ANALYSIS AND DESIGN OF A PROPOSED TWO STOREY SCHOOL BUILDING AT KUJE AREA COUNCIL OF ABUJA

BY

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DECLARATION

I, Adeniyi Idowu Samuel, declare that this work was done by me and has never been presented elsewhere for the award of a Post Graduate Diploma. I hereby relinquish the copyright to the Federal University of Technology, Minna, Niger State. All sources of information used are duly acknowledged.

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CERTIFICATION

The thesis tiled: Analysis and design of a proposed two storey school building at Kuje area council of Abuja by: Adeniyi Idowu Samuel meets the regulations governing the award of the Post Graduate Diploma in Civil Engineering of the Federal University of Technology, Minna and it is approved for its contribution to scientific knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to the glory of Almighty God who is the pillar that holds my life firm and secure, and to my mother Mrs. Adeniyi A. Victoria, whose prayer and support for me have been of immeasurable values. Thank you and God bless.

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Adeniyi Idowu Samuel

ABSTRACT

Presented in this thesis is the analysis and design of a 2- storey School building. The general principle of ultimate and serviceability limit state design has been adopted. In arriving at a good and perfect design, structural components which have the same geometrical properties and boundary conditions are grouped together for load and bending moment analysis, according to the specifications from the code of practice for structural use of concrete (BS 8110) parts, 1, 2 and 3 of 1997. Checks were carried out to ensure that each structural component satisfies the serviceability requirements of BS 8110. The bearing capacity of soil within Kuje Area has been given to be $150KN/m^2$. All design calculations are done in S.I units. The research work has broadens my understanding in analysis and design of a structure.

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LIST OF NOTATIONS

The notations employed in this project are based on those used in BS8110.

Ac.	Area of concrete
As.	Area of tension reinforcement
As'.	Area of compression reinforcement
Asc	Total area of longitudinal reinforcement in columns
As req	Area of tension reinforcement required
Asv	Cross sectional are of two legs of link reinforcement
b	Width of section
bt	Breadth of section at level of tension reinforcement
b_w	Breadth of web
d	Effective depth to tension reinforcement
d'	Effective depth to compression reinforcement
Ec	Short term modulus of elasticity of steel
Es	Modulus of elasticity of steel
F	Total design ultimate load
FEM	Fixed end moment
Ft	Tie force
F_{cu}	Characteristic strength of concrete
Fs	Service stress
F_{bs}	Bond stress
F_y	Characteristic strength of reinforcement
F_{yy}	Characteristic strength of shearing reinforcement
G_k	Characteristic Dead load
Q_k	Characteristic live load

h Overall depth of diameter of section

h_f Thickness of flange

I Second moment of area

La Lever arm factor 2/d

L_{ex} Effective height for bending about major axis

L_{ey} Effective height for bending about minor axis

Lo Clear height of column between end restraint

Lx Length of shorter side of rectangular slab

Ly Length of longer side of rectangular slab

M Bending moment due to ultimate load

Mi Maximum initial moment in column due to ultimate load

M_{sx}, M_{sy} Bending moments at mid spans on strips of unit width and of spans lx and ly respectively

Mu Design ultimate moment of resistance of section

M_x, M_y Moments about major and minor axis of short columns due to ultimate loads

N Ultimate axial load

n Total distributed load per unit area (1.4gk + 1.6qk)

Sv Spacing of links

T Torsional moment due to ultimate loads

V Shearing stress on section due to ultimate loads

Vc Ultimate shearing resistance per unit area provided by concrete alone

X Depth of neutral axis

Z lever arm

ø Bar size

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background/Statement of Problem

The successful completion of any structural design project is dependent on many variables. However, there are a number of fundamental objectives which must be incorporated in any design philosophy to provide a structure which is safe and sound throughout its intended lifespan.

A structure refers to a system of connected parts that resist external and internal action (loads) without undue deformation.

The main function of a structure is to transmit action (load) from the point of application to the point of support and ultimately through the foundation to the ground. When designing a structure to serve a specified function for public use the engineer must account for the safety, aesthetics, serviceability while taking into account economic and environmental constraint solution before final judgement can be made as to which structural form is most appropriate. The design processes is both creative, technical and require a fundamental knowledge of material properties and the laws of mechanics which govern material response. To analyse a structure properly, certain idealization must be made as to how members are supported and connected. The loadings are determined from codes and local specifications, and the forces in the members and their displacements are found using the theory of structural analysis (Mc Kenzie, 1998)

1.2 Justification

1.2.1 Concrete:-

Concrete is a freshly mixed materials which can be moulded into different shape. The relatively quantities of cement, aggregates, sand and water mixed together, control the properties of concrete in the wet state as well as in the harden state. It's also a variable material, having a wide range of strengths and stress-strain.

Table 1:1 Individual Properties of Concrete and Steel

CONCRETE	STEEL
i. **Elasticity** - The modulus of elasticity of concrete is a function of the modulus of elasticity of the aggregates and the cement matrix and their relative proportions. The modulus of elasticity of concrete is relatively constant at low stress levels but starts decreasing at higher stress levels as matrix cracking develops. The elastic modulus of the hardened paste may be in the order of 10-30 GPa and aggregates about 45 to 85 GPa. The concrete composite is then in the range of 30 to 50 GPa. The American Concrete Institute allows the modulus of elasticity to be calculated using the	i. **Elasticity** - the steel behaves as a perfectly elastic material up to a well defined yield point. Removal of stress at level below the yield stress causes the material to reverse to its unstressed dimensions. Linear elastic behaviour ceases at a stress level below the yield point known as the proportional unit. The slope of the stress-strain curve in the elastic range defines the modulus of elasticity. For structural steel, its value is virtually independent of its steel type and its commonly taken as $205KN/mm^2$. ii. **Tensile stress** – the applied stress to cause failure is considerably greater than the yield stress. From the fig the ultimate

following equation:

$$E_c = 33w_c^{1.5}\sqrt{f'_c}\text{(psi)}$$

where

w_c =weight of concrete (pounds per cubic foot) and where

$$90\frac{\text{lb}}{\text{ft}^3} \leq w_c \leq 160\frac{\text{lb}}{\text{ft}^3}$$

f'_c =compressive strength of concrete at 28 days (psi)

This equation is completely empirical and is not based on theory. Note that the value of E_c found is in units of psi. For normalweight concrete (defined as concrete with a w_c of 150 lb/ft^3 and subtracting 5 lb/ft^3 for steel) E_c is permitted to be taken as $57000\sqrt{f'_c}$.

ii **Expansion and shrinkage** - Concrete has a very low coefficient of thermal expansion. However, if no provision is made for expansion, very large forces can be created, causing cracks in parts of the structure not capable of withstanding the force or the

stress is nearly twice the yield stress.

iii. **Ductility** – An important property of steel is its ability to undergo large deformation without fracture. The strain to failure may reach 25% in mild steel, will be less for higher carbon steel and may drastically be curtailed in all steel under circumstances. This may lead to brittle fracture. The elastic strain is a small portion of the total strain possible before failure occurs. In order to analyse the behaviour of steel element which are stressed beyond the elastic limit (yield point) there is need to simplify the real stress- strain curve for steel. The portion of curve from yield to failure is replaced by a horizontal line representing strain at constant stress.

repeated cycles of expansion and contraction. The coefficient of thermal expansion of Portland cement concrete is 0.000008 to 0.000012 (per degree Celsius) (8 to 12 microstrains/°C)(8-12 1/MK).[5] As concrete matures it continues to shrink, due to the ongoing reaction taking place in the material, although the rate of shrinkage falls relatively quickly and keeps reducing over time (for all practical purposes concrete is usually considered to not shrink due to hydration any further after 30 years). The relative shrinkage and expansion of concrete and brickwork require careful accommodation when the two forms of construction interface. Because concrete is continuously shrinking for years after it is initially placed, it is generally accepted that under thermal loading it will never expand to its originally placed volume. Due to its low thermal conductivity, a layer of concrete is frequently used for fireproofing of steel structures. iii **Cracking** - All concrete structures will	

crack to some extent. One of the early designers of reinforced concrete, Robert Maillart, employed reinforced concrete in a number of arched bridges. His first bridge was simple, using a large volume of concrete. He then realized that much of the concrete was very cracked, and could not be a part of the structure under compressive loads, yet the structure clearly worked. His later designs simply removed the cracked areas, leaving slender, beautiful concrete arches. The Salginatobel Bridge is an example of this. Concrete cracks due to tensile stress induced by shrinkage or stresses occurring during setting or use. Various means are used to overcome this. Fiber reinforced concrete uses fine fibers distributed throughout the mix or larger metal or other reinforcement elements to limit the size and extent of cracks. In many large structures joints or concealed saw-cuts are placed in the concrete as it sets to make the inevitable cracks occur where they can be managed and out of sight. Water tanks and highways are examples of structures requiring	

crack control. **iv Creep** - *Creep* is the term used to describe the permanent movement or deformation of a material in order to relieve stresses within the material. Concrete which is subjected to long-duration forces is prone to creep. Short-duration forces (such as wind or earthquakes) do not cause creep. Creep can sometimes reduce the amount of cracking that occurs in a concrete structure or element, but it also must be controlled. The amount of primary and secondary reinforcing in concrete structures contributes to a reduction in the amount of shrinkage, creep and cracking.	

1.2.2 Reinforced concrete:-

The success of concrete as a structural material is due to its versatility, particularly when combined with steel to act compositely as reinforced; whilst harden concrete has a high comprehensive strength its tensile strength is very low which is normally assumed as zero in reinforced concrete design, this minimal tensile strength restricted the use of concrete to circumstances until late 19th century when methods were developed for reinforcing concrete to overcome its weakness in tension.

(a) A vertical element provides supports for the horizontal element and transfers the loadings to the foundations.

(b) The horizontal elements provide the immediate support for general use of the structure.

(c) **Columns** are vertical elements of relatively small cross-section which are efficient to carry vertical loads and transfer it to the foundation. A column can either be braced or embraced in which case can be short or slender column.

(d) **Walls** are vertical elements which are thin compared to their length and they are good for carrying horizontal loads in their plans (laterally loaded wall). In addition to the vertical they resist.

(e) **Beams** are horizontal elements of relatively small cross-section subjected to bending which support the slab and its load and transfer the load to the column.

(f) **Slabs** are used in floors; roofs are walls of building and as the deck of bridges. It can take many forms such as in situ solid slabs, ribbed slab or precast unit. Slab may span in one direction or in two directions and they may be supported on monolithic concrete beams, steel beams, walls or directly by the structure's columns. It also carries the imposed load and its own weight, and then transfers it to the beams.

1.2.3 Roof Truss

A structure that is composed of a number of members pin-connected at their ends to form a stable framework is called a truss. It is generally assumed that loads and reaction are applied to the truss only at the joints. Many truss structure are three dimensional in nature. However in roof systems, the three dimensional framework can be sub-divided into planer components for analysis as planer truss.

1.2.4 Timber

Timber is wood in a form that is stable for construction of carpentry, joinery, or for reconversion for manufacturing purposes. It is used in framing and load bearing structure, where strength is the measure factor as its selection and use. It is a construction material that possesses attributes such as strength, versatility, toughness, flexibility, durability, workability, and is desirable in modern construction. It does not corrode unlike any other building materials. Timber is a good choice for construction materials because of its cost which is cheap in comparison with other materials such as steel and concrete. Its properties enable its usage for the construction of roof trusses.

1.3. Aim and Objectives

1.3.1 Aim

To analyse and design a two storey school building which is economical, safe and able to sustain all loads without deformation of the structure that would impair on the appearance, durability and performance during its life span.

1.3.2 Objectives

i. To analyze the structure using the general principle of ultimate and serviceability limit state

ii. To design the structure according to the specifications from the code of practice for structural use of concrete (BS 8110) parts, 1, 2 and 3 of 1997

iii. To determine the bearing capacity of the soil within Kuje Area Council.

iv. To carry out all necessary checks that would guarantee the structure.

1.4. Scope and Limitation

1.4.1 Scope

The scope of this work deals with the design of roof trusses and how the trusses will withstand the wind loads upon the roof. This work also deals with the design of slabs mainly for bending and in few cases for shear. The beams are also designed to transfer the slab loads to the columns which in turn are designed for axial forces but in few cases for moments. The foundation receives all loads from the columns and spread these loads to the soil. Its primary design is for bending and shear.

Various methods are used in calculating moments such as slope deflection, moment distribution and computer aided method. The method that is appropriate for vertical load analysis is the moment distribution method, because it can be applied to both plastic and semi plastic materials. It can also be used to analyze complex structures and is relatively cheap.

1.4.2 Limitation

The project is a two storey structure for school with a timber roof trusses designed in accordance with B.S 8110 part 1 1997, B.S 8110 part 3, B.S 5268.

1.5 Design Philosophy

1.5.1 Limit State Design

In common with most current UK code of practice, BS 5950 adopts a limit state approach to design. In this approach the designer select a number of criteria by which to assess the proper functioning of the structure and then checks whether they have been satisfied. That is the limit state which they will become unfit for their intended use.

The limit states are states beyond which a structure can no longer satisfy the design performance; it was formulated to achieve the objectives set out that:-

i- The structure is economical to construct.

ii- The structure is economical to maintain.

iii- The structure is serviceable and performs its intended purpose whilst in use.

iv- The structure will possess an acceptable margin of safety against collapse whilst in use.

v- The structure is sufficiently robust such that damage to an extent disproportionate to the original cause will not occur.

vi- The limit state design is of two types:-

1.5.2 Serviceability Limit State.

The serviceability limits state in which a condition for example deflection, vibration of cracking occurs to an extent, which is unacceptable to the owner, occupier or client.

1.5.3 Ultimate Limit State:

The ultimate limit state in which structure, or some part of it, is unsafe for its intended purpose for example compressive, tensile, shear or flexural failure or instability leading to partial or total collapse.

1.5.4 Structural Loading:-

Every structure are subjected to various kind of loading(they are direct forces applied in a structure) they may be permanent such as self-weight, finishes, fitting and fixed equipment or variable such as imposed load, wind loads and snow loads or accidental such as explosions, impact from vehicles.

1.5.5 Imposed Loads;

Imposed loads are loads due to variable effects such as the movement of people, furniture, equipment and traffic. The values adopted are based on observation and measurable and are inherently less accurate than the assessment of dead loads, in British code 6339 clause 5.0 and table 1 define the magnitude of uniformly distributed and concentrated point loads. Imposed roof load depends on its configuration that is, flat roof, sloping roof and curved roof.

1.6 Structural Analysis:-

Analysis of structural element is to obtain a set of internal forces and moments throughout the structure that are in equilibrium with design loads for the required loading combination. Although concrete structure only behave elastically under small loads while the sections remain un-cracked, a linear elastic analysis may still be used for both serviceability and strength limit state to determine the internal forces and moments provided the structure has sufficient ductility to distribute moments from highly stressed regions to less highly stressed regions. The fastest and easiest method is the use of moment distribution. However with the advent of 21^{st} century, soft wares such as RISA, Beam Boy and so on can be used to analyze the structural elements to ascertain its internal force and moments. The trusses of the roof can be analyzed by either joint method of analysis or method of section.

1.6.1 Elements

All buildings are made up of elements which can be categorized as; bending, tension or compression elements. Elements are also categorized as either one or two dimensional. Most of modern buildings are made up of a combination of all these elements.

1.6.2 Types of Elements

(1) Tension

(2) Compression

(3) Bending

(1) **Tension** – Most tension elements are made from steel which is extremely strong in tension. Reinforced concrete elements are rarely design to act in pure tension. This is because concrete is roughly ten times as strong in compression as it is in tension

- Failure of tension elements can be much more dangerous than that of compression
- Failure in tension elements are usually immediate and without warning, as tension element cannot take any load after failure. The load it is carrying is passed immediately to any other. This can in-turn makes the load stretch the tension elements beyond their design capacity and a catastrophic collapse can occur.

(2) **Compression** - compression elements fall into two categories',

(a) Column

(b) Wall

(a) **Column** - A reinforced concrete column is typically a compression member and the reinforcement tensile qualities will help prevent lateral movement and buckling. Tall columns are susceptible to buckling.

Types of columns

i. Short column
ii. Slender column
iii. Axially loaded column
iv. Bi-axially loaded column
v. Uni-axially loaded column

Columns can also be categorized as braced and unbraced. Clause 3.8.1.5 of B.S. 8110: Part -1 1997, defines braced columns as those laterally supported by wall, buttressing etc. designed to resist all lateral forces in that plane. It should otherwise be considered as unbraced. Furthermore, a column may be considered as short column when the ratios Lex/h or Ley/b is less than 15 for braced columns and 10 for unbraced columns. It should otherwise be considered as slender column.

(b) **Wall** - They are non-load bearing walls and they are simply to enclose the space. Walls can be masonry or concrete. Like columns, walls can extend past more than one floor or span from floor to ceiling.

Walls give a building structural rigidity and lateral stability.

(3) **Bending**

(a) **Beams**- The simplest type of bending element is a beam. It is used to span a gap between at least two supports and provide support for slabs that will act as floors or ceilings.

Types of beams

(i) Simply supported beams

(ii) Continuous beams

(b) **Slabs-** They are classed as two dimension bending elements and are used to span the gap between beams. Sometimes voids are incorporated into the slab to help reduce its self weight. The positioning of voids is very important and must be where the element carries the lowest bending moment, therefore cantered around the neutral axis.

Types of slab

i. Flat slab
ii. Flat slab with drops
iii. Waffle slab
iv. One-way spanning slab
v. Band beam and one-way slab
vi. Ribbed slab
vii. Two-way spanning slab
viii. Pre-cast systems
ix. Cantilever

1.7.0 Foundation

Foundations are horizontal or vertical members supporting the entire structure and transmitting the loads to the soil below. They are sub-structures supporting the super-structures of columns, beams, walls, slabs and roofs. Generally, foundations can be classified as shallow foundation or as deep foundation. The choice between the two can be taken after thorough examinations of the following elements:

a. The magnitude of the transmitted load from the super-structure;

b. Soil nature;

c. The economic aspects of the elements of the foundation work and

d. Problems concerning foundation construction.

Types of foundation

i. Shallow foundations:

   a. Strip foundation

   b. Wide strip foundation

   c. Pad foundation

   d. Strap foundation

   e. Raft foundation (slab, slab and beam and cellular).

ii. Deep foundations:

   a. Pile foundation

   b. Diaphragm walls

   c. Displacement foundation

The foundation type to be chosen depends largely on the loads transmitted and the receiving soil strata and must satisfy the following two fundamental and independent requirements:

- The factor of safety against shear failure of the supporting soil must be adequate and
- The settlement should neither cause any unacceptable damage nor interfere with the function of the structure.

1.7.1 Reinforcement

The section 7 of BS8110 part 1 specifies that reinforcement should comply with BS4449, BS4461, and BS4462 which explains that different types of reinforcement may be used for the same members. Hence, for a beam the tensile (main) reinforcements and compressive reinforcement might be high yield bars with fy = 460, 410, 450, and 250. While mild steel are used for the links.

It maybe mathematically cumbersome to use two types of reinforcement as main bars or links since their strengths are not the same. Reinforcements should be kept clean by stacking them off the ground prior to usage, free from mud, oil paint because all these weaken the bonding between the bars with concrete except if the bars are rigidly fixed in the concrete in correct position. And special care should be taken in fixing reinforcements in their correct positions especially in cantilever before pouring concrete. At 28 days, section 3.1.7.2 of standard specifies minimum grades of 25.0N/mm^2 for the project work in both economy and safety of design.

In view of researches made on BS8110via books as reinforcement concrete design by W.H. Mosley and J.H. Bungey, Reinforcement concrete design handbook by Charles Reynolds and James Saleeman, Simplified reinforcement concrete design is highly based on safety and economy. Hence, the use of CP110 and CP114 are becoming highly a solution as they direct their design procedures more to safety. Therefore, the coming of BS8110 of 1985 and 1997 are concentrating on how to cut down on cost as the design is still based on safety. Since the concept of limit state method of design has been introduced, the design of each individual member must satisfy two separate criteria's which are:

i. The ultimate limit state which ensure that the probability of failure is acceptably low and

ii. The limit state serviceability which ensures satisfactory behaviour under service load (i.e. working loads).

Also owing to economy for instance, the use of steel in compression is always uneconomical when the cost of a single member is being considered of the depth of the concrete of that member, they may offset the initial cost of individual member. Finally, the design procedure which is employed in this design, has taken into consideration factor such as economy and safety simultaneously.

In BS8110 part 1 1985 under table 3.1.4 and 3.1.5 for the slab design, codes say that when the ratio of ly/lx < 2, the slab shall be considered span in two direction and if ly/lx > 2, it shall be considered to span in one direction. Table 3.1.4 and 3.1.5 help to determine the short span coefficient Bsx for a particular slab in order to obtain the moment acting on the slab.

In this code, it's also stated that

$$K = \frac{M}{\text{bdfcu}} \leq 0.156$$

This is used to check whether compression reinforcement is required for a particular structural member concerned (i.e. when $K^1 \leq K$) compression reinforcement is not required.

In the design of the roof beam, the Nigeria standard codes of practice 2: 1973 (use of timber) lists the varieties of timber we have in Nigeria and stated each that is good for particular conditions. It grouped the timber into about six groups i.e. N1, N2, N3,...,N6 and stated the uses of each of the grouped density and grade to which each timber belongs. The code also helped to determine the spacing of purlins and gives the value of compression parallel to grain (stress) for each timber. The code also helps in determining the sizes of the timber used and moisture content.

In the column design, both BS8110 part 1, 1985 and 1997. Reinforcement Concrete design by Mosley and Bungey stated that for short column both L_e x/ h < 15 for brace column and L_e y/ b < 10 for unbraced column. While for shorter column, both L_ey/b and L_ex/y > 10 for unbraced column.

Reinforced concrete design manual by Institution of Structural Engineers help to determine the value of the tern MF (fixed end moment), MFu (moment in upper column), Kb (stiffness of upper column), Kb_1 (stiffness of left hand side beam), and Kb_2 (stiffness of right hand side beam etc. Reinforced concrete design by Mosley and Bungey states that for biaxial bending

$\frac{Mx}{h} > \frac{My}{b}$, moment increase about x axis, while

$\frac{Mx}{h} > \frac{My}{b}$, moment increase about y axis.

Reinforced concrete design by Mosley and Bungey, table 9.4 page 257 gives the value of 'β' coefficient using the formula N/bhf_{cu}. BS8110 part 3 1985 design chart No. 28 rectangular column gives the value 100Asv/bh depending on the characteristic strength of reinforcement and d/h used.

In the floor beam analysis, Reinforced concrete design by Mosley and Bungey page 210 stated that;

$Z = \alpha\, d \leq 0.95d$, where /

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

d = Effective depth of the beam

this has been used in design of almost all structural elements and when it is greater than 0.95d, then use 0.95d in design. Also in page 205, 206, and 207 of Reinforced concrete design by Mosley and Bungey, it was stated that service stress (Fs) is given as;

$$Fs = \frac{2fyAsreq}{3Asprov} \cdot \frac{1}{\beta} \quad \text{..} \quad (i)$$

$$\frac{M}{bd^2} \quad \text{...} \quad (ii)$$

$$MF = 0.55 + \frac{(477 - fs)}{120(0.9 + M/bd^2)} \leq 2.0 \text{..........} \quad (iii)$$

The above expressions are used in the calculation of the deflection of the member.

1.8 Staircase

A staircase is a set of steps or flight leading from one floor to another. Materials for construction includes timber, stone and concrete (reinforced). Each step consists of horizontal portion or tread connected to front part known as riser. The going of a step is the horizontal distance between the faces of two consecutive risers. The rise of a step is the vertical distance between the tops of two consecutive treads. It has been found that, for comfortable usage, the best proportions of step are such that: Going + 2xRise = 580 or 600mm.

1.8.1 Types of staircase

i Straight flight stair

ii Quarter-turn stair

iii Free standing stair

iv Half-turn stair

v Spiral stair

vi Helical stair

vii Cantilever stair

CHAPTER TWO

2.0 LITERATURE REVIEW

Designing a reinforced concrete storey building, it is necessary for one to understand and identify irrespective of the structural material, the component members that make up a storey building, the stress conditions these members could be subjected to and how they could be tackled. Also the knowledge of the method of analysis employed, the types of load, how they would occur and how they combine is paramount to design efficiently and also making sure that the concrete building behaviour is satisfactory under service by the use of the code of practice BS 8110 part 1, 2. The provision of reinforcement was based on its intended function to resist failure inherent in monolithic construction and thus resistance is provided against all likely causes of damage to the structure.

2.1 Basis of Structural Design

In principle, this "Basis of Structural Design" requires explicit treatment of
the fundamental performance requirements of structures, such as safety, and the factors affecting the performance of structures. The concept of reliability design shall be applied as a basis for verifying compliance to performance requirements.

(a) This "Basis of Structural Design" covers structures in general in both building and public
works fields. The term "structure" is here defined as "organized construction works designed to provide intended functions while resisting actions."

(a) This "Basis of Structural Design" is a comprehensive framework, which covers both fields
of buildings and public works, and shows the basic issues necessary to establish or revise the technical standard of design for each type of structure. In other words, it is equivalent

to so-called "Code for Code Writers." Some of the basic issues may not be necessary for a specific technical standard of a structure. This "Basis of Structural Design" leaves selection of the necessary issues to the code writers for an individual structure.

(b) Whereas the design of a structure is a comprehensive work taking account of not only

safety, serviceability and restorability but also landscape, impact on the environment, economic efficiency, etc., this code only covers "structural design" considering serviceability, safety, restorability, etc.

(c) The fundamental performance requirements of structures and the factors affecting the

performance of structures are required to be treated in an explicit manner to ensure transparency and accountability of decision making about public structures in terms of structural design, as these have recently become increasingly in demand.

(d) The requirement for "applying the concept of reliability design as a basis" is intended for

"considering limit states and maintaining the probability of exceeding the limits within permissible target ranges during the design working life in consideration of uncertainty of the external actions and resistance of the structure".

It is important to refer to reliable data in the process of setting the basis

on the reliability design concept. It is also important to accumulate such data and open it to the public for this purpose.

When designing a structure, the design working life of the structure should be specified, and the following fundamental performance requirements (1) to (3) should be ensured for the specified period.

(1) Safety of human life in and around the structure is ensured against foreseeable actions

(Safety).

(2) The functions of the structure are adequately ensured against foreseeable actions acting on

structures (Serviceability).

(3) If required, continued use of the structure is feasible against foreseeable actions by restoration using technologies available within reasonable ranges of cost and time (Restorability).

(a) When designing a structure, specifying a design working life is required.

(b) (1) and (2) above refer to fundamental performance requirements for safety and serviceability, respectively.

(c) The concept of safety is based on "human safety," with the requirement being "safety of

human life in and around the structure," including prevention of collapse of constructed structures that are normally unmanned into the concept of safety.

(d) (3) above describes a fundamental performance requirement of "restorability" in addition to the other fundamental performance requirements, safety and serviceability.

The requirement for restorability is intended to control the level of damage, thereby enabling continued use of the structure by repairing damage to the structure from the foreseeable actions using appropriate techniques within reasonable cost and time.

In earthquake-prone Japan, designing public facilities that would restore their functions shortly after an earthquake to allow their continued use is an example of design taking account of restorability. Restorability as a fundamental performance requirement can also be recognized from the standpoint of avoiding the situation in which a great number of buildings are on the verge of collapse after an earthquake, requiring demolishing and rebuilding.

(e) It should be noted, though not specified as a requirement, there is a concept of requirement for structural integrity, or ability of a structure not to be damaged to an extent disproportionate to the original cause, such as local failure producing a fatal effect on the entire structural system. This concept is included in ISO 2394 as a fundamental requirement. Such a concept should also be considered as a part of the fundamental safety and restorability requirements.

Reinforced Concrete Design of Engineering Conference Hotel, Awal Tanko (2010)

The purpose of the project is to analyze and design the structure base on the British standard code of practice to produce a detailed design of the proposed three storey multipurpose structure for a safe, stable, durable and most economical.

A building structure is either framed or unframed. A domestic building (i.e a bungalow or two-storey building) founded on a very good soil may be built without frames. Here, the reinforced concrete slabs may be supported by the walls below which must be treated as load bearing walls. The strip foundation type can then be used.

While such load bearing walls are recommended to be at least 25 blocks from every bag of cement and adequately compacted (favourably, machine moulded). In other hand, buildings that are at least 3 storeys in height (or less but built on very poor soil) must be framed (V.O Oyenuga, 1999).

Structural Design of a Five Storey Reinforced Concrte Hotel building, Atim Lucy (2011)

The design was based on the ultimate limit state method which ensures that the probability of failure of the structure is acceptably low with load analysis of structural members carried out in accordance with the provisions in the BS 8110 1997 part 1 and 1985 part 2 and 3.

Structural Analysis and Design of six storey Hotel building in Oshogbo, Osun State, Sunday Christian Uche (2011)

The design of the structure was carried out bearing in mind safety, durability and economy as much as possible. Proper monitoring was advised during construction and maintenance of the structure. These are to ensure that the safety and durability of the structure are guarantee.

CHAPTER THREE

3.0 MATERIALS AND METHOD

3.1 Materials

Structural design is the process of selecting members of required dimensions such that they provide adequate stability under service loads. There are conditions that a structural designer must keep in mind. One is "stability" and the other is "serviceability, economy and safety". Stability of a structure means that it can resist the loads acting on it satisfactorily and that the structure will not collapse immediately (that is, it provides enough time to escape to safety). Serviceability refers to certain conditions that are required so that the structure remains serviceable. In achieving this, relevant Codes of Practice were employed like BS8110 Parts I, II & III. The Structural use of Concrete, BS5268 Part I. The Structural use of Timber and textbooks

3.2 Method

3.2.1 Analysis and Design of Roof Trusses, Slabs and Beams

The method for this project will be based on limit state approach. This design approach will attained a reasonable possibility that the structure will not fail under working load condition during the lifespan.

The method to be employ in designing the structure is outline below:-

(1) Preparing the general arrangement (G.A) of the structure, that is, the positions of the foundation, roof, beams, slabs and columns.

The general arrangement can be classified as follow:-

i- The ground floors general arrangement indicating column locations, its type and sizes with a typical cross section.

ii- The upper floors general arrangement indicating beams, columns, slabs location and a typical cross section.

iii- The roof's general arrangement; indicating trusses location.

(2) Design of slabs

The reinforcement types, numbers and their spacing are indicated.

(3) Beams analysis

It involves the calculation of the maximum shear force and maximum bending moment on it.

(4) Beams design

It shows the reinforcement types and numbers.

(5) Foundation design

The reinforcement types and their numbers are indicated.

Design was carried out in the following order:

i. Analysis and design of roof trusses which transmit load to the supporting roof beams
ii. Analysis and design of slab which transmit load to supporting beams.
iii. Analysis and design of beam which transmit load to the columns.
iv. Analysis and design of column which transmit load to the foundation.
v. Foundation analysis and design which transmit the load to the ground creating pressure on the soil.

The procedure involved in the design of each element is stated below;

Slab Design:

1. Decide on the material stresses to be used that is f_{cu} and f_y

2. Assume overall thickness *h* of slab
3. Estimate the characteristics loads Q_k and G_k per unit area
4. Calculate the design loads
5. Determine the ultimate bending moment *M*
6. Choose the appropriate concrete cover
7. Calculate the effective depth 'd'
8. Calculate the reinforcement area "A_{Sreq}'
9. Select the reinforcement area to be provided
10. Check the span/effective depth ratio

Beam Design

1. Decide on the material stresses to be that is f_{cu} and f_y
2. Assume beam size
3. Estimate the characteristics loads Q_k and G_k per unit length of the beam
4. Calculate the design loads
5. Determine the ultimate bending moment *M*
6. Choose the appropriate concrete cover
7. Determine the ultimate moment of resistance based on the concrete section (this should be equal to or greater than the ultimate bending moment or a large section be considered)
8. Calculate the reinforcement area
9. Select the reinforcement area to be provided
10. Check the span/effective depth ratio
11. Calculation of the shear reinforcement

CHAPTER FOUR

4.0 RESULTS / PRESENTATION OF DATA AND DISCUSSION

4.1 Design of Roof Truss

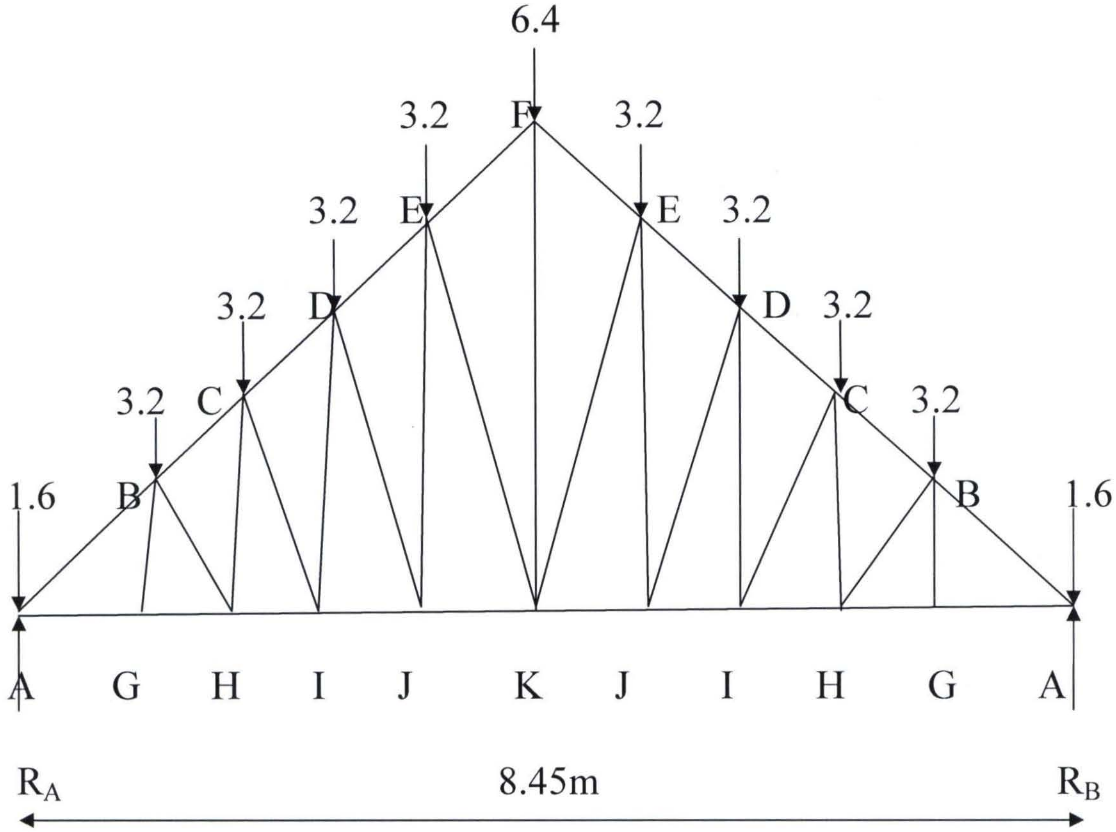


Fig 4.1: Roof Trusses Load Analysis

References	Calculations	Output
Load on Rafter	**Roof truss analysis and design** Assume basic wind speed, V= 40m/s Characteristic wind pressure (W_k) $W_k = 0.613Vs^2 N/m^2$ $V_s = V \times S_1 \times S_2 X S_3$ Where: V_s = design wind speed (m/s) S_1 = multiplying factor relating to topology S_2 = multiplying factor relating to height above ground and wind braking S_3 = multiplying factor relating to life of structure Using topographic factors of number 3 that is country with many wind breaks, small towns, outskirts of large cities Class- B Vertical height of building = 9m Horizontal height of building = 57.80m Ground roughness, building size and height above ground, factor S_2 Using interpolation : Height Factor 9 S_2 15 0.83 20 0.90 $\frac{0.83 - S_2}{0.9 - S_2} = \frac{15 - 9}{20 - 9}$ $11(0.83 - S_2) = 6(0.9 - S_2)$ $S_2 = 0.75$	

:- Vs = 40x1x0.75x1 = 30m/s

$W_k = 0.613 x 30^2 = 551.7 N/m^2$

$W_k = 0.5517 kN/ m^2$

$W_k =$ $0.5517 kN/ m^2$

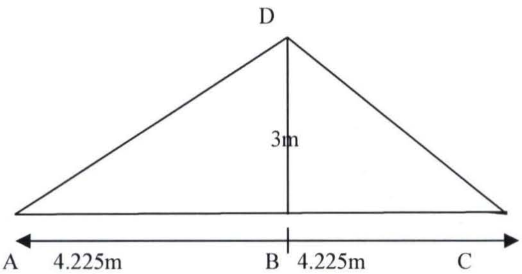


Fig 4.11: Roof Truss

Slope Angle (Θ)

$\Theta = Tan^{-1} \frac{(3)}{4.225} = 35.4^0$

Slope length

$AD^2 = AB^2 + BD^2$

$AD^2 = 4.225^2 + 3^2 = 20.10$

AD = 5.18m

Assumed design parameters

Truss spacing = 1.5m c/c

Purlin spacing =850mm

Dimensions for timber roof members are:

Purlin = 50 x 75mm

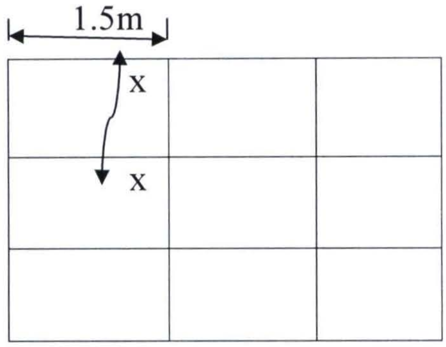
Rafter = 50 x 150mm

Tie beam = 50 x 150mm

King post = 50 x 150mm

Structural Analysis of Roof

Timber type = Mahogany (Benin)

	Analysis	
	Dead load + Imposed load	
		
	36trusses@ 1.5m c/c =54.15m	
	11 purlins @850mm c/c	
	Fig 4.12: Section of Roof Plan	
	Roof Plan	
	Spacing of truss = 1.5m	
	Unit weight of Aluminum sheet + normal laps = $2.44kg/m^2$	
	And fastening (corrugated 0.559mm thick)	
	Density of mahogany = $672kg/m^3$	
	Self weight of Aluminum roofing sheet = 0.85 x 2.44x 9.81/1000 = 0.020kN/m	
	Self weight of Purlin = 0.05 x 0.075 x 672 x 9.81/1000 = 0.025kN/m	
	Total dead load Gk = 0.045kN/m	Gk = 0.045kN/m
	Live load (with access) = 1.5kN/m	
	Total live load (Q_k) = 1.5 x 0.85 = 1.275kN/m	Q_k= 1.275kN/m
	Ultimate design load (n) = 1.4Gk + 1.6Qk	
	1.4 x 0.045 + 1.6 x 1.275 = 2.10kNm	n = 2.10kNm

Consider joint A and B on roof plan

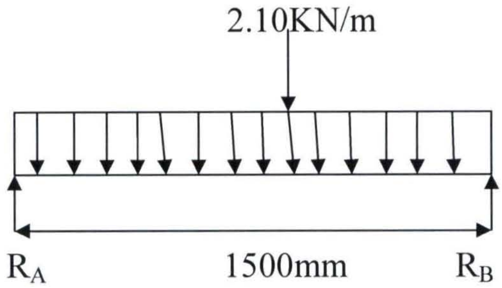


Fig 4.13: Roof Analysis

Load analysis

$R_A = R_B = \frac{wl}{2} = \frac{2.10 \times 1.5}{2} = 1.58kN$

Transferred to rafter at nodes

Hence, for external nodes, $R_A = R_B = P/2 = 1.58kN$

For internal nodes $P = 2 \times 1.58 = 3.16kN$

1.6kN is the load on the external nodes while the internal nodes are loaded with twice its value that is $2 \times 1.6kN$. The load on the kingpost is twice the load on the internal nodes that is $2 \times 3.2kN$.

Slope = 35.4^o

Design of rafter

Reactions to the roof truss

$R_A + R_B = 2(1.6) + 10(3.2) = 35.2kN$ — 35.2kN

But $R_A = R_B$ (symmetrical arrangement)

$R_A + R_B = 2R_A$

$R_A = 35.2/2 = 17.6kN$ — 17.6kN

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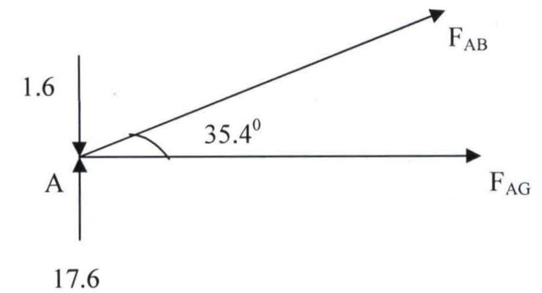
<td>Joint A</td>
<td>


Fig 4.14: Roof analysis Joint A

$\sum Fy = 0$

:. $-1.6+17.6+F_{AB}\sin 35.4^0 = 0$

$F_{AB} = \dfrac{-16}{\sin 35.4} = -27.62kN$

$\sum Fx = 0$

$F_{AG}+F_{AB}co35.4 = 0$

$F_{AG} = -F_{AB}\cos 35.4 = -(-27.62\cos 35.4) = 22.51kN(T)$
</td>
<td>
-27.62kN(C)

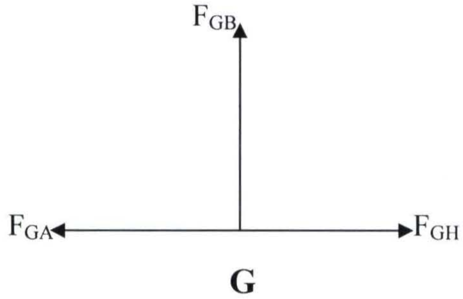
22.51kN(T)
</td>
</tr>
<tr>
<td>Joint G</td>
<td>


Fig 3.15: Roof analysis Joint G

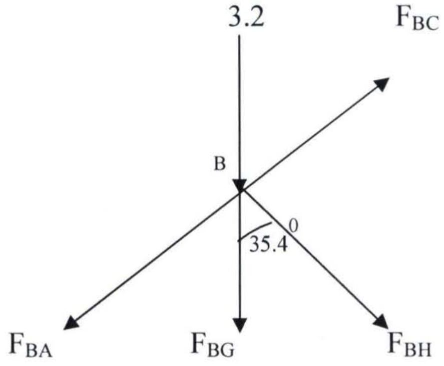
$\sum F_y = 0$, $F_{GB} = 0$

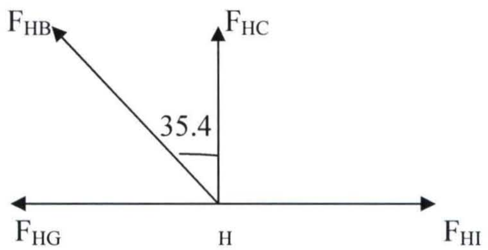
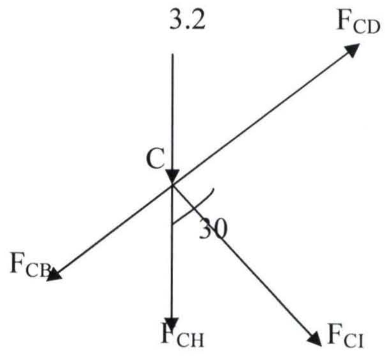
$\sum Fx = 0$,

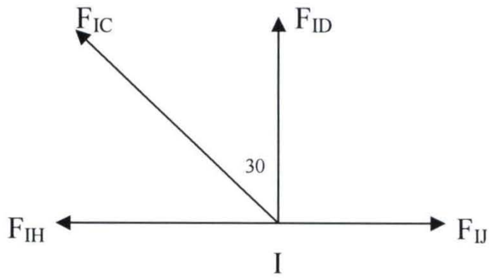
$-F_{GA}+F_{GH} = 0$, $F_{GA} = F_{GH}$

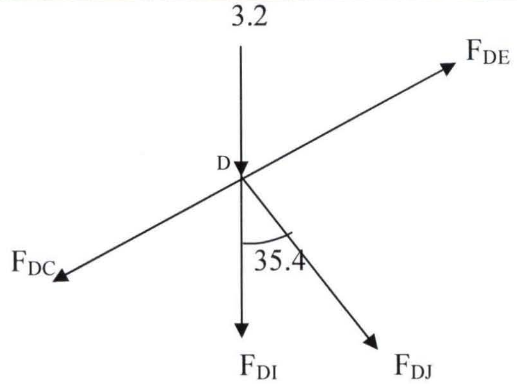
But $F_{GA} = F_{AG}$, $F_{GA} = 22.51kN$

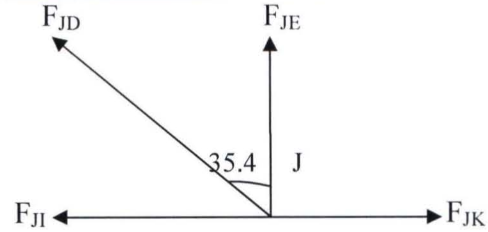
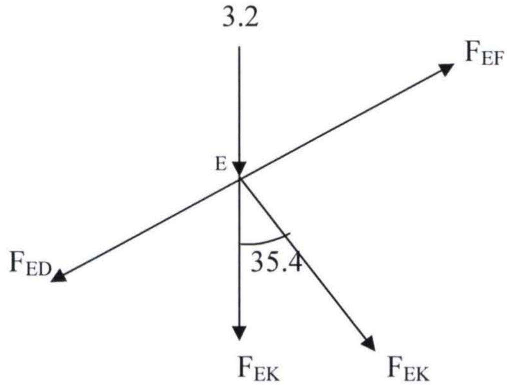
hence $F_{GH} = 22.51kN$
</td>
<td>
22.51 (T)
</td>
</tr>
</table>

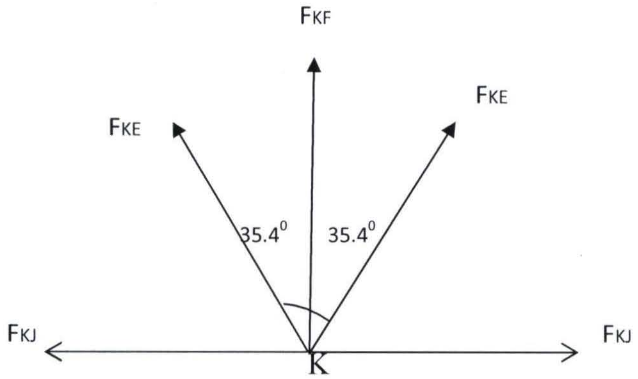
Joint B	 **Fig 4.16: Roof analysis Joint B** $\sum F_y = 0$, $-3.2 - F_{BG} - F_{BA}\sin35.4 - F_{BH}\cos35.4 + F_{BC}\sin35.4 = 0$ $-3.2-(-27.62\sin35.4)-F_{BH}\cos35.4+F_{BC}\sin35.4=0$ $12.8-F_{BH}\cos35.4+F_{BC}\sin335.4=0$ $F_{BH}=\frac{F_{BC}sin35.4+12.8}{cos35.4}$........................1.1 $\sum F_X = 0$ $F_{BC}\cos35.4 + F_{BH}\sin35.4- F_{BA}co35.4 = 0$ $F_{BC}\cos35.4 + F_{BH}\sin3.4- (-27.62\cos35.4) =0$ $F_{BC}\cos35.4+ F_{BH}\sin35.4+22.51 =0$ $F_{BH} = -\frac{(F_{BC}cos35.4+22.51)}{sin35.4}$.....................1.2 Equating 1.1 and 1.2 $-\frac{(F_{BC}cos35.4+22.51)}{sin35.4} = \frac{F_{BC}sin35.4+12.8}{cos35.4}$ Cross multiplying $-(0.82F_{BC}cos35.4 + 18.35) =$ $(0.58F_{BC}sin35.4 + 7.41)$ $0.34F_{BC} + 7.41= - (0.67F_{BC}+ 18.35)$ $-0.67F_{BC} - 0.34F_{BC}= 7.41 + 18.35$	

	$-1.01F_{BC} = 25.76$	
	$F_{BC} = -25.50kN$	-25.50kN(C)
	$F_{BH} = \frac{F_{BC}sin35.4+12.8}{cos35.4} = \frac{-25.5sin35.4+12.8}{cos35.4}$	
	$F_{BH} = \frac{-14.77+12.8}{0.82} = -33.62kN$	-33.62kN(C)
JOINT H	 **Fig 4.17: Roof analysis Joint H**	
	$\sum F_y = 0$	
	$F_{HC} + F_{HB}cos35.4 = 0$	
	$F_{HC} = -F_{HB}cos35.4 = -(-33.62cos35.4) = 27.41kN$	27.41kN(T)
	$\sum F_X = 0$	
	$F_{HI} - F_{HG} - F_{HB}sin35.4 = 0$	
	$F_{HI} = 22.51 - 33.62sin35.4$	
	$= 22.51 - 19.48 = 3.03kN$	3.03kN(T)
JOINT C	 **Fig 4.18: Roof analysis Joint C**	

	$\sum F_Y = 0$ $-3.2 - F_{CH} - F_{CB}\sin 35.4 - F_{CI}\cos 30 + F_{CD}\sin 35.4 = 0$ $-3.2 - 27.41 - (-25.5\sin 30) - F_{CI}\cos 30 + F_{CD}\sin 35.4 = 0$ $-15.84 - F_{CI}\cos 30 + F_{CD}\sin 35.4 = 0$ $F_{CI} = \frac{F_{CD}\sin 35.4 - 15.84}{\cos 30}$ ....2.1 $\sum F_X = 0$ $F_{CB}\cos 35.4 + F_{CI}\sin 35.4 - F_{CD}\cos 35.4 = 0$ $-F_{CD}\cos 35.4 + F_{CI}\sin 35.4 - 20.79 = 0$ $F_{CI} = \frac{(F_{CD}\cos 35.4 + 20.79)}{\sin 35.4}$ ....2.2 Equating 2.1 and 2.2 $\frac{(F_{CD}\cos 35.4 + 20.79)}{\sin 35.4} = \frac{F_{CD}\sin 35.4 + 15.84}{\cos 30}$ $0.37F_{CD} = -8.82$	
	$F_{CD} = \frac{-8.82}{0.37} = -23.84kN$	-23.84kN(C)
	$F_{CI} = \frac{-23.84\sin 35.4 + 15.84}{co35.4} = -2.49kN$	-2.49kN(C)
JOINT I	 **Fig 4.19: Roof analysis Joint I**	
	$\sum F_Y = 0$ $F_{ID} + F_{IC}\cos 35.4 = 0$ $F_{ID} = -F_{IC}\cos 35.4 = -(-2.49\cos 35.4) = 2.03kN$	2.03kN(T)
	$\sum F_X = 0$ $F_{IJ} - F_{IH} - F_{IC}\sin 35.4 = 0$ $F_{IJ} = 3.03 - 2.49\sin 35.4 = 1.58kN$	1.58kN(T)

JOINT D	 **Fig 4.19.1: Roof analysis Joint D** $\sum F_Y=0$ $-3.2 - F_{DI} - F_{DJ}\cos35.4 - F_{DC}\sin35.4 + F_{DE}\sin35.4 = 0$ $-3.2 - 2.03 - F_{DJ}\cos35.4 - (-23.84\sin35.4) + F_{DE}\sin35.4 = 0$ $8.58 - F_{DJ}\cos35.4 + F_{DE}\sin35.4 = 0$ $F_{DE} = \frac{F_{DJ}cos35.4-8.58}{sin35.4}$ ....3.1 $\sum F_X = 0$ $-F_{DC}\cos35.4 + F_{DJ}\sin35.4 + F_{DE}\cos35.4 = 0$ $F_{DE}\cos35.4 + F_{DJ}\sin35.4 - (-23.84\cos35.4) = 0$ $F_{DE} = \frac{-(F_{DJ}sin35.4+19.43)}{cos35.4}$ ....3.2 Equating 3.1 and 3.2 $\frac{F_{DJ}cos35.4-8.58}{sin35.4} = \frac{-(F_{DJ}sin35.4+19.43)}{cos35.4}$ $0.67F_{DJ} - 6.99 = -0.34F_{DJ} - 11.26$ $1.01F_{DJ} = -4.27$ $F_{DJ} = \frac{-4.27}{1.01} = -4.23\text{kN}$ Substituting for F_{DJ} in equation 3.1 $F_{DE} = \frac{-4.23cos35.4-8.58}{sin35.4} = \frac{-12.03}{0.58} = -20.74$	-4.23kN(C) -20.74kN(C)

JOINT J	 **Fig 4.19.2: Roof analysis Joint J**	
	$\sum F_Y=0$ $F_{JE}+F_{JD}\cos35.4=0$ $F_{JE}= -F_{JD}\cos35.4 = 4.23\cos35.4 = 3.45kN$	3.45 (T)
	$\sum F_X=0$ $-F_{JI}+F_{JK}- F_{JD}\sin35.4 =0$ $F_{JK} = -2.45 + 1.58 = -0.87kN$	0.8kN(C)
JOINT E	 **Fig 4.19.3: Roof analysis Joint E**	
	$\sum F_Y=0$ $-3.2 -F_{EJ}- F_{EK}\cos35.4-F_{ED}\sin35.4+F_{EF}\sin35.4=0$ $-3.2-3.45-F_{EK}\cos35.4-(-20.038\sin35.4)+F_{EF}\sin35.4= 0$ $-6.65-F_{EK}\cos35.4+ 11.60+ F_{EF}\sin35.4 = 0$ $F_{DE} =\frac{F_{EK}\cos35.4-4.95}{\sin35.4}$ 4.1	
	$\sum F_X= 0$ $-F_{ED}\cos35.4+F_{EK}\sin35.4+F_{EF}\cos35.4=0$ $+20.03\cos35.4+F_{EK}\sin35.4+F_{EF}\cos35.4 =0$ $F_{EF} = \frac{- F_{EK}\sin35.4-16.33}{\cos35.4}$ 4.2	

	Equating 4.1 and 4.2 $$\frac{F_{EK}cos35.4-4.95}{sin35.4} = \frac{-(F_{EK}sin35.4+16.33)}{cos35.4}$$ $0.67F_{EK} - 4.03 = -0.34F_{EK} - 9.46$ $1.01F_{EK} = 13.49$ $F_{EK} = \frac{13.49}{1.01} = 13.36kN$	13.36kN (T)
	Substituting for F_{EK} in equation 4.1 $F_{EF} = \frac{F_{EK}cos35.4-4.95}{sin35.4}$ $F_{EF} = \frac{13.36cos35.4-4.95}{sin35.4} = \frac{5.94}{0.58} = 10.25kN$	10.25kN (T)
JOINT K	 **Fig 4.19.4: Roof analysis Joint K**	
	$\sum F_Y=0$ $F_{KF} + F_{KE}cos35.4 + F_{KE}cos35.4 = 0$ $F_{KF} = 13.36cos35.4 + 13.36cos35.4$ $F_{KF} = 21.78kN$	21.78 (T)
	Member Internal Forces F_{AB} = -27.62kN C (Compression) F_{AG} = 22.51kN T (Tension) F_{GH} = 22.51kN T (Tension) F_{GB} = 0 F_{BC} = -25.50kN C (Compression)	

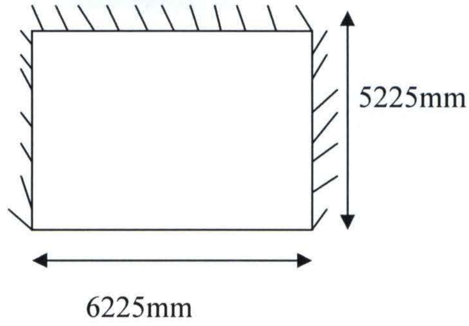
	F_{BH} = -33.62kN C (Compression) F_{HC} = 27.41kN T(Tension) F_{HI} = 3.03kN T(Tension) F_{CD} = -23.84kN C (Compression) F_{CI} = -2.49kN C (Compression) F_{ID} = 2.03kN T(Tension) F_{IJ} = 1.58kN T(Tension) F_{DJ} = -4.23kN C (Compression) F_{DE} = -20.74kN C (Compression) F_{JE} = 3.45kN T(Tension) F_{JK} = 0.80kN T(Tension) F_{EK} = 13.36kN T(Tension) F_{EF} = 10.25kN T(Tension) F_{KF} = 21.78kN T(Tension)	
	Design of critical member in tension **<u>Member AG</u>** Force (F) = 27.41kN Area (A) = 50mm x 150mm = 7500mm^2 Stress = F/A = 27410/7500 = 3.65N/ mm^2	
B.S.5268: (2002)	Permissible stress for grade 68 is 14N/mm^2 Therefore, section is adequate.	3.65N/ mm^2 Section ok
	Design of critical member in compression **<u>Member CD</u>** Force (F) = 33.62kN Area (A) = 50mm x 150mm = 7500mm^2 Stress = F/A = 33620/7500 = 4.48N/ mm^2	4.48N/ mm^2
	Permissible stress for grade 68 is 14N/mm^2 Therefore, section is adequate.	Section ok

4.2 Design of Slab:

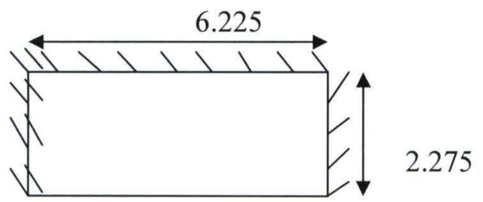
The design procedure is carried out in this order:

- Decide on the material stresses to be used that is f_{cu} and f_y
- Assume overall thickness h of slab
- Estimate the characteristics loads Q_k and G_k per unit area
- Calculate the design loads
- Determine the ultimate bending moment M
- Choose the appropriate concrete cover
- Calculate the effective depth 'd'
- Calculate the reinforcement area "A_{Sreq}'
- Select the reinforcement area to be provided
- Check the span/effective depth ratio

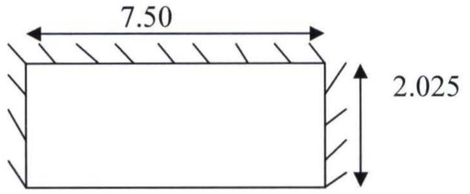
REFERENCE	CALCULATIONS	OUTPUT
Mosley& Bungey, Appendix	Characteristics strength of concrete (F_{cu}) = 25N/mm^2 Yield strength (F_y) =460N/mm^2 Diameter of steel (θ) = 12mm Slab thickness (h) =150mm Unit weight of concrete =24kN/m^3 Concrete cover =20mm Effective depth ratio =26 Unit weight of Terrazzo =22kN/m^3 Unit weight of Cement (mortar) =20kN/m^3 Live Loads (Classrooms) =3.0kN/m^2	
	CALCULATIONS **Slab Design** **Loading** Concrete self weight = 0.15x24.0 =3.6kN/m^2 Finishes =1.0kN/m^2	
Mosley& Bungey, Appendix	Partition allowance =2.5kN/m^2 Total dead load =7.1kN/m^2 Live load =3.0kN/m^2	GK = 7.1kN/m^2 QK = 3.0kN/m^2
	Design load =1.4Gk + 1.6Qk =1.4(7.1) + 1.6(3) =14.74kN/m/m run Say = 15kN/m/m run	**Design load** 15kN/m/m run
Table 3.14 BS8110 1:1997	$\frac{L_y}{L_x} < 2$ (2-way) $\frac{L_y}{L_x} > 2$ (1-way)	
	Effective depth dx = 150 – 20 – 12/2 = 124mm (in the direction of short spam) dy = 150 – 20 – 12 - 12/2=112mm (in the direction of long spam)	d= 124mm d = 112mm

	PANEL ONE (1)  **Fig 4.2:1. Panel 1**	
Table 3.14 BS8110 1:1997	$\frac{L_y}{L_X} = \frac{6225}{5225} = 1.2$ (2-way)	
	Moment coefficient	
	Short span	
Table 3.14 BS8110 1:1997	Negative moment coefficient @ continuous edge = 0.056 Positive moment coefficient @ mid-span = 0.042	
	Long span	
	Negative moment coefficient @ continuous edge = -0.037	
	Positive moment coefficient @ mid-span = 0.028	
	Design Moment	
	Short span	
Clause 3.5.3.4	Continuous edge $M_{sx} = \beta_{sx} n l x^2$	
	$M_{sx} = -0.056 \times 15 \times 5.225^2 = -22.93$kNm	-22.93kNm
	Effective depth	
	d = thickness – cover – θ/2	
	d = 150 – 20 – 12/2 = 124m	
Clause 3.4.4.4	$K = \frac{M}{F_{cu} b d^2}$	
	$= \frac{22.93 \times 10^6}{25 \times 1000 \times (124)^2} = 0.060 < 0.156$	
	$La = 0.5 + \sqrt{0.25 - \frac{k}{0.9}}$	

	La = 0.93 < 0.95	La = 0.93
	Z= lad = 0.93 x 124 = 115.32mm	Z=115.32mm
	$As_{req} = \frac{m}{0.95fyZ} = \frac{22.93\ x10^6}{0.95x460x115.32} = 455mm^2/m$	provide $566mm^2/m$
	$As_{min} = 0.13\%bh$	
	$= \frac{0.13\times1000\times150}{100} = 195mm^2$	
	Provide = T_{12} @ 200 c/c Top ($As_{provide} = 566mm^2/m$	T_{12}@200c/cT
	<u>Mid-span</u>	
	$M_{sy} = -0.042 \times 15 \times 5.225^2 = -17.20kNm$	-17.20kNm
	$K = \frac{17.20\times10^6}{25\times1000\times(124)^2} = 0.045 < 0.156$	La = 0.95
	:. La = 0.95, Z = 117.8mm	Z=117.8mm
	$As_{required} = \frac{17.20 \times 10^6}{0.95 \times 460 \times 117.8} = 334.12mm^2/m$	$334.12mm^2/m$
	Provide T_{12} @ 300 c/c Botom ($As_{provide} = 377mm^2/m$)	T_{12} @ 300 c/c B
	Long span	
	d=150 – 20 – 12 – 6 = 112mm	
Clause 3.5.3.4	<u>continuous edge</u>	
	$M = -0.037 \times 15 \times 5.225^2 = -15.15KNm$	-15.15kNm
Clause 3.4.4.4	$K = \frac{15.15 \times 10^6}{25 \times 1000 \times 112^2} = 0.048 < 0.156$	La = 0.95
	Z=lad=0.95 x 112 = 106.4	Z=106.40
	$As_{provide} = \frac{15.15 \times 10^6}{0.95 \times 460 \times 106.40} = 326mm^2$	
	Provide T_{12} @ 300 c/c Top ($As_{pv} = 377mm^2/m$)	T_{12} @ 300 c/c T.
	<u>Mid span</u>	
	$M = 0.028 \times 15 \times 5.225^2 = 11.47kNm$	11.47kNm
	$K = \frac{11.47 \times 10^6}{25 \times 1000 \times (112)^2} = 0.037 < 0.156$	

	$\therefore$ La = 0.95	La = 0.95
	Z = 106.4	
	$$As_{req} = \frac{11.47 \times 10^6}{0.95 \times 460 \times 106.4} = 247mm^2/m$$	
	Provide T_{12} @ 300 c/c Bottom ($As_{pr} = 377mm^2/m$)	T_{12} @ 300 c/c B ($As_{pr} = 377mm^2/m$)
	Deflection check	
	$\frac{M}{bd^2} = \frac{17.20\times10^6}{1000\times(124)^2} = 1.119N/mm^2$	
	$f_s = {}^2/_3 fy \frac{As_{req}}{As_{prv}} \times \frac{1}{\beta_0}$ where β = 1.0	
	$= {}^2/_3 \times 460 \times \frac{334.12}{377} \times \frac{1}{1} = 271.79N/mm^2$	
	$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^M/_{bd^2}\right)} \leq 2.0$	
	$= 0.55 + \frac{477-271.79}{120(0.9+1.119)} = 1.40$	Mf = 1.4
	$\frac{limit\ span}{depth} = MF \times 26 = 1.40\ x\ 26 = 36.4$	
	$\frac{Actual\ span}{depth} = \frac{5225}{124} = 42.14$	
	Since $\frac{actual\ span}{depth} < \frac{limiting\ span}{depth}$	
	Deflection is OK	**Deflection satisfied**
	PANEL TWO (2)	
		
	Fig. 4.2.2: Panel 2	
	$\frac{ly}{lx} = \frac{6225}{2275} = 2.74$	
	One way simply supported slab. Moment	
	$M_x \frac{WL^2}{8} = \frac{15x2.275^2}{8} = 9.70kNm$	9.70kNm
	$$K = \frac{9.70 \times 10^6}{25 \times 1000 \times (124)^2} = 0.025 < 0.156$$	

	:. La = 0.95, Z=117.8mm $As_{required} = \frac{9.70\times10^6}{0.95\times460\times117.8} = 188.43mm^2$ $As_{minimum} = 0.13\%bh = \frac{0.13\times1000\times150}{100} = 195mm^2$ Provide T_{12} @ 300 c/c ($As_{provide}$ = 377mm^2) **Deflection check** $\frac{M}{bd^2} = \frac{9.70\times10^6}{1000\times(124)^2} = 0.631$ $F_s = {}^2/_3 \times 460 \times \frac{188.43}{377} = 153.28N/mm^2$ $Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^M/_{bd^2}\right)} \leq 2.0$ $= 0.55 + \frac{477-153.28}{120(0.9+0.631)} = 2.31$ $\frac{limit\ span}{depth} = 2\times26 = 52$ $\frac{Actual\ span}{depth} = \frac{2275}{124} = 18.35$ Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$ **Deflection is OK**	La = 0.95 Z=117.8mm T_{12} @ 300 c/c As_{provi}377mm^2 Mf = 2 **Deflection satisfied**

	PANEL THREE (3)	
		
	Fig. 4.2.3: Panel 3	
	$\frac{ly}{lx} = \frac{7500}{2025} = 3.70$	
	One way simply supported slab. Moment	
	$M = \frac{WL^2}{8} = \frac{15x2.025}{8} = 7.69\text{kNm}$	7.69kN/M
	$K = \frac{7.69\times10^6}{25\times1000\times(124)^2} = 0.02 < 0.156$	La = 0.95
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{7.69\times10^6}{0.95\times460\times117.8} = 149.38mm^2$	T_{12} @ 300c/c B
	Provide T_{12} @ 300 c/c ($As_{provide} = 377mm^2$)	
	Deflection check	($As_{prov} = 377mm^2$)
	$\frac{M}{bd^2} = \frac{7.69\times10^6}{1000\times(124)^2} = 0.500$	
	$F_s = 2/3 \times 460 \times \frac{149.38}{377} = 121.51N/mm^2$	
	$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+M/_{bd^2}\right)} \leq 2.0$	
	$= 0.55 + \frac{477-121.51}{120(0.9+0.500)} = 2.12$	Mf = 2

$$\frac{limit\ span}{depth} = 2 \times 26 = 52$$

$$\frac{Actual\ span}{depth} = \frac{2275}{124} = 18.35$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$

Deflection ok

PANEL FOUR (4)

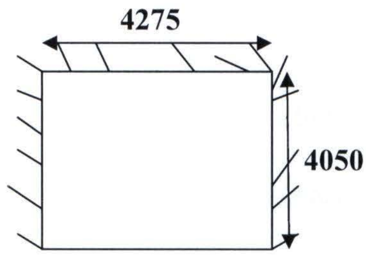


Fig. 4.2.4: Panel 4

Moment coefficient

$\frac{L_y}{L_X} = \frac{4257}{4050} = 1.1$ (2-way)

Short span

Negative moment coefficient @ continuous edge = -0.056

Positive moment coefficient @ mid-span = 0.042

Long span

Negative moment coefficient @ continuous edge = - 0.045

Positive moment coefficient @ mid-span = 0.034

<u>Design Moment</u>

Short span

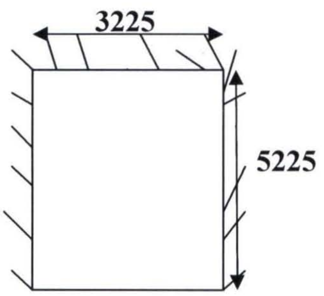
<u>Continuous edge</u>

$M_x = - 0.056 \times 15 \times 4.05^2 = -13.78$kN/m

$m_x = -13.78$kNm

$K = \frac{13.78 \times 10^6}{25 \times 1000 \times (124)^2} = 0.036 < 0.156$

La = 0.95

	Provide T_{12} @ 300 c/c top (As_{prov} = 377mm)	T_{12} @ 300 c/c T
	Deflection check	
	$\frac{M}{bd^2} = \frac{10.3\times10^6}{1000\times(124)^2} = 0.67$	
	$F_s = {}^2/_3 \times 460 \times \frac{200}{377} = 162.69N/mm^2$	
	$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^M/_{bd^2}\right)} \leq 2.0$	
	$= 0.55 + \frac{477-162.69}{120(0.9+0.67)} = 2.21$	Mf =2
	$\frac{limit\ span}{depth} = 2\times26 = 52$	
	$\frac{Actual\ span}{depth} = \frac{4050}{124} = 32.66$	
	Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok
	PANEL FIVE (5)	
		
	Fig. 4.2.5: Panel 5	
	$\frac{Ly}{Lx} = \frac{5225}{3225} = 1.6$ (2-way)	
	Short span	
	Negative moment coefficient @ continuous edge = -0.063	
	Positive moment coefficient @ mid-span = 0.047	
	Long span	
	Negative moment coefficient @ continuous edge = - 0.037	

	Positive moment coefficient @ mid-span = 0.028	
	<u>Design Moment</u>	
	Short span	
	<u>Continuous edge</u>	
	M_x = - 0.063 x 15 x 3.225^2 = -9.83kNm	Mx = -9.83kNm
	$K = \frac{9.83\times10^6}{25\times1000\times(124)^2} = 0.026 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{9.83\times10^6}{0.95\times460\times117.8} = 343mm^2$	
	Provide T_{12} @ 300 c/c top (As_{prov} = 337mm)	T_{12} @ 300 c/c T
	<u>Mid-span</u>	
	M_x = 0.047 x 15 x 3.225^2 =7.3kNm	7.3kNm
	$K = \frac{7.3\times10^6}{25\times1000\times(124)^2} = 0.033 < 0.156$	La = 0.95
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{7.3\times10^6}{0.95\times460\times117.8} = 142mm^2$	
	Provide T_{12} @ 300 c/c bottom (As_{prov} = 377mm)	T_{12} @ 300c/c B
	Long span	
	<u>Continuous edge</u>	
	My = - 0.037 x 15 x 3.225^2 = -5.77kNm	
	$K = \frac{5.77\times10^6}{25\times1000\times(112)^2} = 0.018 < 0.156$	La = 0.95
	:. La = 0.95, Z=117.8mm	Z=117.8mm

	$As_{required} = \frac{5.77\times10^6}{0.95\times460\times112} = 118mm^2$	
	Provide T_{12} @ 300 c/c Top ($As_{prov} = 377mm^2$)	T_{12} @ 300c/c T
	<u>Mid span</u>	
	My = $0.028 \times 15 \times 3.225^2 = 4.4kNm$	My =4.4kNm
	$$K = \frac{4.4\times10^6}{25\times1000\times(112)^2} = 0.014 < 0.156$$	
	:. La = 0.95, Z=117.8mm	La = 0.95 Z=117.8mm
	$As_{required} = \frac{4.4\times10^6}{0.95\times460\times117.8} = 85mm^2$	
	Provide T_{10} @ 300 c/c bottom ($As_{prov} = 377mm^2$)	T_{12} @ 300c/c B
	Deflection check	
	$\frac{M}{bd^2} = \frac{7.3\times10^6}{1000\times(124)^2} = 0.47$	
	$F_s = {}^2/_3 \times 460 \times \frac{142}{377} = 155.51N/mm^2$	
	$$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^{M}/_{bd^2}\right)} \leq 2.0$$	
	$$= 0.55 + \frac{477-155.51}{120(0.9+0.47)} = 2.50$$	Mf =2
	$$\frac{limit\ span}{depth} = 2\times26 = 52$$	
	$$\frac{Actual\ span}{depth} = \frac{3223}{124} = 25.99$$	
	Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok

PANEL SIX (6)

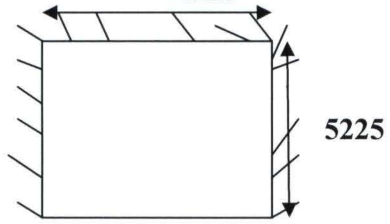


Fig.4.2.6 PANEL 6

$\frac{ly}{lx} = \frac{6225}{5225} = 1.2$ (2 – way)

<u>Moment coefficient</u>

Short span

Negative moment coefficient @ continuous edges = -0.056

Positive moment coefficient @ mid-span = 0.042

Long span

Negative moment coefficient @ continuous edge = -0.037

Positive moment coefficient @ mid-span = 0.028

<u>Design Moment</u>

Short span

<u>Continuous edge</u>

M_x = - 0.056 x 15 x 5.225^2 = -22.93kN/m

$K = \frac{22.93 \times 10^6}{25 \times 1000 \times 124^2} = 0.060 < 0.156$

:. La = 0.95, Z=117.8mm

Z=117.8mm

$As_{required} = \frac{22.93 \times 10^6}{0.95 \times 460 \times 117.8} = 445mm^2$

Provide T_{12} @ 250 c/c top (As_{prov} = 452mm)

T_{12} @ 250 c/c T

	Mid-span	
	Mx = 0.042 x 15 x 5.2252 = 17.2kNm	17.2kNm
	$K = \frac{17.2\times10^6}{25\times1000\times(124)^2} = 0.045 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=118.75mm
	$As_{required} = \frac{17.2\times10^6}{0.95\times460\times117.8} = 334mm^2$	
	Provide T_{12} @ 300 c/c bottom (As_{prov} = 377mm)	T_{12} @ 300 c/c B
	Long span	
	Continuous edge	
	My = - 0.037 x 15 x 5.225^2 = -15.15kNm	My = -15.15kNm
	$K = \frac{15.15\times10^6}{25\times1000\times(112)^2} = 0.048 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{15.15\times10^6}{0.95\times460\times117.8} = 294$	
	Provide T_{12} @ 300 c/c Top (As_{prov} = 377mm^2)	T_{12} @ 300 c/c T
	Mid span	
	My = 0.028 x 15 x 5.225^2 = 11.47kNm	
	$$K = \frac{11.47 \times 10^6}{25 \times 1000 \times (112)^2} = 0.037 < 0.156$$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{11.47\times10^6}{0.95\times460\times117.8} = 223mm^2$	T_{12} @ 300 c/c B
	Provide T_{12} @ 300 c/c Top (As_{prov} = 377mm^2)	

Deflection check

$\frac{M}{bd^2} = \frac{17.2\times10^6}{1000\times(124)^2} = 1.12$

$F_s = {}^2/_3 \times 460 \times \frac{334}{377} = 271.69N/mm^2$

$$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^M/_{bd^2}\right)} \leq 2.0$$

Mf =2

$$= 0.55 + \frac{477 - 155.51}{120(0.9 + 1.12)} = 2.50$$

$$\frac{limit\ span}{depth} = 2 \times 26 = 52$$

$$\frac{Actual\ span}{depth} = \frac{5225}{124} = 42.14$$

Deflection ok

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$

PANEL SEVEN (7)

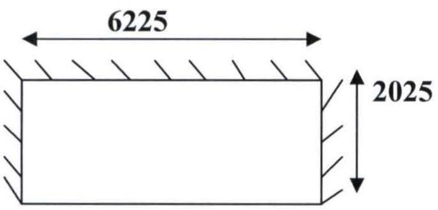


Fig. 4.2.7: Panel 7

$\frac{ly}{lx} = \frac{6225}{2025} = 3.07$

One way simply supported slab. Moment

$M = \frac{WL^2}{8} = \frac{15x2.025x\ 2.025}{8} = 7.69$kNm

M = 7.69kNm

$$K = \frac{7.69 \times 10^6}{25 \times 1000 \times (124)^2} = 0.02 < 0.156$$

:. La = 0.95, Z=117.8mm

$As_{required} = \frac{7.69\times10^6}{0.95\times460\times117.8} = 149.38mm^2$

Prov T_{12}@300c/cB

Provide T_{12} @ 300c/c Bottom (As_{prov} = 377 mm^2)

Deflection check

$$\frac{M}{bd^2} = \frac{7.69\times10^6}{1000\times(124)^2} = 0.500$$

$$F_s = {}^2\!/_3 \times 460 \times \frac{149.38}{377} = 121.51N/mm^2$$

$$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^{M}\!/_{bd^2}\right)} \leq 2.0$$

$$= 0.55 + \frac{477-121.51}{120(0.9+0.5)} = 2.67$$

Mf=2

$$\frac{limit\ span}{depth} = 2\times 26 = 52$$

$$\frac{Actual\ span}{depth} = \frac{2025}{124} = 16.33$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$

Deflection ok

PANEL EIGHT (8)

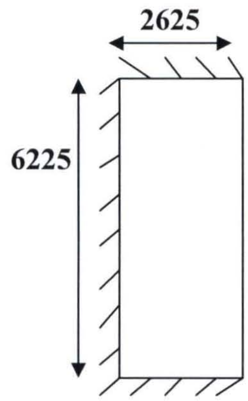


Fig. 4.2.8: Panel 8

$$\frac{ly}{lx} = \frac{6225}{2625} = 2.37$$

One way simply supported slab. Moment

$$M = \frac{WL^2}{8} = \frac{15x2.6252x2.625}{8} = 12.92\text{kNm}$$

M = 12.92kNm

$$K = \frac{12.92\times10^6}{25\times1000\times(124)^2} = 0.034 < 0.156$$

:. La = 0.95, Z=117.8mm

$$As_{required} = \frac{12.92\times10^6}{0.95\times460\times117.8} = 251mm^2$$

Provide T_{12} @ 300c/c Bottom (As_{prov} = 377 mm^2)

Prov T_{12}@300c/cB

Deflection check

$$\frac{M}{bd^2} = \frac{12.92\times10^6}{1000\times(124)^2} = 0.84$$

$$F_s = {}^2/_3 \times 460 \times \frac{251}{377} = 204.17N/mm^2$$

$$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^M/_{bd^2}\right)} \le 2.0$$

$$= 0.55 + \frac{477-204.17}{120(0.9+0.84)} = 1.86$$

Mf=1.86

$$\frac{limit\ span}{depth} = 1.86\times26 = 48.36$$

$$\frac{Actual\ span}{depth} = \frac{2625}{124} = 21.17$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$

Deflection ok

PANEL NINE (9)

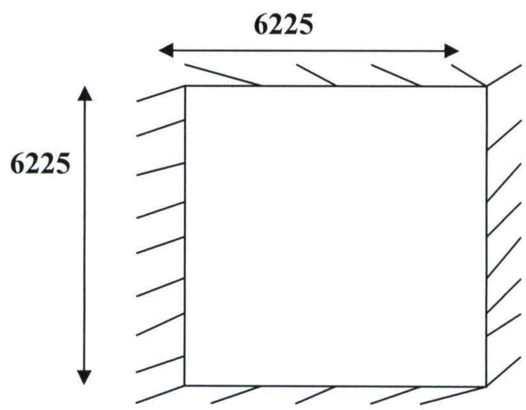


Fig. 4.2.9: Panel 9

$\frac{ly}{lx} = \frac{6225}{6225} = 1.0$ (Design two way)

	Moment coefficient	
	Short span	
	Negative moment coefficient @ continuous edges = -0.031	
	Positive moment coefficient @ mid-span = 0.024	
	Long span	
	Negative moment coefficient @ continuous edge = -0.032	
	Positive moment coefficient @ midspan = 0.024	
	Design Moment	
	Short span	
	Continuous edge	
	M_x = - 0.031 x 15 x 6.225^2 = -18.02kNm	Mx = -18.02kNm
	$K = \frac{18.02 \times 10^6}{25 \times 1000 \times (124)^2} = 0.047 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{18.02 \times 10^6}{0.95 \times 460 \times 117.8} = 350mm^2$	
	Provide T_{12} @300 c/c top (As_{prov} = 377mm)	T_{12} @300 c/cT
	Mid-span	
	M_x = 0.024 x 15 x 6.225^2 =13.95kNm	
	$K = \frac{13.95 \times 10^6}{25 \times 1000 \times (124)^2} = 0.036 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{13.95 \times 10^6}{0.95 \times 460 \times 117.8} = 271mm^2$	
	Provide T_{12} @ 300 c/c bottom (As_{prov} = 377mm)	T_{12} @300 c/cB

Long span

Continuous edge

$As_{required} = \frac{13.95\times10^6}{0.95\times460\times117.8} = 271mm^2$

Provide T_{12} @ 300 c/c bottom (As_{prov} =377mm^2) — T_{12}@300 c/cB

Deflection check

$$\frac{M}{bd^2} = \frac{13.95\times10^6}{1000\times(124)^2} = 0.91$$

$F_s = {}^2/_3 \times 460 \times \frac{271}{377} = 220.44N/mm^2$

$$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$

$$= 0.55 + \frac{477 - 220.44}{120(0.9 + 0.91)} = 2.0$$

Mf = 2.0

$$\frac{limit\ span}{depth} = 2.0 \times 26 = 52$$

$$\frac{Actual\ span}{depth} = \frac{6225}{124} = 50.20$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$ — Deflection ok

PANEL TEN (10)

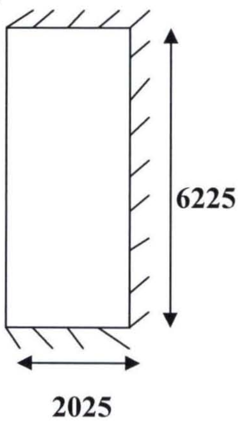


Fig. 4.2.10 PANEL 10

$\frac{ly}{lx} = \frac{6225}{2025} = 3.07$	
One way simply supported slab.	
Moment, $M = \frac{WL^2}{8} = \frac{15x2.0252x2.025}{8} = 7.69\text{kNm}$	
$$K = \frac{7.69 \times 10^6}{25 \times 1000 \times (124)^2} = 0.02 < 0.156$$	
:. La = 0.95, Z=117.8mm	
$As_{required} = \frac{7.69\times10^6}{0.95\times460\times117.8} = 149mm^2$	T_{12} @ 300c/c B
Provide T_{12} @ 300c/c Bottom ($As_{prov} = 377\ mm^2$)	
Deflection check	
$$\frac{M}{bd^2} = \frac{7.69 \times 10^6}{1000 \times (124)^2} = 0.50$$	
$F_s = {}^2/_3 \times 460 \times \frac{149}{377} = 121.20N/mm^2$	
$$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$	
$$= 0.55 + \frac{477 - 121.20}{120(0.9 + 0.50)} = 2.67$$	Mf = 2
$$\frac{limit\ span}{depth} = 2.0 \times 26 = 52$$	
$$\frac{Actual\ span}{depth} = \frac{2025}{124} = 16.33$$	
Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok

PANEL ELEVEN (11)

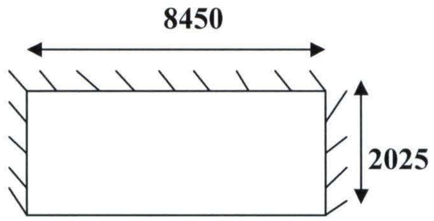


Fig.4.2.11 PANEL11

$\frac{ly}{lx} = \frac{8450}{2025} = 4.17$

One way simply supported slab.

Moment, $M = \frac{WL^2}{8} = \frac{15x2.0252x2.025}{8} = 12.92\text{kNm}$

M = 12.92kNm

$$K = \frac{12.92 \times 10^6}{25 \times 1000 \times (124)^2} = 0.03 < 0.156$$

:. La = 0.95, Z=117.8mm

$As_{required} = \frac{12.92 \times 10^6}{0.95 \times 460 \times 117.8} = 251mm^2$

Provide T_{12} @ 300c/c Bottom (As_{prov} = 377 mm^2)

T_{12} @ 300c/c B

Deflection check

$$\frac{M}{bd^2} = \frac{12.92 \times 10^6}{1000 \times (124)^2} = 0.84$$

$F_s = {}^2/_3 \times 460 \times \frac{251}{377} = 204.17N/mm^2$

$$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$

$$= 0.55 + \frac{477 - 204.17}{120(0.9 + 0.84)} = 1.86$$

Mf = 1.86

$$\frac{limit\ span}{depth} = 1.86 \times 26 = 48.27$$

$$\frac{Actual\ span}{depth} = \frac{2025}{124} = 16.33$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$ | Deflection ok

PANEL ELEVEN (12)

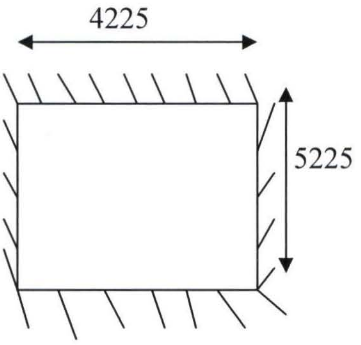


Fig.4.2.12 PANEL 12

$\frac{ly}{lx} = \frac{5225}{4225} = 1.2$

Moment coefficient

Short span

Negative moment coefficient @ continuous edges = -0.048

Positive moment coefficient @ mid-span = 0.036

Long span

Negative moment coefficient @ continuous edge = -0.037

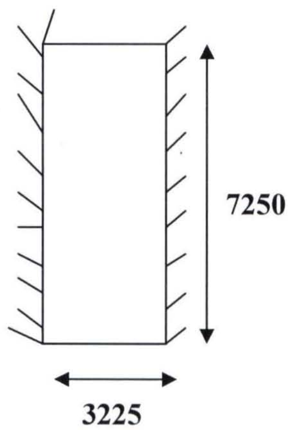
Positive moment coefficient @ mid-span = 0.028

Short span

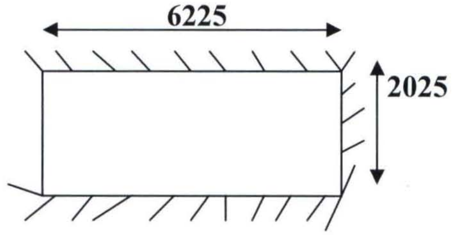
Continuous edge

$M_x = -0.048 \times 15 \times 4.225^2 = -12.85kNm$ | Mx = -12.85kNm

	$K = \frac{12.85 \times 10^6}{25 \times 1000 \times (124)^2} = 0.033 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{12.85 \times 10^6}{0.95 \times 460 \times 117.8} = 250mm^2$	
	Provide T_{12} @300 c/c top (As_{prov} = 377mm)	Prov T_{12} @300 c/c T
	<u>Mid span</u>	
	My = 0.036 x 15 x 4.225^2 = 9.6kNm	My= 9.6kNm
	$K = \frac{9.6 \times 10^6}{25 \times 1000 \times (124)^2} = 0.025 < 0.156$	
	:. La = 0.95, Z=117.8mm	
	$As_{required} = \frac{7.5 \times 10^6}{0.95 \times 460 \times 117.8} = 147mm^2$	
	Provide T_{12} @ 300 c/c B (As_{prov} = 377mm^2)	T_{12} @ 300 c/c B
	Long span	
	<u>Continuous edge</u>	
	My = - 0.037 x 15 x 4.225^2 = -9.9kNm	My= -9.9kNm
	$K = \frac{9.9 \times 10^6}{25 \times 1000 \times (124)^2} = 0.025 < 0.156$	
	:. La = 0.95, Z=117.8mm	
	:. La = 0.95, Z=117.8mm	
	Provide T_{12} @ 300 c/c Top (As_{prov} = 377mm^2)	Prov T_{12} @ 300 c/c T
	<u>Mid span</u>	
	My = 0.028 x 15 x 4.225^2 = 7.5kNm	My= 7.5kNm
	$K = \frac{7.5 \times 10^6}{25 \times 1000 \times (124)^2} = 0.020 < 0.156$	
	:. La = 0.95, Z=117.8mm	

	$As_{required} = \frac{7.5\times10^6}{0.95\times460\times117.8} = 147mm^2$	Prov T_{12} @ 300 c/c B
	Provide T_{12} @ 300 c/c B ($As_{prov} = 377mm^2$)	
	Deflection check	
	$$\frac{M}{bd^2} = \frac{9.6\times10^6}{1000\times(124)^2} = 0.62$$	
	$F_s = {}^2/_3 \times 460 \times \frac{147}{377} = 119.58N/mm^2$	
	$$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$	
	$$= 0.55 + \frac{477 - 119.58}{120(0.9 + 0.62)} = 2.51$$	Mf = 2
	$$\frac{limit\ span}{depth} = 2.0 \times 26 = 52$$	
	$\frac{Actual\ span}{depth} = \frac{2025}{124} = 16.33$	
	Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok
	PANEL THIRTEEN (13)	
		
	Fig.4.2.13 PANEL 13	
	$\frac{ly}{lx} = \frac{7250}{3225} = 2.25$	

	One way simply spanning slab. Moment, M = $\frac{WL^2}{8} = \frac{15x3.225x3.225}{8}$ = 19.5kNm $$K = \frac{19.5 \times 10^6}{25 \times 1000 \times (124)^2} = 0.051 < 0.156$$:. La = 0.95, Z=117.8mm $As_{required} = \frac{19.5\times10^6}{0.95\times460\times117.8} = 379mm^2$ Provide T_{12} @ 300c/c Bottom (As_{prov} = 452 mm^2) **Deflection check** $$\frac{M}{bd^2} = \frac{19.5 \times 10^6}{1000 \times (124)^2} = 1.27$$ $F_s = {}^2/_3 \times 460 \times \frac{379}{452} = 257.14N/mm^2$ $$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$ $$= 0.55 + \frac{477 - 257.14}{120(0.9 + 1.27)} = 1.39$$ $$\frac{limit\ span}{depth} = 1.39 \times 26 = 36.25$$ $$\frac{Actual\ span}{depth} = \frac{3225}{124} = 26.01$$ Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	M = 19.5kN Prov T_{12} @ 300c/c B As_{prov} = 452 mm^2 Mf = 1.39 Deflection ok

	PANEL FOURTEEN (14)	
		
	Fig.4.2.14 PANEL 14	
	$\frac{ly}{lx} = \frac{6225}{2025} = 3.07$	
	One way simply spanning slab.	
	Moment, $M = \frac{WL^2}{8} = \frac{15x2.025x2.025}{8} = 7.7kNm$	M = 7.7kNm
	$K = \frac{7.7 \times 10^6}{25 \times 1000 \times (124)^2} = 0.020 < 0.156$	
	:. La = 0.95, Z=117.8mm	
	$As_{required} = \frac{7.7 \times 10^6}{0.95 \times 460 \times 117.8} = 150mm^2$	Prov T_{12} @ 300c/c B
	Provide T_{12} @ 300c/c Bottom ($As_{prov} = 377\ mm^2$)	$As_{prov} = 377\ mm^2$
	Deflection check	
	$\frac{M}{bd^2} = \frac{7.7 \times 10^6}{1000 \times (124)^2} = 0.50$	
	$F_s = {}^2/_3 \times 460 \times \frac{150}{377} = 122.02N/mm^2$	
	$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$	
	$= 0.55 + \frac{477 - 122.02}{120(0.9 + 0.5)} = 2.66$	Mf = 2
	$\frac{limit\ span}{depth} = 2 \times 26 = 52$	
	$\frac{Actual\ span}{depth} = \frac{2025}{124} = 16.33$	
	Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok

PANEL FIFTEEN (15)

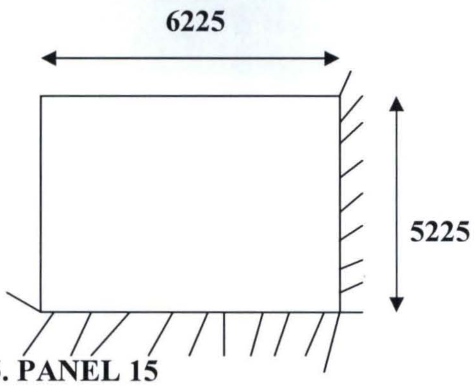


Fig.4.2.15. PANEL 15

$\frac{ly}{lx} = \frac{6225}{5225} = 1.2$

Short span

Negative moment coefficient @ continuous edges = -0.056

Positive moment coefficient @ mid-span = 0.042

Long span

Negative moment coefficient @ continuous edge = -0.037

Positive moment coefficient @ mid-span = 0.028

Short span

Continuous edge

M_x = - 0.056 x 15 x 5.225^2 = -22.93kNm

M = -22.93kNm

$$K = \frac{22.93 \times 10^6}{25 \times 1000 \times (124)^2} = 0.060 < 0.156$$

:. La = 0.95,   Z=117.8mm

Z=117.8mm

$As_{required} = \frac{22.93 \times 10^6}{0.95 \times 460 \times 117.8} = 445mm^2$

Prov T_{12} @300 c/c T

Provide T_{12} @300 c/c top (As_{prov} = 452mm)

(As_{prov} = 452mm)

	Mid span	
	My = 0.042 x 15 x 5.225^2 = 17.20kNm	= 17.20kNm
	$$K = \frac{17.20 \times 10^6}{25 \times 1000 \times (124)^2} = 0.045 < 0.156$$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{17.20\times10^6}{0.95\times460\times117.8} = 334mm^2$	Prov T_{12} @300c/cB
	Provide T_{12} @ 300 c/c B (As_{prov} = 377mm^2)	(As_{prov} = 377mm^2)
	Long span	
	Continuous edge	
	My = - 0.037 x 15 x 5.225^2 = -15.15kNm	M = -15.15Knm
	$$K = \frac{15.15 \times 10^6}{25 \times 1000 \times (124)^2} = 0.039 < 0.156$$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{15.15\times10^6}{0.95\times460\times117.8} = 294mm^2$	
	Provide T_{12} @ 300 c/c Top (As_{prov} = 377mm^2)	Prov T_{12} @ 300 c/c T
	Mid span	(As_{prov} = 377mm^2)
	My = 0.028 x 15 x 5.225^2 = 11.47kNm	M = 11.47kNm
	$$K = \frac{11.47 \times 10^6}{25 \times 1000 \times (124)^2} = 0.030 < 0.156$$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{11.47\times10^6}{0.95\times460\times117.8} = 223mm^2$	Prov T_{12} @ 300 c/c B
	Provide T_{12} @ 300 c/c B (As_{prov} = 377mm^2)	(As_{prov} = 377mm^2)

Deflection check

$$\frac{M}{bd^2} = \frac{17.20 \times 10^6}{1000 \times (124)^2} = 1.12$$

$$F_s = {}^2/_3 \times 460 \times \frac{334}{377} = 271.69N/mm^2$$

$$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$

$$= 0.55 + \frac{477 - 271.69}{120(0.9 + 1.12)} = 1.40$$

Mf = 1.4

$$\frac{limit\ span}{depth} = 1.40 \times 26 = 36.40$$

$$\frac{Actual\ span}{depth} = \frac{5225}{124} = 42.14$$

Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$

Deflection ok

PANEL SIXTEEN (16)

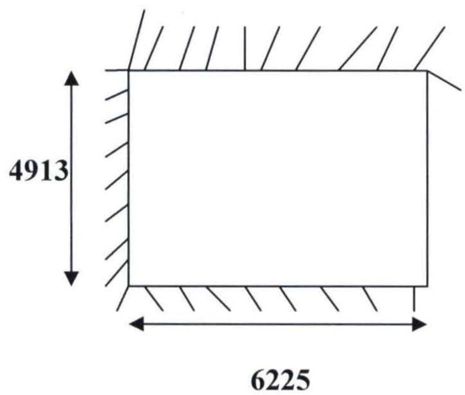


Fig. 4.2.16. PANEL 16

$\frac{ly}{lx} = \frac{6225}{4913} = 1.3$

Short span

Negative moment coefficient @ continuous edges = -0.052

Positive moment coefficient @ mid-span = 0.039

	Long span Negative moment coefficient @ continuous edge = -0.037 Positive moment coefficient @ mid-span = 0.028 **Short span** Continuous edge M_x = - 0.052 x 15 x 4.913^2 = -18.83kNm $$K = \frac{18.83 \times 10^6}{25 \times 1000 \times (124)^2} = 0.049 < 0.156$$:. La = 0.95, Z=117.8mm $As_{required} = \frac{18.83\times10^6}{0.95\times460\times117.8} = 366mm^2$ Provide T_{12} @300 c/c top (As_{prov} = 377mm) Mid span M_x = 0.039 x 15 x 4.913^2 = 14.12KN/m $$K = \frac{14.12 \times 10^6}{25 \times 1000 \times (124)^2} = 0.037 < 0.156$$:. La = 0.95, Z=117.8mm $As_{required} = \frac{14.12\times10^6}{0.95\times460\times117.8} = 274mm^2$ Provide T_{12} @300 c/c top (As_{prov} = 377mm) **Long span** Continuous edge M_y = - 0.037 x 15 x 4.913^2 = -13.40kNm $$K = \frac{13.40 \times 10^6}{25 \times 1000 \times (124)^2} = 0.035 < 0.156$$:. La = 0.95, Z=117.8mm $As_{required} = \frac{13.40\times10^6}{0.95\times460\times117.8} = 260mm^2$	M = -18.83kNm Z=117.8mm Prov T_{12} @300 c/c T (As_{prov} = 377mm) M = 14.12kNm Z=117.8mm Prov T_{12} @300 c/c B (As_{prov} = 377mm) Mx= -13.40kNm Z=117.8mm

	Provide T_{12} @300 c/c top ($As_{prov} = 377mm^2$) <u>Mid span</u> $M_y = 0.028 \times 15 \times 4.913^2 = 10.14$kNm $$K = \frac{10.14 \times 10^6}{25 \times 1000 \times (124)^2} = 0.026 < 0.156$$:. La = 0.95, Z=117.8mm $As_{required} = \frac{10.14 \times 10^6}{0.95 \times 460 \times 117.8} = 197mm^2$ Provide T_{12} @300 c/c B ($As_{prov} = 377mm^2$) **Deflection check** $$\frac{M}{bd^2} = \frac{13.40 \times 10^6}{1000 \times (124)^2} = 0.87$$ $F_s = {}^2/_3 \times 460 \times \frac{240}{377} = 195.23N/mm^2$ $$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$ $$= 0.55 + \frac{477 - 195.23}{120(0.9 + 0.87)} = 1.88$$ $$\frac{limit\ span}{depth} = 1.88 \times 26 = 48.88$$ $$\frac{Actual\ span}{depth} = \frac{4913}{124} = 39.62$$ Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Prov T_{12} @300 c/c T ($As_{prov} = 377mm^2$) My= 10.14kNm Z=117.8mm Prov T_{12} @300 c/c B ($As_{prov} = 377mm^2$) Mf = 1.88 Deflection ok

PANEL SEVENTEEN (17)

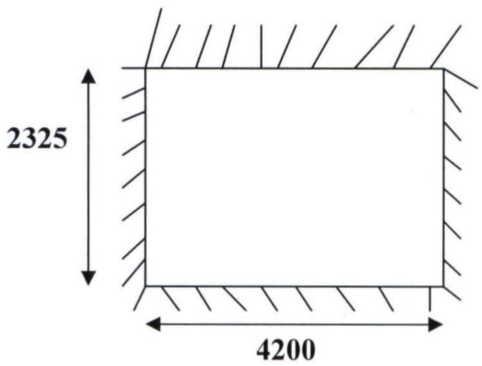


Fig.4.2.17. PANEL 17

$\frac{ly}{lx} = \frac{4200}{2325} = 1.6$

Short span

Negative moment coefficient @ continuous edges = -0.059

Positive moment coefficient @ mid-span = 0.044

Long span

Negative moment coefficient @ continuous edge = -0.032

Positive moment coefficient @ mid-span = 0.024

Short span

<u>Continuous edge</u>

$M_x = - 0.059 \times 15 \times 2.325^2 = -4.78kNm$

M = -4.78kNm

$$K = \frac{4.78 \times 10^6}{25 \times 1000 \times (124)^2} = 0.124 < 0.156$$

Z=117.8mm

:. La = 0.95, Z=117.8mm

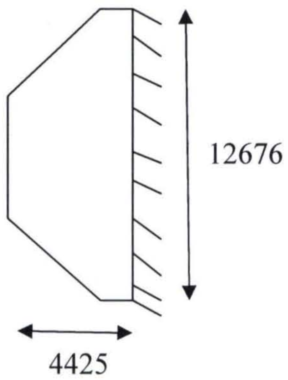
$As_{required} = \frac{4.78 \times 10^6}{0.95 \times 460 \times 117.8} = 93mm^2$

Prov T_{12} @300 c/c T

($As_{prov} = 377mm^2$)

Provide T_{12} @300 c/c top ($As_{prov} = 377mm^2$)

	Mid span	
	$M_x = 0.044 \times 15 \times 2.325^2 = 3.57kNm$	M = 3.57kN/m
	$K = \frac{3.57 \times 10^6}{25 \times 1000 \times (124)^2} = 0.009 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{3.57\times10^6}{0.95\times460\times117.8} = 69mm^2$	
	Provide T_{12}@300 c/c top ($As_{prov} = 377mm^2$)	Prov T_{12} @300 c/c B
	Long span	($As_{prov} = 377mm^2$)
	Continuous edge	
	$M_y = - 0.032 \times 15 \times 2.325^2 = -2.59kNm$	Mx= -2.59kNm
	$K = \frac{2.59 \times 10^6}{25 \times 1000 \times (124)^2} = 0.008 < 0.156$	
	:. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{2.59\times10^6}{0.95\times460\times117.8} = 260mm^2$	
	Provide T_{12}@300 c/c top ($As_{prov} = 377mm^2$)	Prov T_{12} @300 c/c T
	Mid span	($As_{prov} = 377mm^2$)
	$M_y = 0.024 \times 15 \times 2.325^2 = 1.94kNm$	M = 1.94kNm
	$K = \frac{1.94 \times 10^6}{25 \times 1000 \times (124)^2} = 0.005 < 0.15$	
	. La = 0.95, Z=117.8mm	Z=117.8mm
	$As_{required} = \frac{1.94\times10^6}{0.95\times460\times117.8} = 38mm^2$	Prov T_{12} @300 c/c B
	Provide T_{12}@300 c/c B ($As_{prov} = 377mm^2$)	($As_{prov} = 377mm^2$)
	Deflection check	
	$\frac{M}{bd^2} = \frac{3.57 \times 10^6}{1000 \times (124)^2} = 0.23$	

	$F_s = {}^{2}/_{3} \times 460 \times \frac{69}{377} = 56.13N/mm^2$	
	$Mf = 0.55 + \frac{477-Fs}{120\left(0.9+{}^{M}/_{bd^2}\right)} \leq 2.0$	
	$= 0.55 + \frac{477 - 56.13}{120(0.9 + 0.23)} = 3.65$	Mf = 2
	$\frac{limit\ span}{depth} = 2 \times 26 = 52$	
	$\frac{Actual\ span}{depth} = \frac{2325}{124} = 18.75$	
	Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Deflection ok
	Panel Eighteen (18)	
		
	Fig. **Fig.4.2.18. PANEL 18**	
	$\frac{ly}{lx} = \frac{12676}{4425} = 2.86$	
	One way simply supported slab. Moment	
	$M = \frac{WL^2}{8} = \frac{15x4.425x\ 4.425}{8} = 36.71$kNm	
	$K = \frac{36.71 \times 10^6}{25 \times 1000 \times (124)^2} = 0.095 < 0.15$	M = 36.71kNm
	:. La = 0.95, Z=117.8mm	
	$As_{required} = \frac{36.71 \times 10^6}{0.95 \times 460 \times 117.8} = 713mm^2$	Z=117.8mm

	Provide T_{12} @ 150c/c Bottom ($As_{prov} = 754\ mm^2$) **Deflection check** $$\frac{M}{bd^2} = \frac{36.71 \times 10^6}{1000 \times (124)^2} = 2.39$$ $$F_s = {}^2/_3 \times 460 \times \frac{713}{754} = 289.99 N/mm^2$$ $$Mf = 0.55 + \frac{477 - Fs}{120\left(0.9 + {}^M/_{bd^2}\right)} \leq 2.0$$ $$= 0.55 + \frac{477 - 56.13}{120(0.9 + 2.39)} = 1.62$$ $$\frac{limit\ span}{depth} = 1.62 \times 26 = 42.12$$ $$\frac{Actual\ span}{depth} = \frac{4425}{124} = 35.69$$ Since $\frac{limit\ span}{depth} > \frac{Actual\ span}{depth}$	Prov T_{12} @150 c/c B ($As_{prov} = 754mm^2$) Mf = 1.62 Deflection ok

4.3 Design of Beams

1. Decide on the material stresses to be that is f_{cu} and f_y
2. Assume beam size
3. Estimate the characteristics loads Q_k and G_k per unit length of the beam
4. Calculate the design loads
5. Determine the ultimate bending moment *M*
6. Choose the appropriate concrete cover
7. Determine the ultimate resistance moment based on the concrete section (this should be equal to or greater than the ultimate bending moment or a large section be considered)
8. Calculate the reinforcement area
9. Select the reinforcement area to be provided
10. Check the span/effective depth ratio
11. Calculation of the shear reinforcement

BEAM 10

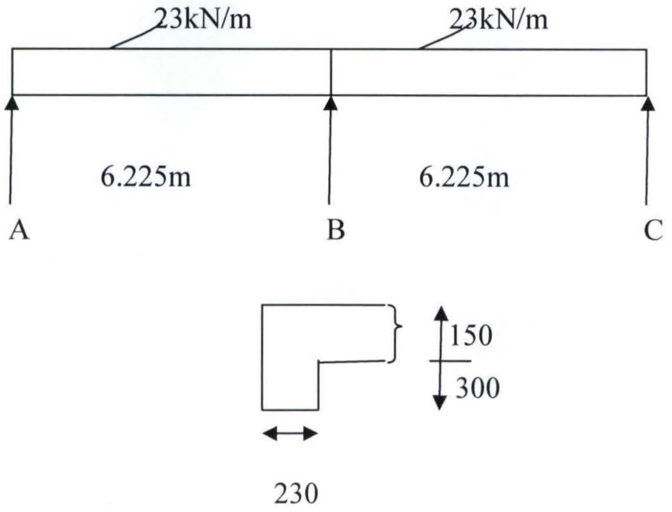


Figure 4.3.1: BEAM 10

Loadings

Span AB & BC

Self weight of beam 0.23 x 0.3 x 24	=1.66KN/m	
Wall load (including rendering) 2.55 x 3.0	=7.65kN/m	
From slab panel 1/2 x 7.1 x 2.025	=7.19kN/m	
Characteristics dead load GK	=16.50kN/m	**GK=16.50Kn**

Impose load

From slab panel 1/2x3x2.025	=3.04kN/m	**QK=3.04 kN**

Ultimate design load

=1.4GK + 1.6QK =1.4(16.50) + 1.6 (3.04)

=27.96kN/m

Say 28kN/m

Design load 28kN/m

USING MOMENT DISTRIBUTION METHOD FOR ANALYSIS

Fixed End moments

$$FEM_{AB} = - FEM_{BA} = \frac{-wl^2}{12} = \frac{28 \times 6.225^2}{12} \quad - 90.42kN/m$$

$FEM_{BC} = - FEM_{CB} = -\frac{wl^2}{12} = -\frac{28 \times 6.225^2}{12}$ -90.42kN/m

Stiffness factors

$K_{AB} = K_{BC} = \frac{4EI}{L} = \frac{4EI}{6.225} = 0.64EI$

Distribution factors

$DF_{BA} = \frac{K_{AB}}{K_{BA} + K_{BC}} = \frac{0.64EI}{0.64EI + 0.64EI} = 0.5$

$DF_{BC} = 1 - 0.5 = 0.5$

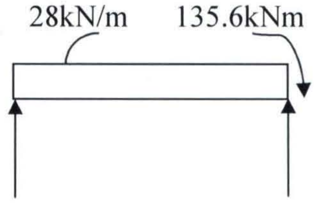
	A	B	B	C
Distribution factors		0.5	0.5	
Fixed End Moment	-90.4	+90.4	-90.4	+90.4
Release A & C	+90.4	0	0	-90.4
Carry over		45.2	-45.2	
Initial Moment	0	+135.6	-135.6	0
Distribution		0	0	
Final Moment		+135.6	-135.6	

Shear Force

Span AB

$\sum M_B = 0$

$6.225R_A - \frac{28 \times 6.225^2}{2} + 135.6 = 0$



$R_A = 65.37kN$

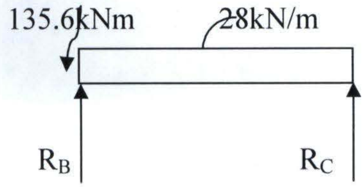
$\sum V = 0$

$R_B = (28 \times 6.225) - 65.37$

$= 108.93kN$

Span BC

$\sum M_C = O$

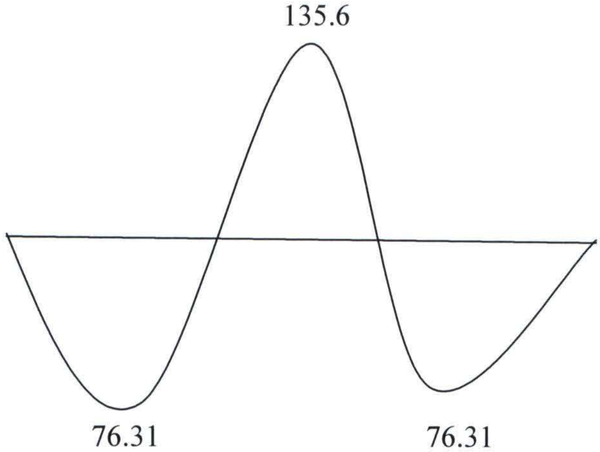


$6.225R_{BR} - \frac{28x\ 6.225^2}{2} - 135.6 = 0, \qquad R_B = -108.93kN$

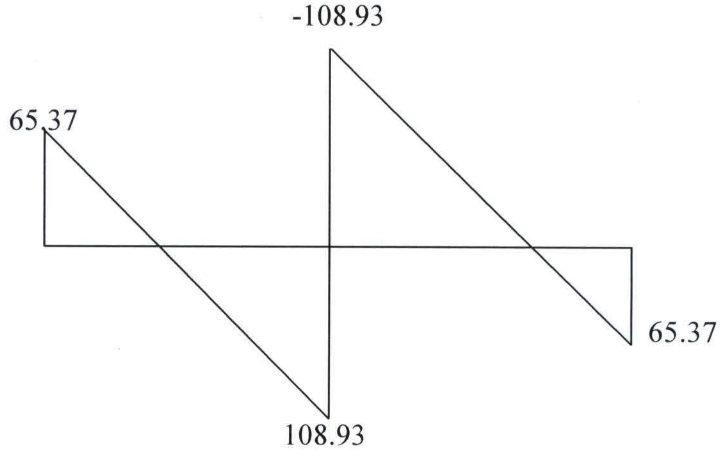
$\sum V = 0$
$R_C = (28x\ 6.225) - 108.93 = 65.37kN$

<u>Span Moments</u>

$M_{AB} = M_{BC} = \frac{V^2_{AB}}{2w} + M_{AB} = \frac{65.37^2}{2X28} + 0 = 76.31\ kNm$



B.M.D



S.F.D

BENDING REINFORCEMENT

Support Moment

M = - 135.6kN/m

Effective dept d= 450-25-10-8 = 407mm

$K = \frac{M}{Fcu\ bd^2} = 135.6x10^6/ (25x230 \times 407^2) = 0.142$

K = 0.142

$La = 0.5 + \sqrt{0.25 - \frac{(0.142)}{0.9}} = 0.80$

L_a = 0.80

Z = La x d = 0.80 x 407 = 327.1mm

Z= 327.1mm

$\text{As required} = \frac{M}{0.95fyZ} = 135.6x10^6/ (0.95x460x\ 327.1)$

= 949mm^2

$As_{min} = \frac{0.13bh}{100} = \frac{0.13\ X230\ X450}{100} = 135.55mm^2$

Provide 3T20mm Bars Top ($A_{s\ provided}$ = 603mm$^{2)}$

3T20mm Top

Span Moment (AB&BC)

M = 76.31KN/m

$b_f = b_w + 0.07L = 230 + 0.07 \times (6225) = 665.75mm$

$K = \frac{M}{F_{cu}b_f d^2}$ $76.31x10^6/(25x665.75x407^2)$ =0.028 < 0.156

: . La = 0.95

Z = Lad = 0.95 X 407 = 386.65mm

$A_{s\ required} = \frac{M}{0.95fyZ} = \frac{76.31X\ 10^6}{0.95\ X\ 460\ X386.65} = 452mm^2$

$A_{s\ Minimum}$ = 0.13%bh = (0.13x230x450)/100 = 134.55mm^2

Provide 4T20mm Bottom (As provided = 603mm^2)

3T16mm Bottom

	SHEAR REINFORCEMENT	
Table 3.8 Bs8110 part 1 1997	Max Shear Force M Beam, Vmax = 108.43kN Shear Stress, $\frac{V}{bd} = \frac{108.93 \times 10^3}{230 \text{ X } 407} = 1.16 N/mm^2$	
Table 3.7 BS8110 part 1 1997	$\frac{100A_{s\,prov}}{bd} = \frac{100 \times 1260}{230 \text{ X } 407} = 1.35$ From table Vc = $0.70 N/mm^2$ $Vc + 0.4 < V > 0.8\sqrt{Fcu}$	
	$\frac{Asv}{Sv} = \frac{b(V - Vc)}{0.95 Fy} = \frac{230\ (1.16 - 0.70)}{0.95 \text{ X } 250} = 0.46$ Provide R8mm @ 200mm c/c As Links $\left(Asv/S_v = 0.503\right)$	**R8@200 c/c**
	DEFELECTION CHECK $\frac{M}{bd^2} = \frac{76.31 \times 10^6}{230 \times 407^2} = 2.00/mm^2$	
	$Fs = {}^2/_3 \, Fy \, \frac{As\ Reg}{As\ Prov} \text{ X } \frac{1}{\beta o} = {}^2/_3 \times \frac{460 \times 452}{603} = 229.87 N/mm^2$	
	$M.F = 0.55 + \frac{(477 - 229.87)}{120(0.9 + 2)} = 1.26$	
	$\frac{\text{Limiting Span}}{\text{Depth}}$ = M.F x Limiting Factor = 1.26x 26 = 32.76 $\frac{\text{Actual Span}}{\text{Depth}} = \frac{6225}{407} = 15.29$ Since $\frac{\text{Liming Span}}{\text{Depth}} > \frac{\text{Actual Span}}{\text{Depth}}$ Deflection is Ok.	**Deflection is Satisfied**

BEAM 17

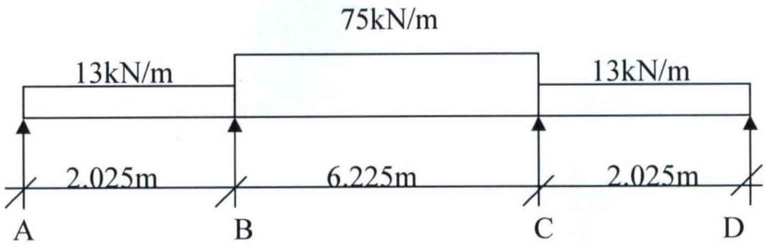


Figure 4.3.2: BEAM 17

Loadings

Span AB And CD

Self Weight Of Beam : 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load; =2.55 X 3.0 = 7.65kN/m

Dead load 9.31kN/m

Ultimate Design Local

1.4 (9.31) = 13.03kN/m

Say 13kN/m

GK=9.31 kN/m

Design load 13kN/m

Span BC

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.93kN/m

Wall Load; =2.55 X 3.0 = 7.65kN/m

From Slab P9; =($^{1}/_{3}$x7.1x6.225)x2 = 29.47kN/m

Dead Load 39.05kN/m

GK=39.05 kN/m

Imposed Load

From Slab P1; =($^{1}/_{3}$x3x6.225)x2 = 12.45kN/m

QK=12.45 kN/m

Ultimate Design Load

1.4(39.05) + 1.6 (12.45) = 74.53kN/m

Say 75kN/m

Design load 75KN/m

USING MOMENT DISTRIBUTION METHOD TO ANALYZE

Fixed End moments

$FEM_{AB} = -FEM_{BA} = FEM_{CD} = -FEM_{DC}$

$$\frac{-wl^2}{12} = \frac{-13 \times 2.025^2}{12} = -4.44kNm$$

$$FEM_{BC} = -FEM_{CB} = -\frac{wl^2}{12} = -\frac{75 \times 6.225^2}{12} = -242kNm$$

Member Stiffness

$$K_{AB} = \frac{4EI}{L} = \frac{4EI}{2.025} = 1.98EI = K_{CD}$$

$$K_{BC} = \frac{4EI}{L} = \frac{4EI}{6.225} = 0.64EI$$

Distribution Factors

$$DF_{BA} = \frac{K_{AB}}{K_{BA} + K_{BC}} = \frac{1.98EI}{1.98EI + 0.64EI} = 0.76$$

$DF_{BA} = 1 - 0.76 = 0.24$

$$DF_{CB} = \frac{K_{CB}}{K_{CB} + K_{CD}} = \frac{0.64EI}{0.64EI + 1.98EI} = 0.24$$

$DF_{BC} = 1 - 0.24 = 0.76$

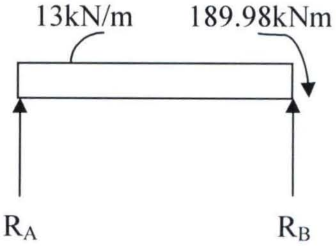
	A	B		C		D
Distr. factors		0.64	0.36	0.36	0.64	
F. E. M	-4.44	4.44	-242	+242	-4.44	+4.44
Release A&D	+4.44					-4.44
Carry over		2.22			-2.22	
Initial Momt.	0	6.66	-242	242	-6.66	0
Distribution		150.6	84.7	-84.7	-150.2	
Carry over			-42.4	42.4		
Distribution		27.14	15.3	-15.3	-27.14	
Carry over			-7.65	7.65		
Distribution		4.9	2.75	-2.75	-4.9	
Carry over			-1.38	1.38		
Distribution		0.88	0.50	-0.50	-0.88	
Carry over			-0.25	0.25		
Distribution		0.16	0.09	-0.09	-0.16	
Carry over			-0.05	0.05		
Distribution		0.03	0.02	-0.02	-0.03	
			-0.01	0.01		
		0.006	0.004	-0.004	-0.006	
Final Moment		189.98	-190.38	190.38	-189.98	

Shear Force

Span AB

$\sum M_B = 0$

$$2.025R_A - \frac{13 \times 2.025^2}{2} + 189.98 = 0$$



$R_A = \;\; -80.65kN$

$\sum V = 0$

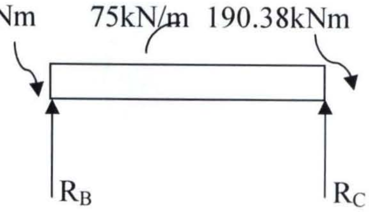
$R_B = -(13 \times 2.025) + (-80.65)$

$= -106.98kN$

Span BC

$\sum M_C = 0$

$$6.225R_B - \frac{75 \times 6.225^2}{2} - 190.38 = 0$$



$R_B = 264.02kN$

$\sum V = 0$

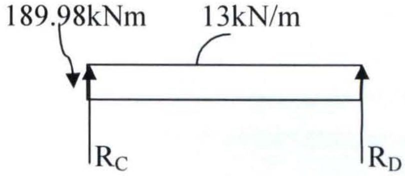
$R_C = -(75 \times 6.225) + (264.95)$

$= -201.93kN$

Span CD

$\sum M_D = 0$

$$2.025R_C - \frac{13 \times 2.025^2}{2} - 189.98 = 0$$



$R_C = 216.63kN$

$\sum V = 0$

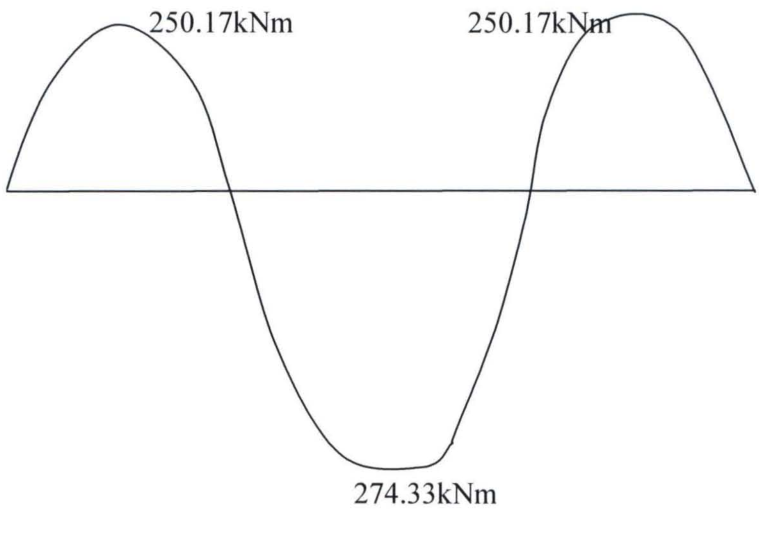
$R_D = -(13 \times 2.025) + (216.63)$

$= -190.31kN$

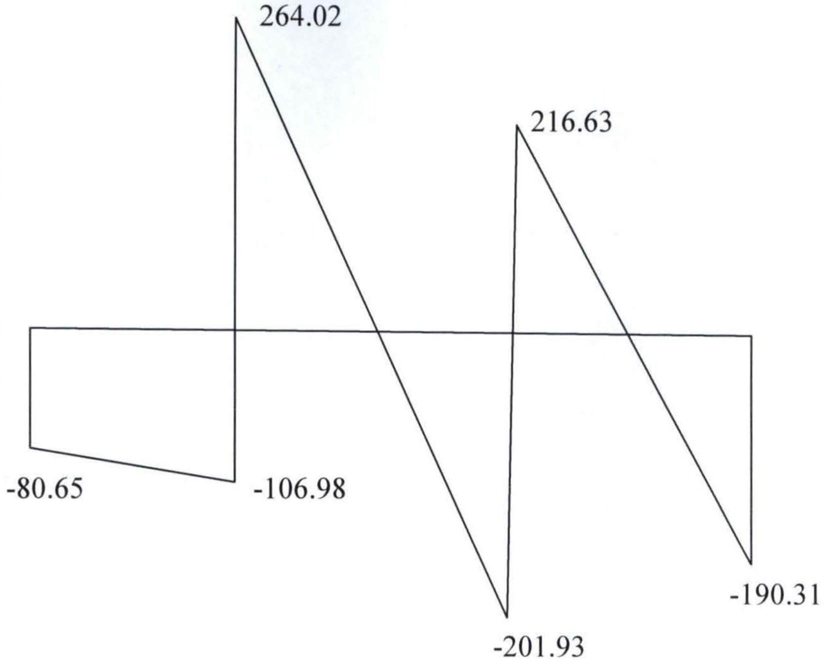
<u>Span Moments</u>

$$M_{AB} = M_{CD} = \frac{V^2_{AB}}{2w} + M_{AB} = \frac{-80.65^2}{2X13} + 0 = 250.17 \text{ kNm}$$

$$M_{BC} = \frac{V^2_{BC}}{2w} + M_{BC} = \frac{264.02^2}{2X75} - 190.38 = 274.33 \text{ kNm}$$



B.M.D



S.F.D

BENDING REINFORCEMENT

Support Moment

Maximum support moment M = 190.38kN/m

M = 190.38kKN/m

$$K = \frac{190.38 \text{ X } 10^6}{25 \text{ X}230 \text{ X } 407^2} = 0.199 > 0.156$$

K=0.199

Therefore, compression reinforcement is required

La=0.67

$$La = 0.5+\sqrt{0.25 - \left(\frac{0.199}{0.9}\right)} = 0.673$$

Z=273.99 mm

Z = La x d = 0.673 X 407 = 273.99mm

$$A^l = \frac{M - 0.156x\ f_{cu}\ x\ bd^2}{0.95 \text{ X } f_y\ (d\text{-}d^l)}$$

$$As^l = \frac{190.38 \text{ x } 10^6 - 0.156x\ 25 \text{ x } 230 \text{ x } 407^2}{0.95 \text{ X } 460 \text{ x } (407\text{-}43)} = 263mm^2$$

$$A_{s\ required} = \frac{0.156f_{cu}bd^2}{0.95\ f_y z} + As^l$$

$A_{s\ required}$ $= \dfrac{0.156 \times 25 \times 230 \times 407^2}{0.95 \times 460 \times 273.99} + 263 = 1504mm^2$

$As_{minimum} = 0.31\%bh = 179.4mm^2$

Provided 5T20mm Bars Top (As Provide = $1570mm^2$)

5T20mm Top

Span Moment

Maximum span moment M = 274.33KN/m

M = 274.33KN/m

b_f = 230 + 0.14 X 6225 = 1101.5mm

$K = \dfrac{274.33 \times 10^6}{25 \times 1101.5 \times 407^2} = 0.060 < 0.156$

K=0.060

La=0.95

La = 0.95

Z = La x d = 0.95 X 407 = 386.65mm

Z=386.65 mm

$As_{required} = \dfrac{274.33 \times 10^6}{0.95 \times 460 \times 386.65} = 1624mm^2$

$As_{minimum}$ = 0.13x1101.5 x 450/100 = $644.38mm^2$

Provided 4T25mm Bars Bottom (As provided = $11960mm^2$)

4T25mm Btm

BEAM 11

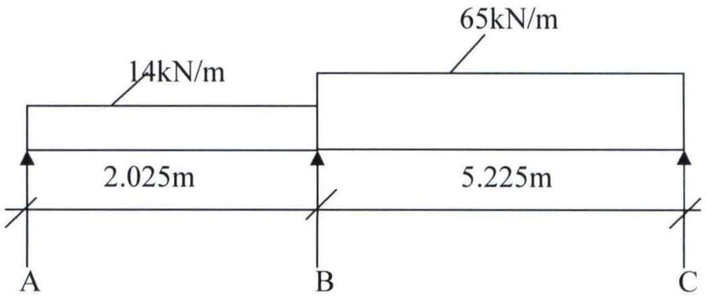


Figure 4.3.3: BEAM 11

Loadings

Span AB

Self Weight Of Beam : 0.23 X 0.30 X 24 = 1.66kN/m

Wall Load ; =2.55 X 3.0 = 7.65kN/m

Dead load 9.31kN/m

Ultimate Design Local

1.4 (9.31) = 13.03kN/m

Say 14kN/m

<u>**Span BC**</u>

Self Weight Of Beam; 0.23 X 0.30 X 24 = 1.66kN/m

Wall Load ; =2.55 X 3.0 = 7.65kN/m

From Slab P1; =($^1/_2$x7.1x5.225) x2 = 24.73kN/m

Dead Load 34.04kN/m

Imposed Load

From Slab P1; =($^1/_2$x3x5.225) x2 = 10.45kN/m

Ultimate Design Load

1.4(34.04) + 1.6 (10.45) = 64.37kN/m

Say 65kN/m

USING MOMENT DISTRIBUTION METHOD TO ANALYZE

Fixed End moments

$$FEM_{AB} = -FEM_{BA} = \frac{-wl^2}{12} = \frac{-14 \times 2.025^2}{12} \quad -4.78\text{kNm}$$

$$FEM_{BC} = -FEM_{CB} = -\frac{wl^2}{12} = -\frac{65 \times 5.225^2}{12} \quad -147.9\text{kNm}$$

Member Stiffness

$$K_{AB} = \frac{4EI}{L} = \frac{4EI}{2.025} = 1.96EI$$

$$K_{BC} = \frac{4EI}{L} = \frac{4EI}{5.225} = 0.77EI$$

Distribution Factors

$$DF_{BA} = \frac{K_{AB}}{K_{BA} + K_{BC}} = \frac{1.96EI}{1.96EI + 0.77EI} = 0.72$$

$DF_{BA} = 1 - 0.72 = 0.28$

G_K=9.31 kN/m

Design load 14kN/m

G_K=34.04 kN/m

Q_K=10.45 kN/m

Design load 65kN/m

	A	B		C
Distr. factors		0.72	0.28	
F. E. M	-4.78	4.78	-147.9	147.9
Release A&C	+4.78	0	0	-147.9
Carry over		+2.39	-74	
Initial moment		7.17	-221.9	
Distribution		154.6	60.12	
Final moment		162	-162	

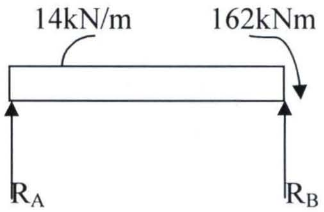
Shear Force

Span AB

$\sum M_B = 0$

$$2.025R_A - \frac{14 \times 2.025^2}{2} + 162 = 0$$

$R_A = -65.83kN$



$\sum V = 0$

$R_B = (14 \times 2.025) - (65.83)$

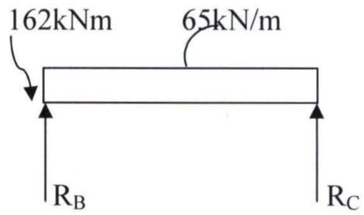
$= -37.48kN$

Span BC

$\sum M_C = 0$

$$5.225R_B - \frac{65 \times 5.225^2}{2} - 162 = 0$$

$R_B = 200.82kN$



$\sum V = 0$

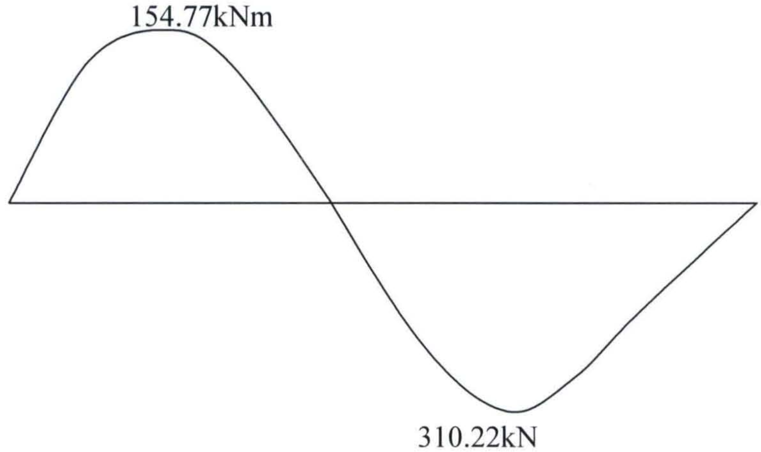
$R_C = (65 \times 5.225) - (200.82)$

$= 138.81kN$

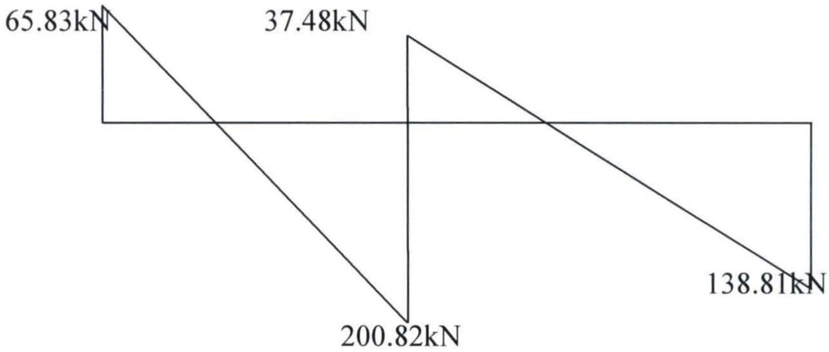
Span Moments

$$M_{AB} = \frac{V^2_{AB}}{2w} + M_{AB} \quad = \frac{65.83^2}{2X14} + 0 = 154.77 \text{ kNm}$$

$$M_{BC} = \frac{V^2_{BC}}{2w} + M_{BC} \quad = \frac{200.82^2}{2X65} + 0 = 310.22 \text{ kNm}$$



B.M.D



S.F.D

BENDING REINFORCEMENT

Support Moment

Maximum support moment M = 162kNm

M = 162kNm

$$K = \frac{162 \times 10^6}{25 X230 X 407^2} = 0.170 > 0.156$$

K=0.170

Compression Reinforcement is required

La=0.75

$$La = 0.5 + \sqrt{0.25 - \left(\frac{0.170}{0.9}\right)} = 0.75$$

	$Z = La \times d = 0.75 \times 407 = 304.11mm$	**Z = 304.11 mm**
	$A^1 = \dfrac{M - 0.156x\ f_{cu} \times bd^2}{0.95 \text{ X } f_y (d-d^1)}$	
	$As^1 = \dfrac{162 \times 10^6 - 0.156x\ 25 \times 230 \times 407^2}{0.95 \text{ X } 460 \times (407-43)} = 84.32mm^2$	
	$A_{s\ required} = \dfrac{0.156f_{cu}bd^2}{0.95\ f_y z} + As^1$	
	$A_{s\ required} = \dfrac{0.156x25x230x407^2}{0.95x460x304.11} + 84.32 = 1202.39mm^2$	
	$A_{S\ minimum} = 0.31^0/_0 bh = 134.55mm^2$	**4T20mm**
	Provided 4T20mm Bars Top (As Provide = $1260mm^2$)	**Top**
	Span Moment	
	Span AB	
	$M = 154.77kN/m$	
	$B_F = 230 + 0.14 \times 2025 = 513.5mm$	
	$K = \dfrac{154.77x\ 10^6}{25 \times 230 \times 407^2} = 0.162 > 0.156$	
	Compression Reinforcement is required	
	$La = 0.5 + \sqrt{0.25 - \left(\dfrac{0.162}{0.9}\right)} = 0.76$	
	$Z = La \times d = 0.76 \times 407 = 309.32mm$	
	$A^1 = \dfrac{M - 0.156x\ f_{cu} \times bd^2}{0.95 \text{ X } f_y (d-d^1)}$	
	$As^1 = \dfrac{154.77 \times 10^6 - 0.156x\ 25 \times 230 \times 407^2}{0.95 \text{ X } 460 \times (407-43)} = 38.87mm^2$	

$$A_{s\ required} = \frac{0.156 f_{cu} b d^2}{0.95 f_y z} + As^I$$

$$A_{s\ required} = \frac{0.156 x 25 x 230 x 407^2}{0.95 x 460 x 309.32} + 38.87 = 1138.11mm^2$$

$A_{S\ minimum} = 0.13x527.3 \ x \ 450/100 = 308.47mm^2$

Provided 4T20mm Bars Bottom (As provided = $1260mm^2$)

4T20mm Btm

Span BC

M = 310.22kN/m

$B_F = 230 + 0.14 \ x \ 5225 = 961.5mm$

$$K = \frac{310.22 \ x \ 10^6}{25 \ x \ 961.5 \ x \ 407^2} = 0.078 < 0.156$$

$$La = 0.5 + \sqrt{0.25 - \left(\frac{0.078}{0.9}\right)} = 0.90$$

Z = La x d = 0.90 X 407 = 368.04mm

$$As_{Required} = \frac{310.22 \ x \ 10^6}{0.95 \ X \ 460 \ X \ 368.04} = 1929mm^2$$

$A_{S\ minimum} = 0.13x961.5x \ 450/100 = 562.48mm^2$

Provided 4T25mm Bars Bottom (As provided = $1960mm^2$)

4T25mm Btm

BEAM 9

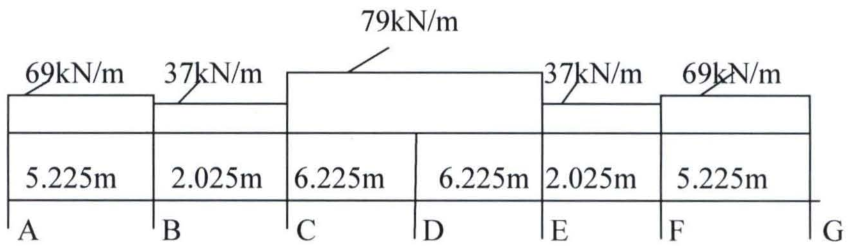


Figure 4.3.4: BEAM 9

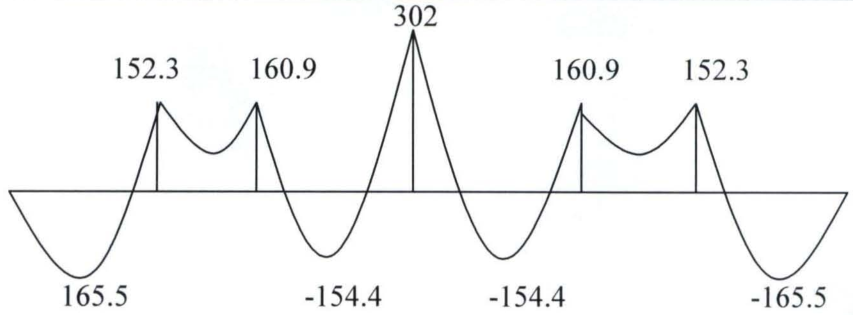
Loadings

Span AB & FG

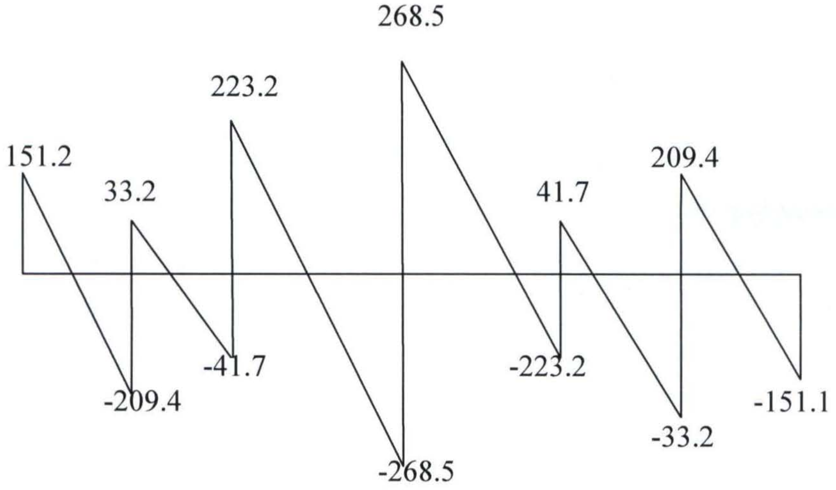
Self Weight Of Beam : 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load (Include Rendering) ; 3.47 X 3.0 = 10.41kN/m

	From Slap Panels ; ($^1/_3$ x 7.1x 5.225)x2 = 24.73kN/m	
	Dead load 36.76kN/m	**G_K=36.76 kN/m**
	Impose Load	
	From Slab P1 ; ($^1/_3$ x3 x 5.225)2 = 10.45kNm	**Q_K=10.45 kN/m**
	Ultimate Design Local	**Design load**
	1.4 (36.76) + 1.6(10.45) = 68.184kN/m	
	Say 69KN/m	**69kN/m**
	Span CD & DE	
	Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m	
	Wall Load ; =3.47 X 3.0 = 10.41kN/m	
	From Slab P1; =($^1/_3$x7.1x 6.225)2 = 29.47kN/m	
	Dead Load = 41.5kN/m	**G_K=41.5 kN/m**
	Imposed Load	
	From Slab P1; =($^1/_3$x3x6.225)2 = 12.45kN/m	**Q_K=12.45 kN/m**
	Ultimate Design Load	**Design load**
	1.4(41.5) + 1.6 (12.45) = 78.02kN/m	
	Say 79kN/m	**79kN/m**
	Span BC & EF	
	Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m	
	Wall Load ; =3.47 X 3.0 = 10.41kN/m	
	From Slab P1; =($^1/_3$x7.1x 2.025)2 = 9.59kN/m	
	Dead Load = 21.62kN/m	**GK=21.62 kN/m**
	Imposed Load	**QK=4.05 kN/m**
	From Slab Panel = ($^1/_3$x3x2.025) 2 = 4.05kN/m	
	Ultimate Design Load	**Design load**
	1.4(21.62) + 1.6 (4.05) = 36.748kN/m	
	Say 37kN/m	**37kN/m**



B.M.D



S.F.D

BENDING REINFORCEMENT

Support Moment

Maximum support moment M = 302kN/m

$$K = \frac{302 \times 10^6}{25 \times 230 \times 555^2} = 0.17$$

K=0.17

$$La = 0.5 + \sqrt{0.25 - \left(\frac{0.17}{0.9}\right)} = 0.75$$

La=0.95

$Z = La \times d = 0.95 \times 555 = 527.3$mm

$$A_{s\ required} = \frac{302 \times 10^6}{0.95 \times 460 \times 527.3} = 1311\text{mm}^2$$

Z=527.3

$A_{S\ minimum} = 0.31\,{}^{0}/_{0}\,bh = 179.4\text{mm}^2$

Provided 5T20mm Bars Top (As Provide = 1570mm^2)

5T20mm Top

Span Moment

Maximum span moment M = 154.4kN/m

M = 154.4KN/m

B_F = 230 + 0.14 x 6225 = 1101.5mm

$$K = \frac{154.4 \times 10^6}{25 \times 1101.5 \times 555^2} = 0.018 < 0.156$$

K=0.018

La = 0.95

La=0.95

Z = La x d = 0.95 X 555 = 527.3mm

Z=527.3

$$As_{Required} = \frac{154.4 \times 10^6}{0.95 \times 460 \times 527.3} = 670mm^2$$

$As_{minimum}$ = 0.13x1101.5 x 600/100 = 859.2mm^2

Provided 3T20mm Bars Bottom (As provided = 943mm^2)

3T20mm Btm

BEAM 18

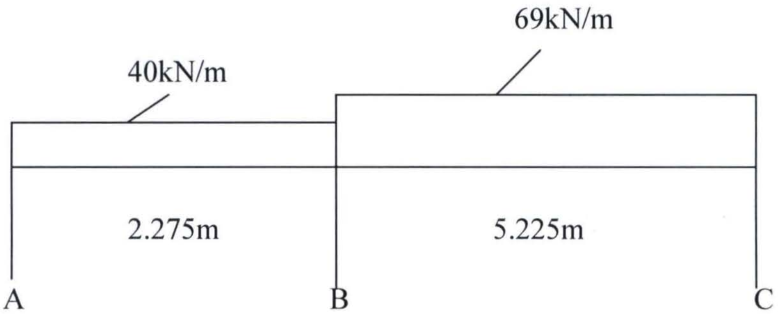


Figure 4.3.5: BEAM 18

Loadings

Span AB

Self Weight Of Beam : 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load (Include Rendering) ; 3.47 X 3.0 = 10.41kN/m

From Slap Panels ; ($^1/_3$ x 7.1x 2.75)x2 = 10.77kN/m

Dead load = 22.8kN/m

Impose Load

From Slab Panels ; ($^1/_3$ x3 x 2.275)2 = 4.55kNm

Ultimate Design Local

1.4 (22.8) + 1.6(4.55) = 39.21kN/m

Say 40kN/m

Span BC

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load ; =3.47 X 3.0 = 10.41kN/m

From Slab Panel; $=(^{1}/_{3}x7.1x\ 5.225)2$ = 24.7kKN/m

Dead Load = 36.76kN/m

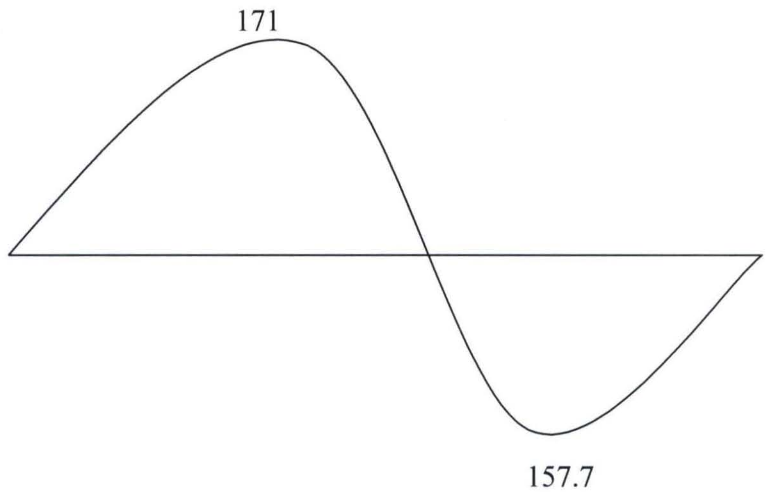
Imposed Load

From Slab Panel; $=(^{1}/_{3}x3x5.225)2$ = 10.45kN/m

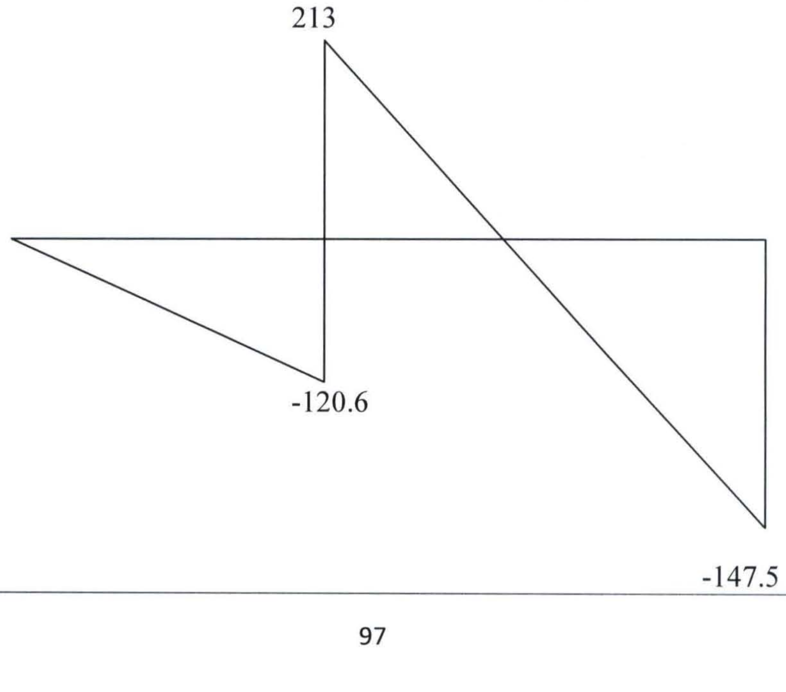
Ultimate Design Load

1.4(36.76) + 1.6 (10.45) = 68.18kN/m

Say 69kN/m



B.M.D



S.F.D

BENDING REINFORCEMENT

Support Moment

Maximum support moment M = 171kN/m

M = 171kN/m

$$K = \frac{171 \times 10^6}{25 \text{ X} 230 \text{ X } 555^2} = 0.097 < 0.156$$

K=0.097

$$La = 0.5 + \sqrt{0.25 - \left(\frac{0.097}{0.9}\right)} = 0.88$$

La=0.88

Z=527.3m

Z = La x d = 0.95 x 555 = 527.3mm

$$A_{s\ required} = \frac{171 \times 10^6}{0.95 \text{ X } 460 \times 527.3} = 742mm^2$$

$A_{S\ minimum} = 0.31^{0}/_{0}bh = 179.4mm^2$

Provided 4T16mm Bars Top (As Provide = $804mm^2$)

4T16mm Top

Span Moment

Maximum span moment M = 157.7kN/m

M = 157.7KN/m

B_F = 230 + 0.14 x 5225 = 961.5mm

$$K = \frac{157.7 \times 10^6}{25 \times 961.5 \times 555^2} = 0.021 < 0.156$$

K=0.021

La = 0.95

La=0.95

Z=488.4mm

Z = La x d = 0.95 X 555 = 488.4mm

$$A_{S\ Required} = \frac{157.7 \times 10^6}{0.95 \text{ X } 460 \text{ X } 488.4} = 734mm^2$$

$A_{S\ minimum}$ = 0.13x 961.5 x 600/100 = $750mm^2$

Provided 4T16mm Bars Bottom (As provided = $804mm^2$)

4T16mm Btm

BEAM 7

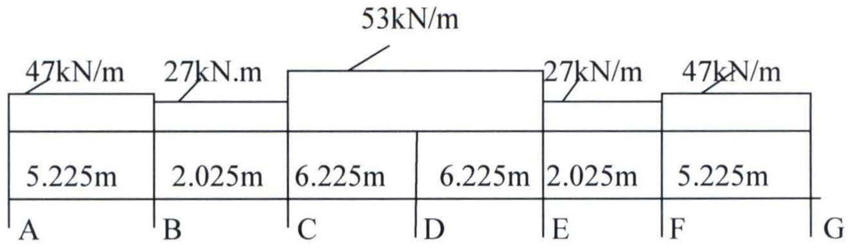


Figure 4.3.6: BEAM 7

Loadings

Span AB & FG

Self Weight Of Beam : 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load (Include Rendering) ; 3.47 X 3.0 = 10.41kN/m

From Slap Panels ; $^{1}/_{2}$x7.1x5.225(1-1/3x1.2^{2}) = 14.23kN/m

Dead load 26.26kN/m

G_K=26.26 kN/m

Impose Load

From Slab Panel ;= $^{1}/_{2}$x3x5.225(1-1/3x1.2^{2}) = 6.02KNm

Q_K=6.02 kN/m

Ultimate Design Local

1.4 (26.26) + 1.6(6.02) = 46.396kN/m

Say 49kN/m

Design load 49kN/m

Span CD & DE

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load ; =3.47 X 3.0 = 10.41kN/m

From Slab Panel; = $^{1}/_{2}$x3x6.225(1-1/3x1.2^{2}) = 16.98kN/m

Dead Load = 29.01kN/m

G_K=29.01 kN/m

Imposed Load

From Slab Panel; = $^{1}/_{2}$x3x6.225(1-1/3x1.2^{2}) = 7.18kN/m

Q_K=7.18 kN/m

Ultimate Design Load

1.4(29.01) + 1.6 (7.18) = 52.102kN/m

Say 53kN/m

Design load 53kN/m

Span BC & EF

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load ; =3.47 X 3.0 = 10.41kN/m

From Slab Panel; =($^1/_3$x7.1x 2.025) = 4.79kN/m

G_K=16.82 kN/m

Dead Load = 16.82kN/m

Q_K=2.03 kN/m

Imposed Load

From Slab Panel = ($^1/_3$x3x2.025) = 2.03kN/m

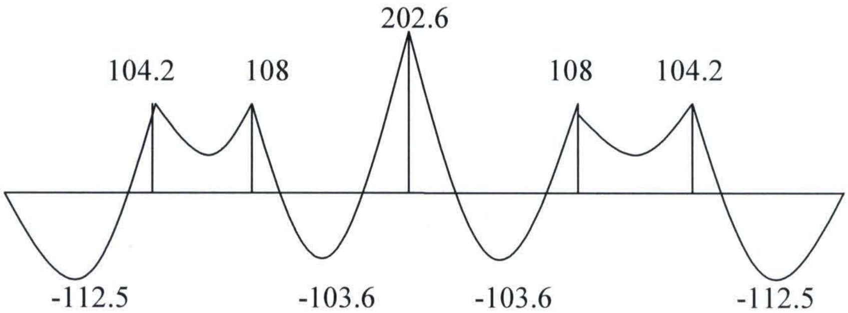
Design load 27kN/m

Ultimate Design Load

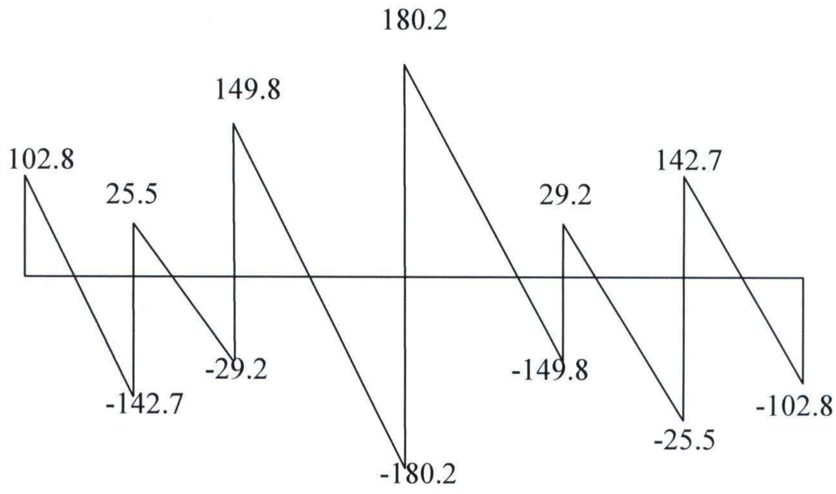
1.4(16.82) + 1.6 (2.03) = 26.796kN/m

Say 27kN/m

Xdc /u.u.iiu



B.M.D



S.F.D

	BENDING REINFORCEMENT	
	Support Moment	
	Maximum support moment M = 202kN/m	
	M = 202kN/m	
	$K = \frac{202 \times 10^6}{25 \times 230 \times 555^2} = 0.114$	**K=0.114**
	$La = 0.5 + \sqrt{0.25 - \left(\frac{0.114}{0.9}\right)} = 0.85$	**La=0.95**
	Z = La x d = 0.95 x 555 = 527.3 mm	**Z=527.3**
	$A_{s\ provided} = \frac{202 \times 10^6}{0.95 \times 460 \times 488.4} = 947mm^2$	
	$A_{S\ minimum} = 0.31\%bh = 131.63mm^2$	**4T20mm**
	Provided 4T20mm Bars Top (As Provide = $1260mm^2$)	**Top**
	Span Moment	
	Maximum span moment M = 112.5kN/m	
	M = 112.5kN/m	
	$B_F = 230 + 0.07 \times 5225 = 595.75mm$	
	$K = \frac{112.5 \times 10^6}{25 \times 595.75 \times 555^2} = 0.025 < 0.156$	**K=0.025**
	La = 0.95	**La=0.95**
	Z = La x d = 0.95 X 555 = 527.3mm	**Z=527.3**
	$A_{S\ Required} = \frac{112.5 \times 10^6}{0.95 \times 460 \times 488.4} = 669mm^2$	
	$A_{S\ minimum} = 0.13 \times 595.75 \times 600/100 = 465mm^2$	**3T16mm**
	Provided 3T16mm Bars Bottom (As provided = $603mm^2$)	**Btm**

BEAM 14

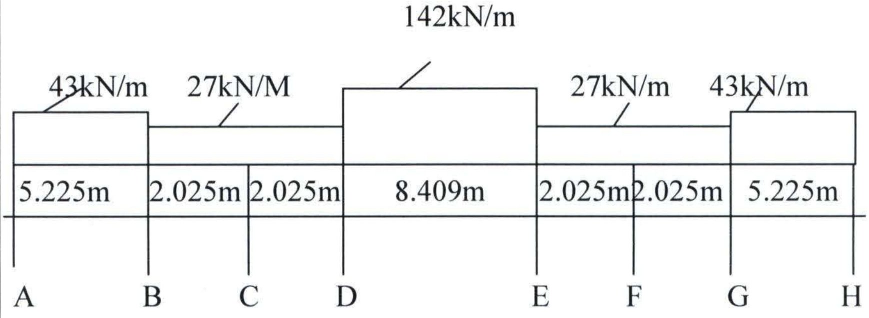


Figure 4.3.7: BEAM 14

Loadings

Span AB & GH

Self Weight Of Beam : 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load (Include Rendering) ; 3.47 X 3.0 = 10.41kN/m

From Slab Panels ; $^{1}/_{3}$x7.1x5.225 = 12.3kN/m

Dead load 24.40kN/m

G_K=24.40 kN/m

Impose Load

From Slab Panel ; = $^{1}/_{3}$x3x5.225 = 5.23kNm

Q_K=5.23kN/m

Ultimate Design Local

1.4 (24.40) + 1.6(5.23) = 42.528kN/m

Say 43kN/m

Design load 43kN/m

Span BC, CD , EF & FG

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load ; =3.47 X 3.0 = 10.41kN/m

From Slab Panel; = $^{1}/_{3}$x3x2.025 = 4.79kN/m

Dead Load = 16.82kN/m

G_K=16.82 kN/m

Imposed Load

From Slab Panel; = $^{1}/_{2}$x3x2.025 = 2.03kN/m

Q_K=2.03kN/m

Ultimate Design Load

1.4(16.82) + 1.6 (2.03) = 26.796kN/m

Say 27kN/m

Design load 27kN/m

Span DE

Self Weight Of Beam; 0.23 X 0.3 X 24 = 1.66kN/m

Wall Load ; =3.47 X 3.0 = 10.41kN/m

From Slab Panel; = $^1/_2$x7.1x 8.175(1-1/3x2.1^2) = 53.66kN/m

Dead Load = 70.52kN/m

G_K=70.52 kN/m

Imposed Load

From Slab Panel = $^1/_2$x3x 8.175(1-1/3x2.1^2) = 26.68kN/m

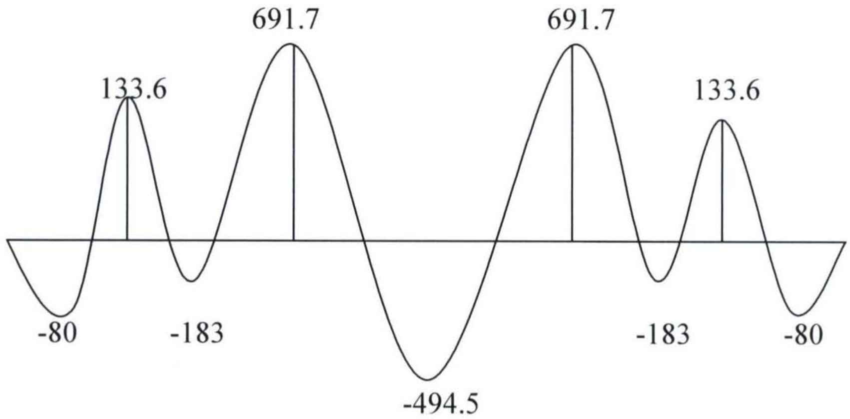
Q_K=26.68 kN/m

Ultimate Design Load

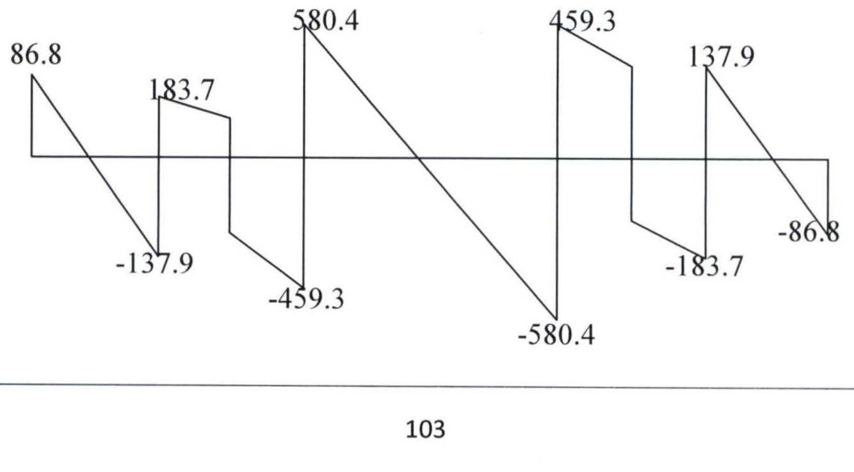
1.4(70.52) + 1.6 (26.68) = 141.416kN/m

Say 142kN/m

Design load 142kN/m



B.M.D



S.F.D

	BENDING REINFORCEMENT	
	Support Moment	
	Maximum support moment $M = 691.7kNm$	
	$M = 691.7kNm$	
	$K = \frac{691.7 \times 10^6}{25 \times 230 \times 655^2} = 0.28$	**K=0.28**
	$La = 0.5 + \sqrt{0.25 - \left(\frac{0.28}{0.9}\right)} = 0.93$	**La=0.93**
	$Z = La \times d = 0.95 \times 655 = 622.3$ mm	
	$A_{s\ provided} = \frac{691.7 \times 10^6}{0.95 \times 460 \times 622.3} = 2544mm^2$	**Z=622.3 mm**
	$A_{S\ minimum} = 0.31^{0}/_{0}bh = 209.3mm^2$	
	Provided 5T25mm Bars Top (As Provide = $2950mm^2$)	**6T25mm Top**
	Span Moment	
	Maximum span moment $M = 494.5kN/m$	
	$M = 494.5kN/m$	
	$B_F = 230 + 0.07 \times 8175 = 802.3mm$	
	$K = \frac{494.5 \times 10^6}{25 \times 802.3 \times 555^2} = 0.08 < 0.156$	**K=0.08**
	$La = 0.95$	**La =0.95**
	$Z = La \times d = 0.95 \times 555 = 488.4mm$	**Z=488.4 mm**
	$A_{S\ Required} = \frac{494.5 \times 10^6}{0.95 \times 460 \times 488.4} = 2317mm^2$	
	$A_{S\ minimum} = 0.13 \times 802.3 \times 600/100 = 626mm^2$	
	Provided 5T25mm Bars Bottom (As provided = $2450mm^2$)	**5T25mm Btm**

	4.4 ANALYSIS AND DESIGN OF COLUMN	
	For load estimation, static reactions only are considered	
	COLUMN C_2 A on grid A1	
	Loadings	
	From 2nd floor to roof:-	
	Column own weigh t= 0.225^2 x 3 x 24 = 3.65kN/m	
	From roof beam = 0.5 (18 x 3.375) = 30.38 kN/m	
	From roof beam = 0.5 (18 x 5.225) = 47.03kN/m	
	81.07kN/m	Say = 82kN
	From 1st floor to 2nd floor: -	
	Column own weight = 3.65kN/m	
	Load form above = 81.07kN/m	
	Load from beam 1:- 0.5 (34 x 3.375) = 57.38kN/m	
	Load from beam 14:- 0.5 (43 x 5.225) = 112.34kN/m	
	254.44kN/m	Say = 255kN
	From Ground floor to 1st floor :-	
	Column own weight = 3.65kN/m	
	Load from above = 254.44kN/m	
	Load from beam 1:- 0.5 (34 x 3.375) = 57.38kN/m	
	Load from beam 14:- (43 x 5.225) = 112.34 kN/m	
	427.81kN/m	Say = 428kN
	COLUMN C_2 I on grid I1	
	From 2nd floor roof :-	
	Column own weight = 3.65kN/m	
	From roof beam = 0.5 (18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 2.625) = 23.63kN/m	
	From roof beam = 0.5(18 x 5.225) = 47.03kN/m	
	130.34kN/m	Say = 131kN

	From first to 2nd floor:-	
	Column own weight = 3.65kN/m	
	Load from above = 130.34kN/m	
	From beam 1:- = 0.5 (53 x 6.225) = 164.96kN/m	
	From beam 1:- = 0.5 (30 x 2.625) = 39.38kN/m	
	From beam 7:- = 0.5 (47 x 5.225) = 122.79kN/m	
	461.12kN/m	**Say = 462kN**
	From Ground to 1st floor :-	
	Column own weight = 3.65kN/m	
	From above = 461.12kN/m	
	From Beam 1:- 0.5 (53 x 6.225) = 164.96kN/m	
	From Beam 1:- 0.5 (30 x 2.625) = 39.38kN/m	
	From Beam 7:- 0.5 (47 x 5.225) = 122.79kN/m	
	= 791.9kN/m	**Say = 792kN**
	COLUMN C_3 J on grid J3	
	From 2nd floor roof :-	
	Column own weight = 3.65kN/m	
	From roof beam = 0.5 (18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 5.225) = 47.03kN/m	
	From roof beam = 0.5(18 x 2.025) = 18.23kN/m	
	180.97kN/m	**Say = 181kN**
	From 1st to 2nd floor:-	
	Column own weight = 3.65kN/m	
	Load from above = 180.97kN/m	
	From beam 2:- = 0.5 (88 x 6.225) = 273.90kN/m	
	From beam 2:- = 0.5 (88 x 6.225) = 273.90kN/m	
	From beam 11:- = 0.5 (65 x 5.225) = 169.81kN/m	
	From beam 11:-= 0.5 (34 x 2.025) = 34.43kN/m	
	936.66kN/m	**Say = 937kN**

	From Ground to 1st floor :-	
	Column own weight = 3.65kN/m	
	From above = 936.66kN/m	
	From Beam 2:- = 273.90kN/m	
	From Beam 2:- = 273.90kN/m	
	From Beam 11:- = 169.81kN/m	
	From Beam 11:- = 34.43kN/m	
	1692.35kN/m	**Say = 1693KN**
	COLUMN C_4 J on grid J4	
	From 2nd floor roof :-	
	Column own weight = 3.65kN/m	
	From roof beam = 0.5 (18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 2.025) = 18.23kN/m	
	133.94kN/m	**Say = 134kN**
	From first to 2nd floor:-	
	Column own weight = 3.65kN/m	
	Load from above = 133.94kN/m	
	From beam 3:- = 0.5 (58 x 6.225) = 180.53kN/m	
	From beam 3:- = 0.5 (58 x 6.225) = 180.53kN/m	
	From beam 11:- = 0.5 (34 x 2.025) = 34.43kN/m	
	= 533.08 kN/m	**Say = 534kN**
	From Ground to 1st floor :-	
	Column own weight =3.65kN/m	
	From above = 533.08kN/m	
	From Beam 3:- = 180.53kN/m	
	From Beam 3:- = 180.53kN/m	
	From Beam 11:- = 34.43kN/m	
	= 932.22kN/m	**Say = 933kN**

	COLUMN C_4 G on grid F4	
	From 2^{nd} floor roof :-	
	Column own weight = 3.65kN/m	
	From roof beam = 0.5 (18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 6.225) = 56.03kN/m	
	From roof beam = 0.5(18 x 2.025) = 18.23kN/m	
	From roof beam = 0.5(18 x 2.025) = 18.23kN/m	
	152.17kN/m	**Say = 153kN**
	From 1^{st} to 2^{nd} floor:-	
	Column own weight = 3.65kN/m	
	Load from above = 152.17kN/m	
	From beam 9:- = 0.5 (79 x 6.225) = 245.89kN/m	
	From beam 9:- = 0.5 (37 x 2.025) = 37.46kN/m	
	From beam 3:- = 0.5 (79 x 6.225) = 245.89kN/m	
	From beam 3:- = 0.5(37 x 2.025) = 37.46kN/m	
	722.52KN/m	**Say = 723kN**
	From Ground to 1^{st} floor :-	
	Column own weight = 3.65kN/m	
	From above = 722.52kN/m	
	From Beam 9:- = 245.89kN/m	
	From Beam 9:- = 37.46kN/m	
	From Beam 3:- = 245.8kKN/m	
	From beam 3:- = 37.46kN/m	
	1292.87kN/m	**Say = 1293kN**
	COLUMN C_1 A_1 (Circular Column) on grid	
	From 2^{nd} floor roof :-	
	Column own weight = 24 x x 3 (π x $0.3^2/4$) = 5.09kN/m	
	From roof beam = 0.5 (18 x 5.203) = 46.83kN/m	
	From roof beam = 0.5(18 x 4.561) = 41.05kN/m	
	92.97kN/m	**Say = 93kN**

	From first to 2^{nd} floor:- Column own weight = 5.09kN/m Load from above = 92.97kN/m From beam 16:- = 0.5 (40 x 4.561) = 91.22kN/m From beam :- = 0.5 (43 x 5.203) = 111.89kN/m = 301.17 kN/m	**Say = 302kN**
	From Ground to 1^{st} floor :- Column own weight = 5.09kN/m From above = 301.17kN/m From Beam 16:- = 91.22kN/m From Beam :- = 111.89kN/m = 509.37kN/m	**Say = 510kN**
	COLUMN DESIGN The design is done as axially loaded column Designing for the most critically loaded column **Column C_2 J** N = 180.97kN $F_y = 460N/mm^2$ $F_{cu} = 25N/mm^2$ $A_c = (225 \times 225) = 50625mm^2$ $Asc = (N - 0.4f_{cu} A_c) / (0.8fy - 0.4f_{cu})$	
	From 2^{nd} floor to roof:- N=180.97kN $Asc = ((180.97 \times 10^3) - (0.4 \times 25 \times 50625))/(0.8 \times 460) - (0.4 \times 25)$ $= 834mm^2$ Provide 4T20mm bars (As provided $=1260mm^2$)	**4T20mm bars**

	Links Provide T10mm @ 200mm centres	**T10@200mm $^{c}/_{c}$**
	From 1^{st} floor to 2^{nd} floor	
	N= 936.66KN	
	Asc $= \dfrac{(936.66 \text{ x } 10^3) - (0.4\text{x}25\text{x } 50625)}{(0.8 \text{ x } 460) \quad - (0.4 \text{ x } 25)} = 1103\text{mm}^2$	
	Provide 4T20 mm bars (As provided = 1260mm^2)	**4T20mm bars**
	Links Provide T10mm @ 200mm centres	**T10@200mm $^{c}/_{c}$**
	From ground floor to 1^{st} floor :-	
	N = 1692.35KN	
	Asc $= \dfrac{(1692.35 \text{ x } 10^3) - (0.4 \text{ x}25 \text{ x } 50625)}{(0.8 \text{ x } 460) \quad -(0.4 \text{ x } 25)} = 3040\text{mm}^2$	
	Provide 4T32mm bars (Asprov = 3440mm^2)	**4T32mm bars**
	Links Provide T10mm @ 200mm centres	**T10@200mm $^{c}/_{c}$**
	Circular Column C1A1	
	From 2^{nd} floor to roof:-	
	N = 92.97kN	
	$A_c = (\pi \text{ x } 300^2/4) = 70685.8\text{mm}^2$	
	$A_{sc} = (92.97 \text{ x}10^3) - (0.4\text{x}25\text{x}70685.8)/ (0.8 \text{ x}460 - 0.4\text{x}25)$ $= 1986\text{mm}^2$	
	Provide nominal reinforcement I.e. $0.4^{o}/_{o}A_c$	
	$A_{S\,min} = (0.4 \text{ x } 70685.8)/100 = 283\text{mm}^2$	
	Provide 6T16mm bars ($A_{S\,prov} = 1210\text{mm}^2$)	**6T16mm bars**
	Links Provide T10mm @ 200mm centres	**T10@200mm $^{c}/_{c}$**

	From 1st floor to 2nd floor	
	$N = 301.17kN$	
	$A_{SC} = ((301.17x10^3) - (0.4x25x70685.8))/ (0.8\ x460 - 0.4x25)$	
	$= -1040mm^2$	
	Provide nominal reinforcement i.e $0.4\%A_c$	
	$A_{S\ min} = (0.4\ x\ 70685.8)/100 = 283mm^2$	
	Provide 6T16mm bars ($A_{S\ prov} = 1210mm^2$)	**6T16mm bars**
	Links	
	Provide T10mm @ 200mm centres	**T10@200mm $^c/_c$**
	From ground floor to 1st floor	
	$N = 509.37kN$	
	$A_{SC} = ((509.37x10^3) - (0.4x25x70685.8))/(0.8\ x460 - 0.4x25)$	
	$= 507mm^2$	
	Provide nominal reinforcement i.e $0.4\%A_c$	
	$A_{S\ min} = (0.4\ x\ 70685.8)/100 = 283mm^2$	
	Provide 6T16mm bars ($A_{S\ prov} = 1210mm^2$)	**6T16mm**
	Links	
	Provide T10mm @ 200mm centres	**T10mm@ 200mm c/c**

	4.5 ANALYSIS AND DESIGN OF FOUNDATION **PAD FOOTING** Column : 225mm x 225mm Dead load from column= 1692.35kN Allowable bearing pressure = $150kN/m^2$ Fy = 460 Fcu = $25N/mm^2$ Let self weight of footing be 5% of the Dead load from column Therefore, 1692.35 x 5/100 = 84.62kN Total Load = 1692.35 + 84.62 = 1776.97kN For Ultimate limit state Design Load = $1.4G_k + 1.6Q_k$ = 1776.97kN For serviceability limit state = 1776.97/1.44 = $1234.01kN/m^2$ Required base area = 1234.01/150 = $8.23m^2$ Provide base 2.9m x 2.9m, Area = $8.41m^2$ Earth pressure = ultimate load/ Base area provided = $(1776.97)/(2.9^2)$ = $211.29kNm^2$ Assume a 500 thick footing and a minimum concrete cover of 50mm, Therefore, d = 500 – 50 – 16/2 = 442mm Anchorage compressive strength = 24ϕ Using 16mm down bars =24 x 16 = 384mm Minimum corer = 50mm Thickness of footing =384 + 50 +16 + 16 = 466mm Provide base thickness of 500mm Effective depth = h – corer –$\theta/2$ = 500 – 50 – 16/2 = 442mm	**Provide 2.9m x 2.9m base**

Net upward pressure = earth pressure – self weight of base

= 211.29– (2.9x2.9 x 24 x 0.5)

110.37kN/m^2

Punching shear

Critical perimeter = column perimeter + 3Лh

=4 (225) + 3Л (500) = 5612.39mm

Area within critical perimeter =

= $(225 + 3h)^2 – (4-Л\ )(1.5h)^2$

$(225 + 3 (500)^2 – (4 -Л\ ) (1.5 x 500)^2$

= 2.49 x 10^6 mm^2

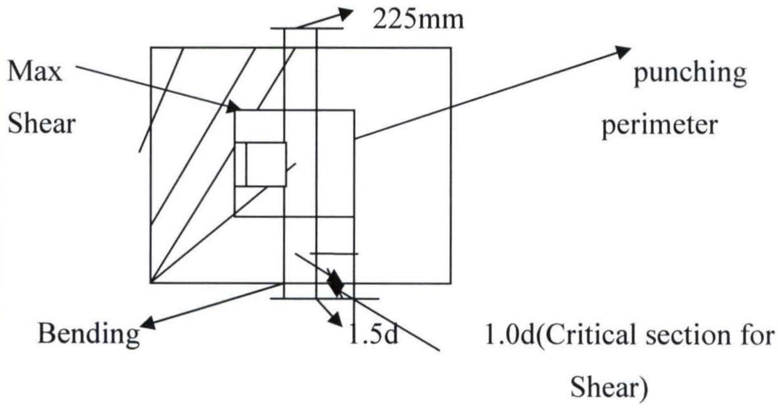
=2.5m^2

Punching shear

= (Net upward pressure) x (area provided-A critical)

= 110.37x (2.9^2 – 2.5)

= 652.28kN



Area within perimeter = $[225+3d]^2$

= $[225+3x442]^2$ = 1825201mm^2

Therefore, punching shear force

= P(A1 – A2) = 1234.01(8.41-1.5)

= 8527.01kNm

Punching shear will occur at 1.5d from the face of the column

H = 500mm will be suitable

Punching shear stress, v,

$$= \frac{V}{\text{Critical perimeter x effective depth}}$$

$(652.28x\ 10^3)/\ (5612.39\ x\ 442) \quad = 0.262$

$0.262 < 0.8\ \sqrt{fcu}$

Thickness of base provide is adequate

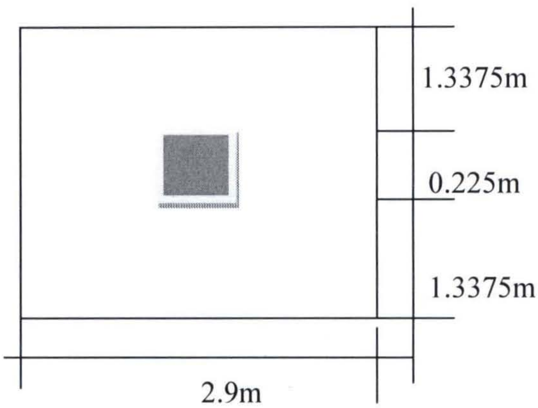
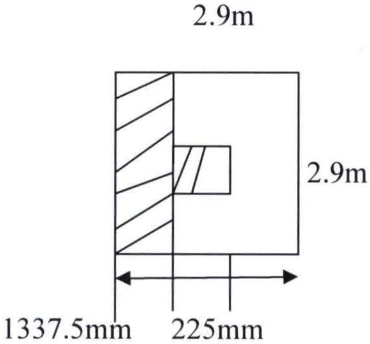


Fig 4.5.1: Foundation Plan

BENDING REINFORCEMENT

At column face which is the critical section



$M = 110.37x\ 2.9\ x\ 0.5\ (1.3375)^2 = 286.29kNm$

$$K = \frac{M}{f_{cu} d^2} = \frac{286.29 \times 10^6}{25 \times 2900 \times 442^2} = 0.020 < 0.156$$

La =0.95

Z = lad = 0.95 x 442 = 419.9mm

$$As = \frac{286.29 \times 10^6}{0.87 \times 460 \times 419.9} = 1{,}704 mm^2$$

Provide 17T20 @175mm centres (As provided = 1800 mm^2)

Therefore, 100As/bh = 100x1800/1000x500 = 0.36> 0.13 as required by Code.

The spacing of 175mm is ok as provided by the code

T20@175 mm

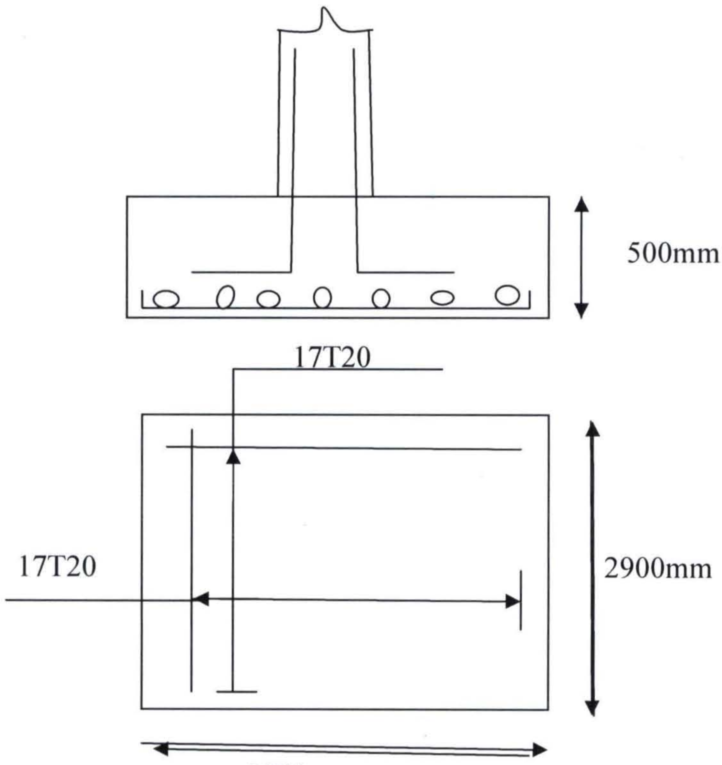


Fig 4.5.2 Foundation Reinforcement Layout

Effective depth = 500 – 40 – 8 = 452mm

Net upward pressure = Earth pressure – self weight

$211.40 - (1.4 \times 24 \times 0.5) = 194.60 kN/m^2$

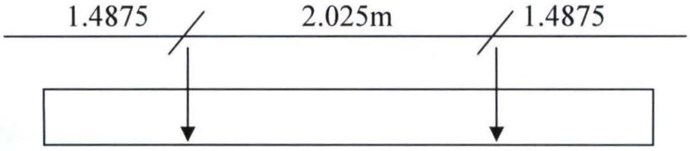
$$\text{Shear stress } v = \frac{LOAD}{COLUMN\ PERIMETER\ EFFECTIVE\ DEPTH}$$

For both column C_3 G and C_4 G

$$V = \frac{1297.22 \times 10^3}{4 \times 225 \times 452} = 3.19 < 0.8\sqrt{Fcu}$$

Thickness provided is adequate to resist shear

Thickness ok



$194.60 KN/m^2 x3 = 583.80KN/m$

Overhang moment (Left and Right)

$583.80 \times 1.4875^2/2 = 645.87 kNm$

Span moment $= wl^2/8 = 583.80 \times 2.025^2/8 = 299.24 kNm$

BENDING REINFORCEMENT

Left and Right overhang

M = 645.87KNm

$K = (645.87 \times 10^6)/(25 \times 1000 \times 452^2) = 0.126 < 0.156$

La = 0.95

Z = la x d = 0.95x452 = 429.4mm

As required

$= (645.87 \times 10^6)/(0.95 \times 460 \times 429.4) = 3442 KN/m$

Provide T25mm @ 125mm centres Bottom

(As provided = $3930 mm^2$)

T25@125

	Span moment M = 299.24KNm $K = (299.24x10^6)/(25x1000x452^2) = 0.059 < 0.156$ La = 0.95 Z = 0.946 x 452 = 429.4mm As required = $(299.24x10^6)/(0.95x460x429.4) = 1595mm^2$ Provide = T25 @ 300mmTop (As provided = $1640mm^2$)	**mm Btm** **T25@300 mm Top**

4.6 ANALYSIS AND DESIGN OF STAIRCASE

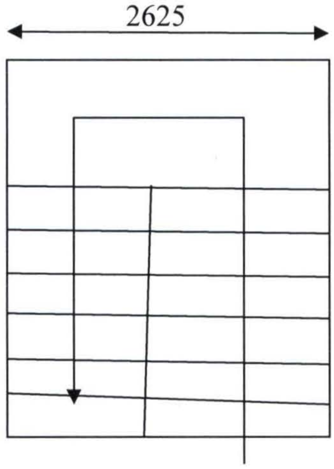


Fig. 4.6.1 staircase plan

Riser = 150mm

No of risers = 13

Tread = 250mm

No of tread = 12

Thickness = 150mm

Concrete cover = 25mm

Effective depth d = 150 – 25 – 6 = 119mm

FIRST FLIGHT

span, L, = (12 x 250) + 0.5 (1500) = 3750mm

Rise = 13 x 150 = 1950mm = 1.95m

slope length = $\sqrt{(3.75^2 + 1.95^2)}$ = 4.23m

Slope factor = $\sqrt{\dfrac{R^2+T^2}{T}}$

$= \sqrt{\dfrac{1.95^2 + 3.25^2}{3.25}} = 1.1662$

loadings :

considering 1m width of stairs

wt of stairs + step=[(0.15x4.23)+(0.25x1.95/2)]24 x 1.1662

= 24.58kN/m

finishes (say) 1.75 kN/m

characteristics dead load = 26.33kN/m

Imposed load, School = 3.0kN/m

characteristics imposed load = 3.0 x 3.75 = 11.25kN/m

ultimate design load,

F= 1.4(26.33)+ 1.6(11.25) = 54.86kN/m

F= 54.86KN/m

M = FL/8 = (54.86 x 3.75) / 8 = 25.72kN/m

K = 25.72 x 10^6 /(25 x 1000 x 119^2) = 0.073

la = 0.5 + √((0.25 - (0.073/ 0.9) = 0.91

z = la x d = 0.91 x 119 = 108.46mm

As = (25.72 x 10^6) / (0.95 x 460 x 108.46) = 543mm^2

Provide T12@200mm centres Bottom (As prov. = 566mm^2)

T12@ 200mm c/cBtm

Transverse distribution steel

As = 0.13% bh = 0.13 x 1000 x 150 /100 = 195mm^2

Provide T12@300 mm centres (As prov. = 377mm^2)

T12@300 mm Btm

Deflection Check:

M/bd^2 = (25.72 x 10^6) / (1000 x 119^2) = 1.82N/mm^2

fs = (2/3)fy(As $_{req.}$ / As $_{prov.}$)

=(2 x 460 x 543)/(3 x 566) = 294.20 N/mm^2

M.F = 0.55 + ((477 – 294.2)/ 120(0.9 + 1.82)) = 1.23

	Limiting span/depth = 1.23 x 26 = 32.0 Actual span / depth = 3750 / 119 = 31.5 Since 32.5 > 31.5 **Deflection is OK**	**Deflection is Satisfied**
	SECOND FLIGHT span, L, = (13x250) + 0.5 (1500 + 1800) = 3600mm Rise = 13 x 150 = 1950mm = 1.95m slope length = $\sqrt{(3.6^2 + 1.95^2)}$ = 4.09m **loadings :** wt of stairs+ step =[(0.15x4.09)+(0.25x1.95/2)]24 = 20.57kN/m finishes (say) = 1.5kN/m characteristics dead load = 22.07kN/m Imposed load, School = 3.0kN/m characteristics imposed load = 3.0 x 3.6 = 10.8kN/m ultimate design load, F = 1.4(22.07) + 1.6(10.8) = 48.18kN/m	**F = 48.18kN/m**
	M = FL/8 = (48.18 x 3.6) / 8 = 21.68kNm K = 21.68 x 10^6/(25 x 1000 x 119^2) = 0.061 la = 0.5 + √((0.25 - (0.061/ 0.9) = 0.93 z = la x d = 0.93 x 119 = 110.67mm As = (21.68 x 10^6) / (0.95 x 460 x 110.67) = 448.3mm^2 Provide T12@200mm centres Bottom (As prov. = 566mm^2)	
	Transverse distribution steel	**T12@200 mm Btm**
	As = 0.13% bh = 0.13 x 1000 x 150 /100 = 195mm^2 Provide T12@300 mm centres (As prov. = 377mm	**T12@300 mm B**

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

For a civil engineer to achieve his objective which is to develop a structure that meets its functional requirements, he must be sure of his analysis and the methods used in the analysis. As such, all the elements were analyzed based on the determined loads and then check for safety. Slabs were designed as one-way, two-way spanning slabs and the other structural members were designed accordingly.

The provision of reinforcement was based on its intended function to resist failure inherent in monolithic construction and thus resistance is provided against all likely causes of damage to the structure.

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