh$

$= 0.0013 \times 1000 \times 175$

$= 227.5 mm^2/m$

$As_{prov} > As_{min}$

PROVISION IS O.K

LONG SPAN

Continous Edge

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

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

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

$= 17.69 kN.m$

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

$= 175 - 25 - 12 - 6$

$= 132 mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{17.69 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.04$

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

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

$= 0.95$

Use la 0.95 since La = 0.95

$Z = la\,d = 0.95d = 0.95 \times 132$

$= 125.4$

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

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

Mid-Span

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

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

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

$= 13.27 kN.m$

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

$= 175 - 25 - 12 - 6$

$= 132 mm$

$K = \frac{M}{f_{cu} b d^2} = \frac{13.27 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.03$

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

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

$= 0.96$

Use la = 0.95, Since 0.96 > 0.95

$Z = la\,d = 0.95 \times 132$

$= 125.4$

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

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

REFERENCE	CALCULATION		OUTPUT
	Continous Edge	Mid-Span	
	Provide Y12 @ 300mm c/c $As_{prov} = 377mm^2/m$ $As_{min} = 0.13\% bh$ $= 0.13\% \times 1000 \times 175$ $= 227.5mm^2/m$ $As_{prov} > As_{min}$	Provide Y12 @ 300mm c/c $As_{prov} = 377mm^2/m$ $As_{min} = 0.13\% bh$ $= 0.0013 \times 1000 \times 175$ $= 227.5mm^2/m$ $As_{prov} > As_{min}$	PROVISION IS O.K.

DEFLECTION CHECK.

We check for deflection at the short-span mid span

Service Stress, $f_s = \frac{2}{3} \times f_y \times \frac{As_{req}}{As_{prov}} = \frac{2}{3} \times 410 \times \frac{332}{377}$

$= 240.7 N/mm^2$

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

$\frac{M}{bd^2} = \frac{17.69 \times 10^6}{1000 \times 144^2} = 0.85$

$m.f = 0.55 + \frac{477 - 240.7}{120(0.9 + 0.85)} = 1.67$

$m.f = 1.67$

Limiting Span $= 1.67 \times 26 = 43.42$

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

Since Actual span < limiting span.
Deflection is O.K

DEFLECTION IS O.K.

SOLID SLAB

DESIGN TO EUROCODE

PANEL 1:

Aspect ratio

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

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

$C = 25mm$, $h = 200mm$, $b = 1000mm$.

Additional Partition load:

$= 3.47 \times 1.35 \times 3.425 \times (6 + 6)$ KN

$= 192.53$ KN

This can be assumed to be distributed over the entire area of 6 X 6 m.

Hence, the U.D.L $= \frac{192.53}{6 \times 6}$ KN/m²

$= 5.35$ KN/m²

Total U.D.L $= 5.35 + 15.45 = 20.8$ KN/m².

SHORT SPAN

Continuous Edge	Midspan
$B_{sx}^- = 0.047$	$B_{sx}^+ = 0.036$
$M_{sx}^- = B_{sx}^- n l_x^2$	$M_{sx}^+ = B_{sx}^+ n l_x^2$
$= 0.047 \times 20.8 \times 6^2$	$= 0.036 \times 20.8 \times 6^2$
$= 35.19$ KNm	$= 26.96$ KNm
$d = h - c - \phi/2$	$d = 200 - 25 - 12/2$
$= 200 - 25 - 12/2$	$= 169mm$
$= 169mm$	
$K = \frac{M}{f_{ck} b d^2} = \frac{35.19 \times 10^6}{25 \times 1000 \times 169^2}$	$K = \frac{M}{f_{ck} b d^2} = \frac{26.96 \times 10^6}{25 \times 1000 \times 169^2}$
$= 0.049$	$= 0.038$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.95$	$= 0.96$
Since $l_a = 0.95$, l_a is o.k	Since $l_a > 0.95$, $l_a = 0.95$
$Z = l_a d = 0.95 \times 169$	$Z = l_a d = 0.95 \times 169$
$= 160.55mm$	$= 160.55mm$
$As_{req} = \frac{M}{0.87 f_{yk} Z}$	$As_{req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{35.19 \times 10^6}{0.87 \times 410 \times 160.55} = 614.48mm^2/m$	$= \frac{26.96 \times 10^6}{0.87 \times 410 \times 160.55} = 470mm^2/m$
Provide Y12mm @ 175mm c/c	Provide Y12mm @ 200mm c/c
$As_{prov} = 646mm^2/m$	$As_{prov} = 566mm^2/m$
$As_{min} = 0.0013bd$	$As_{min} = 0.0013bd$
$= 0.0013 \times 1000 \times 169$	$= 0.0013 \times 1000 \times 169$

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge

$\beta^-_{sy} = 0.045$

$M^-_{sy} = \beta^-_{sy} n l_x^2$

$= 0.045 \times 20.8 \times 6^2$

$= 33.70\,kNm$

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

$= 200 - 25 - 6 - 12$

$= 157\,mm$

$K = \dfrac{33.7 \times 10^6}{25 \times 1000 \times 157^2}$

$= 0.055$

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

$= 0.95$

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

$Z = l_a d = 0.95 \times 157 = 149.15\,mm$

$As_{req} = \dfrac{M}{0.87 f_{yk} Z}$

$= \dfrac{33.7 \times 10^6}{0.87 \times 410 \times 149.15}$

$= 633.14\,mm^2/m$

$As_{min} = 0.0013bd$

$= 0.0013 \times 1000 \times 157$

$= 204.1\,mm^2/m$

Provide Y12mm @ 175mm c/c

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

Since $As_{prov} > As_{min}$

Mid Span

$\beta^+_{sy} = 0.034$

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

$= 0.034 \times 20.8 \times 6^2$

$= 25.46\,kNm$

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

$= 200 - 25 - 12 - 6$

$= 157\,mm$

$K = \dfrac{25.46 \times 10^6}{25 \times 1000 \times 157^2}$

$= 0.041$

$l_a = 0.5 + \sqrt{0.25 - 0.041/0.9}$

$= 0.95$

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

$Z = l_a d = 149.15\,mm$

$As_{req} = \dfrac{M}{0.87 f_{yk} Z}$

$= \dfrac{25.46 \times 10^6}{0.87 \times 410 \times 149.15}$

$= 478.56\,mm^2/m$

$As_{min} = 0.0013bd$

$= 204.1\,mm^2/m$

Provide Y12mm @ 200mm c/c

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

Since $As_{prov} > As_{min}$

PROVISION IS O.K

DEFLECTION CHECK

We check for deflection at short span mid-span.

Service stress, σ_s, $= \dfrac{5}{8} \times f_{yk} \times \dfrac{As_{req}}{As_{prov}}$

$= \dfrac{5}{8} \times 410 \times \dfrac{470.77}{566} = 213.14\,N/mm^2$

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

Interpolating to get the basic span-depth ratio,

$0.5 - 28$

$0.28 - x$

REFERENCE	CALCULATIONS	OUTPUT
	$\frac{0.28 - 0.5}{0.15 - 0.5} = \frac{x - 29}{38 - 28}$	
	$x = 34.29$	
	Limiting span-depth ratio $= 1.45 \times 34.29$	
	$= 49.72$	
	Actual span-depth ratio $= \frac{l_a}{d} = \frac{6000}{169} = 35.5$	
	Since actual span-depth ratio < 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.

$C = 25mm$, $h = 175mm$, $b = 1000mm$.

Additional Partition load:

$= 3.47 \times 1.35 \times 3.425 \times (6+6)\ KN$

$= 192.53KN$.

This can be assumed to distributed over the entire area of 6m X 6m.

Hence, the U.D.L = $\frac{192.53}{6 \times 6}$ KN/m²

$= 5.35KN/m^2$

Total U.D.L $= 5.35 + 14.64$

$= 19.99KN/m^2$.

SHORT SPAN

Continuous Edge

$\beta^-_{sx} = 0.039$

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

$= 0.039 \times 19.99 \times 6^2$

$= 28.07KNm$

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

$= 175 - 25 - 12/2$

$= 144mm$

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

$= \frac{28.07 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.054$

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

$= 0.95$

Since $l_a = 0.95$, l_a is o.k

$Z = l_a d = 0.95 \times 144$

$= 136.8mm$

$As_{req} = \frac{M}{0.87 f_{yk} Z}$

$= \frac{28.07 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 575.05$ ...

Midspan

$\beta^+_{sx} = 0.029$

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

$= 0.029 \times 19.99 \times 6^2$

$= 20.87KNm$

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

$= 175 - 25 - 12/2$

$= 144mm$

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

$= \frac{20.87 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.040$

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

$= 0.96$

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

$Z = l_a d = 0.95 \times 144$

$= 136.8mm$

$As_{req} = \frac{M}{0.87 f_{yk} Z}$

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

$=$ ...

REFERENCES	CALCULATIONS	OUTPUT

SHORT SPAN

Continuous Edge	Mid span
Provide Y12mm @ 175mm c/c	Provide Y12mm @ 200mm c/c
$A_{s prov} = 646 mm^2/m$	$A_{s prov} = 566 mm^2/m$
$A_{s min} = 0.13\% bd$	$A_{s min} = 0.13\% bd$
$A_{s min} = 0.0013 \times 1000 \times 144$	$= 0.0013 \times 1000 \times 144$
$= 187.2 mm^2/m$	$= 187.2 mm^2/m$
Since $A_{s prov} > A_{s min}$	Since $A_{s prov} > A_{s min}$

PROVISION IS O.K

LONG SPAN

Continuous Edge	Mid span
$\beta_{sy}^- = 0.037$	$\beta_{sy}^+ = 0.028$
$M_{sy}^- = \beta_{sy}^- n l_x^2$	$M_{sy}^+ = \beta_{sy}^+ n l_x^2$
$= 0.037 \times 19.99 \times 6^2$	$= 0.028 \times 19.99 \times 6^2$
$= 26.63 KNm$	$= 20.15 KNm$
$d = h - c - \phi/2 - \phi$	$d = h - c - \phi/2 - \phi$
$= 175 - 25 - 12 - 6$	$= 175 - 25 - 12 - 6$
$= 132mm$	$= 132mm$.
$K = \frac{M}{f_{ck} b d^2}$	$K = \frac{M}{f_{ck} b d^2}$
$= \frac{26.63 \times 10^6}{25 \times 1000 \times 132^2}$	$= \frac{20.15 \times 10^6}{25 \times 1000 \times 132^2}$
$= 0.06$	$= 0.046$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.06}{1.134}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.046}{1.134}}$
$= 0.94$	$= 0.96$
Since $l_a = 0.94 < 0.95$, l_a is o.k	Since $l_a = 0.96 > 0.95$ we use $l_a = 0.95$
$Z = l_a d = 0.95 \times 132$	$Z = l_a d = 0.95 \times 132$
$= 125.4mm$	$= 125.4 mm$
$A_{s req} = \frac{M}{0.87 f_{yk} Z}$	$A_{s req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{26.63 \times 10^6}{0.87 \times 410 \times 125.4}$	$= \frac{20.15 \times 10^6}{0.87 \times 410 \times 125.4}$
$= 595.35 mm^2/m$	$= 450.48 mm^2/m$
Provide Y12mm @ 175mm c/c	Provide Y12mm @ 200mm c/c
$A_{s prov} = 646 mm^2/m$	$A_{s prov} = 566 mm^2/m$
$A_{s min} = 0.0013bd$	$A_{s min} = 0.0013bd$
$= 0.0013 \times 1000 \times 132$	$= 0.0013 \times 1000 \times 132$
$= 171.6 mm^2/m$	$= 171.6 mm^2/m$
Since $A_{s prov} > A_{s min}$	Since $A_{s prov} > A_{s min}$

PROVISION IS O.K

REFERENCE	CALCULATIONS	OUTPUT
	DEFLECTION CHECK	
	We check for deflection at the short-span mid span.	
	Service stress, $\sigma_s = \frac{5}{8} \times f_{yk} \times \frac{As_{req}}{As_{prov}}$	
	$= \frac{5}{8} \times 410 \times \frac{427.69}{566}$	
	$= 193.63\ N/mm^2$.	
	Modification factor $= \frac{310}{\sigma_s} = \frac{310}{193.63} = 1.6$	
	Interpolating to get the basic span-depth ratio	
	0.5 – 28	
	0.28 – x	
	0.15 – 38	
	$\frac{0.28 - 0.5}{0.15 - 0.5} = \frac{x - 28}{38 - 28}$	
	$x = 34.30$.	
	∴ Limiting span-depth ratio $= 1.6 \times 34.3$	
	$= 54.88$	
	Actual span depth ratio $= \frac{L_x}{d} = \frac{6000}{144} = 41.67$	
	Since actual span-depth ratio is less than the limiting span-depth ratio, deflection is O.K	DEFLECTION IS SATISFACTORY.

REFERENCE	CALCULATIONS	OUTPUT

PANEL 3:

Aspect ratio

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

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:

$= 3.47 \times 1.35 \times 3.425 \times (6.0 + 4.593) kN$

$= 169.96 kN$

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

Hence, the U.D.L $= \frac{169.96}{6 \times 4.593} = 6.17 kN/m^2$.

Total U.D.L $= 6.17 + 14.64 = 20.81 kN/m^2$

SHORT SPAN

Continuous Edge	Mid span
$\beta_{sx}^- = 0.052$	$\beta_{sx}^+ = 0.039$
$M_{sx}^- = \beta_{sx}^- n l_x^2$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$
$= 0.052 \times 20.81 \times 4.593^2$	$= 0.039 \times 20.81 \times 4.593^2$
$= 22.83 kNm$	$= 17.12 kNm$
$d = h - c - \phi/2$	$d = h - c - \phi/2$
$= 175 - 25 - 12/2$	$= 175 - 25 - 12/2$
$= 144mm$	$= 144mm$
$K = \frac{M}{f_{ck} bd^2} = \frac{22.83 \times 10^6}{25 \times 1000 \times 144^2}$	$K = \frac{M}{f_{ck} bd^2} = \frac{17.12 \times 10^6}{25 \times 1000 \times 144^2}$
$= 0.044$	$= 0.033$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.96$	$= 0.97$
Since $l_a = 0.96 > 0.95$, we use $l_a = 0.95$	Since $l_a = 0.97 > 0.95$, we use $l_a = 0.95$
$Z = l_a d = 0.95 \times 144$	$Z = l_a d = 0.95 \times 144$
$= 136.8mm$	$= 136.8mm$
$As_{req} = \frac{M}{0.87 f_{yk} Z}$	$As_{req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{22.83 \times 10^6}{0.87 \times 410 \times 136.8}$	$= \frac{17.12 \times 10^6}{25 \times 1000 \times 136.8}$
$= 467.86 mm^2/m$	$= 350.84 mm^2/m$
Provide Y12mm @ 200mm c/c	Provide Y12mm @ 250mm c/c
$As_{prov} = 566 mm^2/m$	$As_{prov} = 452 mm^2/m$
$As_{min} = 0.13\% bd$	$As_{min} = 0.13\% bd$
$= 0.0013 \times 1000 \times 144$	$= 0.0013 \times 1000 \times 144$

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge

$\beta^-_{sy} = 0.037$

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

$= 0.037 \times 20.81 \times 4.593^2$

$= 16.24 kNm$

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

$= 175 - 25 - 12 - 6$

$= 132mm$

$K = \frac{16.24 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.037$

$l_a = 0.5 + \sqrt{0.25 - \frac{0.037}{1.134}}$

$= 0.97$

Since $l_a = 0.97 > 0.95$; we use $l_a = 0.95$

$Z = l_a d = 0.95 \times 132$

$= 125.4mm$

$As_{req} = \frac{M}{0.87 f_{yk} Z}$

$= \frac{16.24 \times 10^6}{0.87 \times 410 \times 125.4}$

$= 363.07 mm^2/m$

Provide Y12mm @ 250mm c/c

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

$As_{min} = 0.0013bd$

$= 0.0013 \times 132 \times 1000$

$= 171.6mm^2/m$

Since $As_{prov} > As_{min}$

Mid span

$\beta^+_{sy} = 0.028$

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

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

$= 12.29 kN.m$

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

$= 175 - 25 - 12 - 6$

$= 132mm$

$K = \frac{12.29 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.028$

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

$= 0.97$

Since $l_a = 0.97 > 0.95$; we use $l_a = 0.95$

$Z = l_a d = 0.95 \times 132$

$= 125.4mm.$

$As_{req} = \frac{M}{0.87 f_{yk} Z}$

$= \frac{12.29 \times 10^6}{0.87 \times 410 \times 125.4}$

$= 274.76 mm^2/m$

Provide Y12mm @ 300mm c/c

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

$As_{min} = 0.0013bd$

$= 0.0013 \times 132 \times 1000$

$= 171.6mm^2/m$

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

DEFLECTION CHECK

We check for deflection on the short-span midspan

Service stress, $\sigma_s = \frac{5}{8} \times f_{yk} \times \frac{As_{req}}{As_{prov}}$

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

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

Interpolating to obtain basic span-depth ratio:

0.5 - 28

0.24 - x

0.15 - 38

$\frac{0.24 - 0.5}{...} = \frac{x - 28}{...}$

REFERENCE	CALCULATIONS	OUTPUT
	$\therefore$ Limiting span-depth ratio $= 1.56 \times 35.43$ $= 55.27$	
	Actual span-depth ratio $= \frac{4593}{144} = 31.895$	
	Since actual span-depth ratio is less than limiting span-depth ratio, deflection is O.K.	DEFLECTION IS SATISFACTORY.

REFERENCE	CALCULATIONS	OUTPUT
	PANEL 4.	
	Aspect ratio $l_y/l_x = \frac{2590}{1050} = 2.5$	
	Since $l_y/l_x > 2$, the slab is a one-way spanning slab.	
	$C = 25mm$, $h = 175mm$	
	$n = 14.64 KN/m^2$, $b = 1000mm$	
	SUPPORT MOMENT	
	Moment @ support $= \frac{9wl^2}{128} = \frac{9 \times 14.64 \times 1.05^2}{128}$ $= 1.13 KNm$	
	$d = h - c - \phi/2$ $= 175 - 25 - 12/6 = 144mm$	
	$K = \frac{M}{f_{cu}bd^2} = \frac{1.13 \times 10^6}{25 \times 1000 \times 144^2} = 0.002$	
	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$ $= 0.99$	
	Since $l_a > 0.95$ ∴ use $l_a = 0.95$	
	$Z = l_a d = 0.95 \times 144$ $= 136.8 mm$	
	$A_{s req} = \frac{M}{0.87 \times f_y \times Z} = \frac{1.13 \times 10^6}{0.87 \times 410 \times 136.8}$ $= 23.15 mm^2/m$	
	$A_{s min} = 0.13\% bd$ $= 0.0013 \times 1000 \times 144$ $= 187.2 mm^2/m$	
	Since $A_{s min} > A_{s req}$, we use $A_{s min}$	
	Provide Y12mm @ 300mm c/c	
	$A_{s prov} = 377 mm^2/m$	PROVISION IS O.K
	SPAN MOMENT	
	Moment @ span $= \frac{wl^2}{12} = \frac{14.64 \times 1.05^2}{12} = 1.3 KNm$	
	$d = h - c - \phi/2 = 175 - 25 - 12/2$ $= 144mm$	
	$K = \frac{M}{f_{cu}bd^2} = \frac{1.3 \times 10^6}{25 \times 1000 \times 144^2} = 0.003$	
	$l_a = 0.5 + \sqrt{0.25 - 0.003/1.134}$	
	Use $l_a = 0.95$, since $0.99 > 0.95$	
	$Z = l_a d = 0.95d = 0.95 \times 144$ $= 136.8mm$	

REFERENCE	CALCULATIONS	OUTPUT
	$A_{s\,req} = \frac{M}{0.87 \times f_{yk} \times z} = \frac{1.3 \times 10^6}{0.87 \times 410 \times 136.8} = 26.64\,mm^2/m$	
	$A_{s\,min} = 0.13\%\,bd$	
	$= 0.0013 \times 1000 \times 144$	
	$= 187.2\,mm^2/m$	
	Since $A_{s\,min} > A_{s\,req}$, we use $A_{s\,min}$	
	Provide Y12mm @ 300mm c/c.	
	$A_{s\,prov} = 377\,mm^2/m$	PROVISION IS O.K
	DEFLECTION CHECK	
	At the mid span	
	Service stress; $\sigma_s = \frac{5}{8} \times f_{yk} \times \frac{A_{s\,req}}{A_{s\,prov}}$	
	$= \frac{5}{8} \times 410 \times \frac{187.2}{377}$	
	$= 127.24\,N/mm^2$	
	Modification factor $= \frac{310}{\sigma_s} = \frac{310}{127.24} = 2.4$	
	Since M.F > 2, we use M.F $= 2$.	
	Limiting span-depth ratio $= 2 \times$ basic ratio;	
	$A_s/bd \times 100\% = \frac{187.2}{100 \times 144} \times 100\% = 0.13$	
	Interpolating to obtain basic span ratio (interior span)	
	$0.5 - 30$	
	$0.13 - x$	
	$0.15 - 33$	
	$\frac{x - 30}{33 - 30} = \frac{0.13 - 0.5}{0.15 - 0.5}$	
	$x = 33.17$	
	Limiting span $= 2 \times 33.17$	
	$= 66.34$	
	Actual span-depth ratio $= l_x/d = \frac{1050}{144} = 7.3$	
	Since actual span-depth ratio is less than the limiting span-depth ratio, deflection is O.K.	DEFLECTION IS O.K.

REFERENCE	CALCULATIONS	OUTPUT

PANEL 5:

Aspect ratio:

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

Since aspect ratio is less than 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

$\beta_{sx}^- = 0.039$

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

$= 0.039 \times 14.64 \times 6^2$

$= 20.55 KNm$

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

$= 175 - 25 - 12/2$

$= 144mm$

$$K = \frac{M}{f_{cu} bd^2} = \frac{20.55 \times 10^6}{25 \times 1000 \times 144^2}$$

$= 0.04$

$Z = l_a d$

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

$= 0.5 + \sqrt{0.25 - 0.04/1.134}$

$= 0.96$

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

$Z = l_a d = 0.95 \times 144$

$= 136.8 mm$

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

$$= \frac{20.55 \times 10^6}{0.87 \times 410 \times 136.8}$$

$= 421.1 mm^2/m$

$A_{s\,min} = 0.0013 bd$

$= 0.0013 \times 1000 \times 144$

$= 187.2 mm^2/m$

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

Provide Y12mm @ 250mm c/c

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

Mid span

$\beta_{sx}^+ = 0.029$

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

$= 0.029 \times 14.64 \times 6^2$

$= 15.28 KNm$

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

$= 175 - 25 - 12/2$

$= 144mm$

$$K = \frac{15.28 \times 10^6}{25 \times 1000 \times 144^2}$$

$= 0.03$

$Z = l_a d$

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

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

$= 0.97$

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

$Z = l_a d = 0.95 \times 144$

$= 136.8mm$

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

$$= \frac{15.28 \times 10^6}{0.87 \times 410 \times 136.8}$$

$= 313.1 mm^2/m$

$A_{s\,min} = 0.0013 bd$

$= 0.0013 \times 1000 \times 144$

$= 187.2 mm^2/m$

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

Provide Y12mm @ 300mm c/c

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

PROVISION IS OK

CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge	Mid span
$\beta_{sy}^{-} = 0.037$	$\beta_{sy}^{+} = 0.028$
$M_{sy}^{-} = \beta_{sy}^{-} n l_x^2$	$M_{sy}^{+} = \beta_{sy}^{+} n l_x^2$
$= 0.037 \times 14.64 \times 6^2$	$= 0.028 \times 14.64 \times 6^2$
$= 19.5\,kNm$	$= 14.76\,kNm$
$d = h - c - \phi/2 - \phi$	$d = h - c - \phi/2 - \phi$
$= 175 - 25 - 12/2 - 12$	$= 175 - 25 - 12/2 - 12$
$= 132mm$	$= 132mm$
$K = \frac{M}{f_{ck} b d^2}$	$K = \frac{M}{f_{ck} b d^2}$
$= \frac{19.5 \times 10^6}{25 \times 1000 \times 132^2} = 0.04$	$= \frac{14.76 \times 10^6}{25 \times 1000 \times 132^2} = 0.03$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$	$l_a = 0.5 + \sqrt{0.25 - K/1.134}$
$= 0.96$	$= 0.97$
Since $l_a > 0.95$, we use	Since $l_a > 0.95$, we use
$l_a = 0.95$	$l_a = 0.95$
$Z = l_a d = 0.95 \times 132$	$Z = l_a d = 0.95 \times 132$
$= 125.4mm$	$= 125.4mm$
$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$	$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{19.5 \times 10^6}{0.87 \times 410 \times 125.4}$	$= \frac{14.76 \times 10^6}{0.87 \times 410 \times 125.4}$
$= 435.9mm^2/m$	$= 330mm^2/m$
$A_{s\,min} = 0.13\% bd$	$A_{s\,min} = 0.13\% bd$
$= 0.0013 \times 1000 \times 132$	$= 0.0013 \times 1000 \times 132$
$= 171.6mm^2/m$	$= 171.6mm^2/m$
$A_{s\,min} < A_{s\,req}$.	$A_{s\,min} < A_{s\,req}$
Provide Y12mm @ 250mm c/c	Provide Y12mm @ 300mm c/c
$A_{s\,prov} = 566mm^2/m$	$A_{s\,prov} = 377mm^2/m$

PROVISION IS O K

DEFLECTION CHECK

Check for deflection at short-span mid-span.

$$\text{Service stress}, \sigma_s = \frac{5}{8} \times f_{yk} \times \frac{A_{s\,req}}{A_{s\,prov}}$$

$$= \frac{5}{8} \times 410 \times \frac{313.1}{377}$$

$$= 212.82 N/mm^2$$

$$\text{Modification factor} = \frac{310}{\sigma_s} = \frac{310}{212.82} = 1.46$$

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

REFERENCE	CALCULATIONS	OUTPUT
	Interpolating to obtain basic span ratio	
	$A_s/_{bd} \times 1000\% = \frac{313.1}{1000 \times 144} \times 100\% = 0.2$	
	$0.5 - 28$	
	$0.2 - x$	
	$0.15 - 38$	
	$\frac{x - 28}{38 - 28} = \frac{0.2 - 0.5}{0.15 - 0.5}$	
	$x = 36.6$	
	Limiting span-depth ratio $= M.F \times$ basic span ratio $= 1.46 \times 36.6 = 53.44$	
	Actual span-depth ratio ; deflection is O.K	DEFLECTION IS O.K

REFERENCE	CALCULATIONS	OUTPUT

PANEL 6:

Aspect ratio

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

Since $l_y/l_x < 2$; slab is a two-way spanning slab.

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

SHORT SPAN

Continuous Edge	Mid span
$\beta_{sx}^- = 0.031$	$\beta_{sx}^+ = 0.024$
$M_{sx}^- = \beta_{sx}^- n l_x^2$ $= 0.031 \times 14.64 \times 6^2$ $= 16.34\ kNm$	$M_{sx}^+ = \beta_{sx}^+ n l_x^2$ $= 0.024 \times 14.64 \times 6^2$ $= 12.65\ kNm$
$d = h - c - \phi/2$ $= 175 - 25 - 12/2$ $= 144mm$	$d = h - c - \phi/2$ $= 175 - 25 - 12/2$ $= 144mm$
$K = \frac{M}{f_{cu}bd^2} = \frac{16.34 \times 10^6}{25 \times 1000 \times 144^2}$ $= 0.032$	$K = \frac{12.65 \times 10^6}{25 \times 1000 \times 144^2}$ $= 0.024$
$l_a = 0.5 + \sqrt{0.25 - K/1.134}$ $= 0.5 + \sqrt{0.25 - \frac{0.032}{1.134}}$ $= 0.97$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.024}{1.134}}$ $= 0.98$
Since $l_a = 0.97 > 0.95$, we use $l_a = 0.95$	Since $l_a = 0.98 > 0.95$, we use $l_a = 0.95$
$Z = l_a d = 0.95 \times 144$ $= 136.8mm$	$Z = l_a d = 0.95 \times 144$ $= 136.8mm$
$A_{s\,req} = \frac{16.34 \times 10^6}{0.87 \times 410 \times 136.8}$ $= 334.86mm^2/m$	$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$ $= \frac{12.65 \times 10^6}{0.87 \times 410 \times 136.8}$ $= 254.24mm^2/m$
Provide Y12mm @ 300mm c/c	Provide Y12mm @ 300mm c/c
$A_{s\,prov} = 377mm^2/m$	$A_{s\,prov} = 377mm^2/m$
$A_{s\,min} = 0.13\% bd$ $= 0.0013 \times 1000 \times 144$ $= 187.2mm^2/m$	$A_{s\,min} = 0.13\% bd$ $= 0.0013 \times 1000 \times 144$ $= 187.2mm^2/m$
Since $A_{s\,prov} > A_{s\,min}$	Since $A_{s\,prov} > A_{s\,min}$

PROVISION IS OK

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge

$\beta_{sy}^- = 0.032$

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

$= 0.032 \times 14.64 \times 6^2$

$= 16.87 kN.m$

$d = h - c - \phi/_2 - \phi$

$= 175 - 25 - 12/_2 - 12$

$= 132mm$

$K = \frac{16.87 \times 10^6}{25 \times 1000 \times 132^2}$

$= 0.039$

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

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

$= 0.96$

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

$Z = l_a d$

$= 0.95 \times 132 = 125.4mm$

$A_{sreq} = \frac{16.84 \times 10^6}{0.87 \times 410 \times 125.4}$

$= 376.48 mm^2/m$

Provide Y12mm @ 250mm c/c

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

$A_{smin} = 0.13\%$ bd

$= 0.0013 \times 1000 \times 132$

$= 171.6 mm^2/m$

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

Mid span.

$\beta_{sy}^+ = 0.024$

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

$= 0.024 \times 14.64 \times 6^2$

$= 12.65 kNm$

$d = h - c - \phi/_2 - \phi$

$= 175 - 25 - 12/_2 - 12$

$= 132mm$

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

$= 0.029$

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

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

$= 0.97$

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

$Z = l_a d$

$= 0.95 \times 132 = 125.4mm$

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

$= 282.81 mm^2/m$

Provide Y12mm @ 300mm c/c

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

$A_{smin} = 0.0013 bd$

$= 0.0013 \times 1000 \times 132$

$= 171.6 mm^2/m$

Since $A_{sprov} > A_{smin}$ — PROVISION IS O.K

DEFLECTION CHECK

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

Service stress, $\sigma_s = \frac{5}{8} \times 410 \times \frac{282.81}{377}$

$= 192.23 N/mm^2$

Modification factor $= \frac{310}{\sigma_s} = \frac{310}{192.23} = 1.61$

REFERENCE	CALCULATIONS	OUTPUT

Interpolating to obtain the basic span depth ratio

$$P = \frac{As_{req}}{bd} \times 100 = \frac{259.24}{1000 \times 144} \times 100 = 0.18$$

0.5 — 28

0.18 — x

0.15 — 38

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

$$x = 37.14$$

Basic spandepth ratio = 37.14

Limiting span-depth ratio = 1.61 × 37.14 = 59.80

Actual span-depth ratio = $\frac{l_{x}}{d} = \frac{6000}{144} = 41.67$

Since actual span depth ratio is less than the limiting span-depth ratio, deflection is o.k

DEFLECTION IS SATISFACTORY.

PANEL 7 (SEVEN)

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

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

$c = 25mm \quad h = 175mm$

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

SHORT SPAN

Continuous Edge

$\beta_{sx}^- = 0.042$

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

$= 0.042 \times 14.64 \times 6^2$

$= 22.13 \, kNm$

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

$= 175 - 25 - 6$

$= 144mm$

$K = \frac{M}{f_{ck} b d^2} = \frac{22.13 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.04$

$Z = l_a d$

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

$= 0.5 + \sqrt{0.25 - 0.04/1.134}$

$= 0.96$

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

$Z = l_a d = 0.95 \times 144$

$= 136.8$

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

$= \frac{22.13 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 453.5 \, mm^2/m$

$As_{min} = 0.13\% bd$

$= 0.0013 \times 1000 \times 144$

= [illegible]

Mid-Span

$\beta_{sx}^+ = 0.032$

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

$= 0.032 \times 14.64 \times 6^2$

$= 16.86 \, kNm$

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

$= 175 - 25 - 6$

$= 144mm$

$K = \frac{M}{f_{ck} b d^2} = \frac{16.86 \times 10^6}{25 \times 1000 \times 144^2}$

$= 0.03$

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

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

$= 0.97$

Since $0.97 > 0.95$ use $l_a = 0.95$

$Z = l_a d = 0.95 \times 144$

$= 136.8$

$A_{req} = \frac{M}{0.87 f_y Z}$

$= \frac{16.86 \times 10^6}{0.87 \times 410 \times 136.8}$

$= 345.5 \, mm^2/m$

$As_{min} = 0.13\% bd$

$= 0.0013 \times 1000 \times 144$

= [illegible]

REFERENCE	CALCULATIONS	OUTPUT

LONG SPAN

Continuous Edge	Mid span
$\beta_{sy}^- = 0.032$	$\beta_{sy}^+ = 0.024$
$M_{sy}^- = \beta_{sy}^- n l_x^2$	$M_{sy}^+ = \beta_{sy}^+ n l_x^2$
$= 0.032 \times 14.64 \times 6^2$	$= 0.024 \times 14.64 \times 6^2$
$= 16.57\,kNm$	$= 12.65\,kNm$
$d = h - c - \phi - \phi/2$	$d = h - c - \phi - \phi/2$
$= 175 - 25 - 12 - 6$	$= 175 - 25 - 12 - 6$
$= 132mm$	$= 132mm$
$K = \frac{M}{f_{cu} b d^2} = \frac{16.57 \times 10^6}{25 \times 1000 \times 132^2}$	$K = \frac{12.65 \times 10^6}{25 \times 1000 \times 132^2}$
$= 0.04$	$= 0.03$
$l_a = 0.5 + \sqrt{0.25 - \frac{0.04}{1.134}}$	$l_a = 0.5 + \sqrt{0.25 - \frac{0.04}{1.134}}$
$= 0.96$	$= 0.97$
Since $l_a > 0.95$, we use $l_a = 0.95$	Since $l_a > 0.95$, we use $l_a = 0.95$
$Z = l_a d = 0.95 \times 132$	$Z = l_a d = 0.95 \times 132$
$= 125.4mm$	$= 125.4mm$
$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$	$A_{s\,req} = \frac{M}{0.87 f_{yk} Z}$
$= \frac{16.87 \times 10^6}{0.87 \times 410 \times 125.4}$	$= \frac{12.65 \times 10^6}{0.87 \times 410 \times 125.4}$
$= 377.15mm^2/m$	$= 282.8mm^2/m$
$A_{s\,min} = 0.13\%\,bd$	$A_{s\,min} = 0.13\%\,bd$
$= 0.0013 \times 1000 \times 132$	$= 0.0013 \times 1000 \times 132$
$= 171.6mm^2/m$	$= 171.6mm^2/m$
Provide Y12mm @ 250mm c/c	Provide Y12mm @ 250mm c/c
$A_{s\,prov} = 452mm^2/m$	$A_{s\,prov} = 452mm^2/m$
Since $A_{s\,prov} > A_{s\,min}$	Since $A_{s\,prov} > A_{s\,min}$

PROVISION IS O.K.

DEFLECTION CHECK

Check for deflection at short span midspan

Service stress $\sigma_s = \frac{5}{8} f_y \frac{A_{s\,req}}{A_{s\,prov}} = \frac{5}{8} \times \frac{410 \times 345.5}{377}$

$= 234.84\,N/mm^2$

Modification factor $= \frac{310}{\sigma_s} = \frac{310}{234.84} = 1.32$

We interpolate to obtain basic span depth ratio.

$A_s/bd \times 100\% = \frac{345.5}{1000 \times 144} \times 100\% = 0.24$

0.5 − 30

0.24 − x

0.15 − 33

$\frac{x - 30}{33 - 30} = \frac{0.24 - 0.5}{0.15 - 0.5}$

EC2.7

REFERENCE	CALCULATIONS	OUTPUT
	Limiting span-depth ratio = M.F × basic span ratio $= 1.32 \times 32.23$ $= 42.55$ Actual span-depth ratio $= \frac{l_x}{d} = \frac{6000}{144}$ $= 41.67$ Since actual span-depth ratio < limiting span-depth ratio, ∴ deflection is o.k	DEFLECTION IS SATISFACTORY.

CHAPTER FOUR

DATA ANALYSIS

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 4.2 shows the percentage area of steel required for slab in the short span and long span

Table 4.3 shows the span moment of beam

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)		% difference
	BS 8110	EC 2	
Short span mid span	301.03	313.1	3.85
Short span continuous edge	404.8	421.1	3.87
Long span mid span	316.93	330	3.96
Long span continuous edge	423.35	435.9	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	BS8110	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 Eurocode2 exceeds that of the BS8110 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 slab. The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.88% for column.

CHAPTER FIVE

5.0 CONCLUSION

The results of the comparative study led to the following conclusions:

i. The BS8110 moments exceeds that of the Eurocode2 by an average of about 4.29% at spans and 4.31% at supports for beams.

ii. The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.25% for slab.

iii. The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.88% for column.

iv. The Eurocode2 is more conservative in terms of the partial factors of safety for loadings.

v. Based on the results obtained, we can conclude that buildings designed by the provisions of the Eurocodes are more economical with the required margin of safety.

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 Eurocode2 exceeds that of the BS8110 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 slab. The average percentage difference in total weight of steel required for the BS8110 exceeds that of the Eurocode by 1.88% for column.

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