STRUCTURAL DESIGN OF AN OFFICE COMPLEX

BY

JAGUN BOLAKALE

PGD/CIVIL/2009/075

DEPARTMENT OF CIVIL ENGINEERING

FEDERAL UNIVERSITY OF TECHNOLOGY MINNA

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CERTIFICATION

The thesis titled by meets the regulations governing the award of the Post Graduate Diploma of the Federal University of Technology, Minna and it is approved for its contribution to scientific knowledge and literary presentation

Engr. S.F. Oritola **(Project Supervisor)**	20-06-2012 **Signature & Date**
Engr. Professor S. Sadiku **(Head of Department)**	**Signature & Date**
Professor M. S. Abolarin **Dean SEET**	**Signature & Date**
Professor Mrs S. N. Zubairu **Dean PG School**	**Signature & Date**
External Examiner	**Signature & Date**

DECLARATION

I hereby declare that this thesis titled STRUCTURAL DESIGN OF AN OFFICE COMPLEX is a collection of my original research work and it has not been presented for any other qualification anywhere. Information from other sources (published or unpublished) has been duly acknowledged.

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Thank you all God bless.

ABSTRACT

Presented in this project is the Analysis and Design of an Office Complex. The purpose of this project is to enable me have an idea of the structural members of a building using as offices and how to calculate using the limit state method to determine the ultimate limit state in accordance to BS 8110.

Although the project research seems to be simple job at first instant, but in the course of the research, there were many challenging factors, these therefore broaden my understanding in analysis and design of structure.

NOTATIONS

AS	Area of tension reinforcement
AS^1	Area of compression reinforcement
AC	Area of concrete section
Asc	Area of vertical reinforcement in column
Asprov	Area of reinforcement provided
Asv	Area of shear reinforcement
bf	Breadth of flange
bw	Breadth of rib
fy	Characteristics strength of steel
fyv	Characteristics strength of links
Φ	Steel diameter
Asreq	reinforcement area required
Fcu	Characteristics strength of concrete
M	Bending moment
h	overall thickness of slab
qk	Characteristics of live loads
gk	characteristics of dead loads
C	concrete cover
Lx	length of short span
Ly	Length of long span
n	Design ultimate load per unit are area
Vc	Shear force
Mf	Modification factor
Fyv	Characteristics strength of shear reinforcement

Fs	Estimated design service stress of steel
d	Effective depth of tension reinforcement
d^1	Effective depth of compression reinforcement
Ley	Effective height along Y-axis
Lex	Effective height along X-axis
Msx	Design moment about short span
Msy	Design moment about long span
V	Design shear stress at any cross section
Asmin	Minimum area of reinforcement provided
Asmax	Maximum area of reinforcement provided
ULS	Ultimate limit state
SLS	Serviceability limit state

DESIGN INFORMATION

Floor Slabs, Stair Case and Roof Gutter

Characteristics Strength of Steel (fy)	= 410N/mm^2
Concrete Cover (c)	= 20mm
Fire Resistant	= 1 hr
Characteristics strength of concrete (fcu)	= 25N/mm^2

Floor Beams

Characteristics strength of concrete (fcu)	= 25N/mm^2
Characteristics strength of steel (fy)	= 410N/mm^2
Concrete cover (c)	= 20mm
Characteristics strength of steel for stirrup (fyv)	= 250N/mm^2

Retaining Walls

Characteristics strength of steel (fy)	= 410N/mm^2
Concrete cover (c)	= 20mm
Bearing pressure	= 120 KN/m^2
Characteristics strength of concrete	= 25N/mm^2

Columns

Characteristics strength of concrete (fcu)	= 25N/mm^2
Characteristics strength of steel (fy)	= 410N/mm^2
Concrete cover (c)	= 40mm

Foundations

Characteristics strenght of concrete (fcu)	= 25N/mm^2
Characteristics strenght of steel (fy)	= 410N/mm^2
Concrete cover (c)	= 50mm

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CHAPTER ONE

1.1 INTRODUCTION

The purpose of this project is to provide clear analysis of design and detailing of an office complex six storey reinforced concrete building using limit State method in designing.

For this purpose of this project a manual structure design was used for this office complex with retaining walls and elevator.

In order to make the designing easy, the first step is to know the individual properties of materials to be used which consist mainly concrete and steel. Combination of these two component result in a firm durable, long lasting reinforced concrete structure. The use of concrete and steel has grown tremendously over the past decades and it still growing. Concrete has considerable crushing strength, is durable, has good fire resistance but offers little or no strength tension but fair in shear. On the other hand, steel has good tensile proportion, poor resistance to fire and very good in both shear and in compression.

With advancement in the global world, this call for the construction of multi- complex building for variety of purposes such as offices and residential.

1.1.1 AIMS AND OBJECTIVES.

The purpose of design is to achieve acceptable probability that a structure will not reach limit point that will not become unfit for its intended use so that it can have a high degree of safety, sustaining all loads and deformation of normal construction and having adequate durability and good resistance to the effect of misuse and fire.

A reinforces concrete design must satisfy the following objectives:

1) The structure must be economical, that is the factor of satisfy should not be too large to the extent that the cost of structure becomes unbearable.
2) The structure must be safe under the worst system loading.
3) Another vital objective is that the structural elements are to be designed such that economy, safety serviceability, durability and fire resistance will be achieved.

1.2 SCOPE OF WORKS

The scope of this project comprises of designing and detailing of the structural elements which includes roof beam, stab, floor beams, columns, staircases, retaining walls and foundation.

Many materials can be used in calculating moments such as Stiffness flexibility method, Slope Deflection Moments (SDM), Moment Distribution Method (MDM). 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 be also be used to analysis complex structures and is relatively cheap.

CHAPTER ONE

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The first step is designing floor slabs. Slabs are designed mainly for bending. After which the designing of floor beams follow.

All loads from the entire structure from the roof are carried by columns transferred to the foundation. The designs of columns are mainly for axial forces but in few cases for moments.

Lastly, the foundation in designed. Foundations relieve all loads from the columns and spread these loads to the soil. its primary design is for bending and shear.

CHAPTER TWO

2.1 LITERATURE REVIEW

In designing reinforced concrete structure building it is very important for one to understand and identify irrespective of the structural material, the component members that make up a story building, the stress condition these members could be subjected to and how they could be tackled. Also the knowledge of the method of analysis employed, the type of load, how they would occur and how they combined is essential to design efficiently and also making sure that the concrete building behavior is satisfactory under service by the use of BS8110.

Also in designing building itself weight must be known: a building must be able to support itself. The combine weight of all members must be transferred through the columns to the foundation. This will not be problem with building having few floors, this loads will be small compared with the maximum load on each column. As building increases in height the load that must be carrier at the base increases. The advent of steel construction with its highter framework allowed much taller buildings and the skyscraper was born. Whatever the building is to be used for, the structure will have to withstand the loads inherently imposed by its function. For example, taking an office block load caused by furnishing such as Desks, Photocopiers, Cabinets, and so on will have to be considered.

An office building usually designs to accommodate people. The structure must be designed to perform in function and not to interfere with building purpose.

Whatever the size and shape of structure, it will definitely experience loading effects from wind. The large the size of structure the large the effects will be. Large structures would build up in structural members perhaps causing stress fractures or even collapse depending on the strength of the wind on the period of time the building is exposed to it.

The understanding of the individual properties of concrete and steel has lead to the further improvement and use.

2.1.1 ADVANTAGE OF REINFORCED CONCRETE.

1. It has relatively high compressive strength.
2. It has better resistance to fire than steel.
3. It has long service life with low maintenance cost.
4. In some type of structures, such as dams, piers and footings, it is most economical structural material.
5. It can be cast to take the shape required, making it widely used in pre-cast structural components.
6. By using steel, cross sectional dimensions of structural members can be reduced e.g. in lower floor columns.

Factors affecting the joint performance of Steel and Concrete:

2.2 Reinforced Cement Concrete Design Philosophy and Concepts.

The design of a structure may be regarded as the process of selecting proper materials and proportioned elements of the structure, according to the art, engineering science and technology. In order to fulfill its purpose, the structure must meet its conditions of safety, serviceability, economy and functionality.

2.2.1 Strength Design Method

It is based on the ultimate strength of the structural members assuming a failure condition, whether due to the crushing of concrete or due to the yield of reinforced steel bars. Although there is additional strength in the bar after yielding (due to Strain Hardening), this additional strength in the bar is not considered in the analysis or design of the reinforced concrete members. In the strength design method, actual loads or working loads are multiplied by load factor to obtain the ultimate design loads. The load factor represents a high percentage of factors for safety required in the design. The Bs8110 code emphasizes this method of design.

2.2.2 Working Stress Design

This design concept is based on elastic theory, assuming a straight line stress distribution along the depth of the concrete. The actual load or working loads acting on the structure are estimated and members are proportioned on the basis of certain allowable stresses in concrete and steel. The allowable stresses are fractions of the crushing strength of concrete (fc') and the yield strength (fy). Because of the differences in realism and reliability over the past several decades, the strength design method has displaced the older stress design method.

2.2.3 Limit State Design

It is a further step in the strength design method. It indicate the state of the member in which it ceases to meet the service requirements, such as, loosing its ability to withstand external loads or damage. According to limit design, reinforced concrete members have to be analyzed with regard to three limit states:

1. Load carrying capacity (involves safety, stability and durability).

2. Deformation (deflection, vibrations, and impact).

3. The formation of cracks.

The aim of this analysis is to ensure that no limiting sate will appear in the structural member during its service life.

2.3 Fundamental Assumptions for Reinforced Concrete's Behavior

Reinforced concrete's sections are heterogeneous, because they are made up of two different materials - steel and concrete. Therefore, proportioning structural members by ultimate stress design is based on the following assumption:

1. Strain in concrete is the same as in reinforcing bars at the same level, provided that the bond between the concrete and steel is adequate.
2. Strain in concrete is linearly proportional to the distance from the neutral axis.
3. Modulus of elasticity for all grades of steel is taken as Es = 29 x 10 ^ 6psi. The stress in the elastic range is equal to the strain multiplied by Es.
4. Plane cross sections continue to be plane after bending.
5. Tensile strength of concrete is neglected because:
   - Concrete's tensile strength is about 1/10 of its compressive strength.
6. Cracked concrete is assumed to be not effective before cracking; the entire cross section is effective in resisting the external moments.
7. The method of elastic analysis, assuming an ideal behavior at all levels of stress is not valid. At high stresses, non-elastic behavior is assumed, which is in close agreement with the actual behavior of concrete and steel.

2.3.1 Loads

Structural members must be designed to support specific loads. Loads are those forces for which a structure should be proportioned; loads that act on structure can be divided into three categories.

1. Dead loads
2. Live loads
3. Environmental loads.

2.3.2 Dead loads:

Dead loads are those that rare constant in magnitude and fixed in location throughout the lifetime of the structure. It includes the weight of the structure and any permanent materiel placed on the structure, such as roofing, tiles, walls etc. they can be determine with a high degree of accuracy from the dimensions of the elements and the unit weight of the material.

2.3.3 Live loads:

Live loads are those that may vary in magnitude and may also change in location. Live loads consists chiefly occupancy loads in buildings and traffic loads in bridges. Live loads at any given time are uncertain, both in magnitude and distribution.

2.3.4 Environmental Loads:

Consists mainly of snow loads, wind pressure and suction, earthquake loads (i.e. inertial forces) caused by earthquake motion. Soil pressure on subsurface portion of structures. Loads from possible pounding of rainwater bon flat surface and forces caused by temperature differences. Like environmental loads at any given time are uncertain both in magnitude and distribution.

2.4 DESIGN CODES AND STANDARDS.

Basic codes are taken from BS8110 –part 1 and part 2 1985. Also the use of example of the design of reinforced concrete building to BS 8110 fouth edition by charles Reynolds and James C Steadman, Reinforced concrete design by W. H. M and J. H. Bungey and simplified Reinforced concrete design by Engineer Victor O. Oyenuga.

2.5 STRUCTUREAL ANALYSIS.

Structural analysis can be described as a physical laws and mathematical calculation required for prediction of any structures behavior. It is an important part of structural engineering which mainly integrates the judgment of survival loads to be applied on structure. It is used to satisfy the main goals of any structure like internal forces, loads and stresses.

Determination of forces in each member can be determined by the following methods.

a) Apply moment and shear co-efficient.

b) Manual calculatoion.

c) Computer methods.

2.5.1 CONCRETE:

Concrete is the most rudely used man-made construction materials in the world, and is second only to water as the most utilized substance on the planet. It is obtained by mixing cementation materials, water and aggregates (and sometimes admixtures) in required proportions. Aggregates are of two categories namely fine (Sand) and coarse (Gravel or Crushed stones) aggregates. There are two types of concrete.

a) Dense concrete: average density is 2400kglm$^{b.}$

b) Light concrete: average density is 160kglm$^{3.}$

The four main properties of concrete are:

1) Workability.

2) Cohesiveness.

3) Strength

4) Durability.

2.5.2 CEMENT:

The commonly used cement is Portland cement. Other type of cement are rapid-hardening Portland cement, blast furnace Portland cement, high alumina cement low heat Portland cement etc. cement binds the rest of the materials comprises sulphur, aluminum and other elements. It is used in the formation of concrete and plays important roles in making cements mortar mixed with sand for partitioning and other functions.

2.5.3 AGGREGATES:

These are inert filler in the concrete mixture which consist of 70-75% by volume of the mixture. Aggregates are divided into fine aggregate and these include sand and quarry dust and coarse aggregate which are gravel and crushed stones. Aggregates to be used in concrete mix must free impurities, such as Clay, Hard, Tough and Strong

2.5.4 WATER:

The Water used for the mixing and curing of concrete should be free from impurities. Water is most important and least expensive ingredient of concrete. In general, portable water is suitable for concreting.

2.5.5 REINFORCEMENT

They are iron bars or steel of different strength. It could be mild steel or high tensile steel of different sizes. These are embedded in the concrete to increase its strength, making the structure to attain service of construction. Mild steel usually have a smooth surface so that the bond with the concrete is by adhesion only. High yield steel bars have ribbed surface in the form of a twisted square. Therefore, because of their significant stress advantage, high yield bars are the more economical.

2.6 METHODOLOGY AND DESIGN.

The method used in this project research is **LIMIT STATE METHOD**. This method overcomes the disadvantages of other method of designs. It usually involve application of partial factors of safety, both to the load and to the material strength and the magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate State or with more elastic stress range at working load.

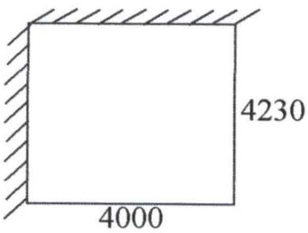
With the method, the purpose of design is to achieve acceptable probabilities that a structure will not become unfit for its intended use that is in will not reach limit State. When designing in accordance with limit state principles as described in CP110 and similar document such as BS8110 part 1 and 2. (1997) each reinforced concrete section is designed first to meet the most critical limit State and then checked to ensure that the remaining limit State are not reached. In using this method, the design of each individual member or section of a member must satisfy the following criteria.

1) The limit State which ensure that the probability of failure in acceptably low.
2) The limit State of serviceability which ensures Safe factor behaviors under service working load. Principal criteria relating to serviceability are prevention of excessive cracking, limit State fatigue, vibration, durability, fire resistance and earth quake resistance must be taken into account into some special cases.

In assesing a particular limit State for a structure it is necessary to consider all the possible variable parameters such as loads, material strengths and constructional tolerances.

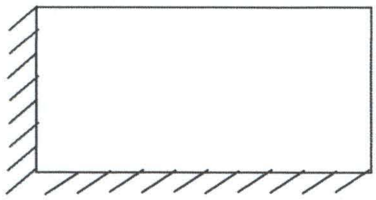
In order to calculate the bending moment and shear forces which member subjected the characteristic loads are multiplied by a partial factor of safety λ_M Hence, if characteristic loads are multiplied by the value of λ_M corresponding to the ultimate limit State, the moments and shear force subsequently determine will represent those occurring at failure and the section must be designed accordingly.

Other methods being previously used in structural designing are the modular ratio method also called elastic theory loads and load factor method.

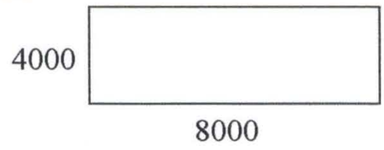
Reference	Calculation	Out put
	CHAPTER THREE	
	GROUND FLOOR SLAB (CAR PARK)	
	Design information	
	Design stresses	
	$Fy = 410N/mm^2$	
	Durability and fire resistance exposure conditions,	
	moderate fire resistance = 1hr	
	Concrete cover = 25mm	
	LOADING	
	Self weight of slab $= 0.15 \times 24 = 3.6KN/m^2$	
	Finishing (say) $= 1.0KN/m^2$	
	Asphalt $= 0.5KN/m^2$	
	Total dead load $= 5.1KN/m^2$	$gk = 5.1KN/m^2$
BS110 Table 2.	Imposed load car park $= 2.5KN/m^2$	$qk = 2.5KN/m^2$
	Design load (n) = 1.4gk + 1.6qk	
	$= 1.4 \times 5.1 + 1.6 \times 2.5$	$n = 11.14KN/m^2$
	$n = 11.14KN/m^2$	
	Panel 1 	
	Ly = 4230	
	Lx = 4000	

Reference	Calculation	Out put
	$ly/lx = \frac{4230}{4000} = 1.06 = 1.1$	
	Depth along short span (dx) = $150 - 25 - \frac{12}{2} = 119mm$	
	Depth along long span (dy) = $150 - 250 - 12 - \frac{12}{2}$	
	$= 107mm$	
	Ultimate bending moment	
	$M_{sx} = B_{sx}nl^2x$	
	$M_{sy} = B_{sy}nl^2x$	
BS8110	$B_{sx} = -0.056 \quad + \quad 0.042$	
Part 1:	$M_{sy} = -0.045 \quad + \quad 0.034$	
1985	$M_{sx} = -0.056 \times 11.14 \times 42 = 9.98Knm$	
Table 3	$M_{sx} = +0.042 \times 11.14 \times 42 = 7.46Knm$	
	$M_{sy} = -0.045 \times 11.14 \times 42 = 8.02Knm$	
	$M_{sy} = +0.034 \times 11.14 \times 42 = 6.06Knm$	
	<u>Short span</u>	
	At edge	
	$K = \frac{M}{Fcu\ bd^2} = \frac{9.98 \times 10^6}{25 \times 1000 \times 119^2} = 0.028$	K = 0.028
	$Z = d(0.5 + \sqrt{0.25 - \frac{K}{0.9}}$	
	$= 119\ (0.5 + \sqrt{0.25 - \frac{0.028}{0.9}} = 115.1mm$	Z = 115.1mm
	$A_{SX} = \frac{9.98 \times 10^6}{0.87 \times 410 \times 115.1} = 243.08mm^2$	
	Provide Y10 @ $200^c/_c$	A_{sx} 243.08mm^2

Reference	Calculation	Out put
	As provide = 393mm^2/m	A_s prov = 393mm^2/m
	At Spam	
	$A_{sx} = \frac{7.46 \, x \, 10^6}{0.87 \, x \, 410 \, x \, 115.1} = 243.08mm^2$	A_{sx} = 182 mm^2/m
	Provide Y 10 @ 200$^c/_c$	As =
	As provide = 393mm^2/m	393mm^2/m
	Long span / *At edge*	
	$K = \frac{8.02 \, x \, 10^6}{25 \, x \, 1000 \, x \, 107^2} = 0.028$	
	$Z = 107(0.5 + \sqrt{0.25 - \frac{0.028}{0.9}} = 103.5$	
	$A_{sy} = \frac{8.02 \, x \, 10^6}{0.87 \, x \, 410 \, x \, 103.5} = 217mm^2$	
	Provide Y10 @ 200$^2/_c$	
	As provide = 393mm^2/m	
	As span	
	$A_{sy} = \frac{8.02 \, x \, 10^6}{0.87 \, x \, 410 \, x \, 103.5} = 164.94mm^2$	
	Provide Y10 @ 200$^c/_c$	
	As prove = 393mm^2/m	
	As prove = 393mm^2/m	
	<u>Check deflection</u>	
BS8110 3.4.6.7 Part 1 1985	$fs = 5/8 \, fy \, x \, \frac{Area \, required}{Area \, prove} \, x \, \frac{1}{\beta}$	

Reference	Calculation	Out put
	$= 5/8 \times 410 \times \frac{243.08}{393} \, x \, \frac{1}{1} = 158.50$N/mm	
	$Mf = 0.55 + \frac{477 - fs}{120\,(0.9 + M/bd^2}$	
	$= 0.55 + \frac{477 - 158.50}{120\left(0.9 + \frac{9.98 \, x \, 10^6}{1000 \, x \, 119^2}\right)} = 2.2$	MF = 2.2
	Limiting $\frac{span}{depth} = 2.2 \times 26 = 57.30$	
	Actual $\frac{span}{depth} = \frac{4000}{119} = 33.61$	
	Deflection 2	
	Panel 2 	
	Ly/lx = 8000 = 2.0	
	$B_{sx} = -0.093$; $B_{sRx} = +0.070$	
	$B_{sy} = -0.045$; $B_{sy} = 0.034$	
	$M_{sBx} = -0.093 \times 11.14 \times 4^2 = 16.57$KN/m^2	
	$M_{sBx} = -0.070 \times 11.14 \times 4^2 = 12.48$KN/m^2	
	$M_{sy} = -0.045 \times 11.14 \times 4^2 = 8.02$KN/m^2	
	$M_{sy} = -0.034 \times 11.14 \times 4^2 = 6.06$KN/m^2	
	$\frac{\textit{At short span}}{\textit{At edge}}$	
	$K = \frac{16.57 \; x \; 10^6}{1000 \; x \; 119^2 \; x \; 25} = 0.047$	K = 0.047
	$Z = 119(0.5 + \sqrt{0.25 - \frac{0.047}{0.9}} = 112.5$mm	Z = 112.5mm

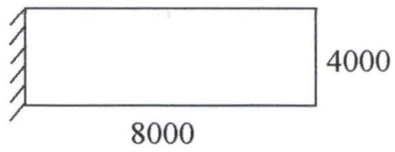
Reference	Calculation	Out put
	$A_{sx} = \dfrac{16.57 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 112.5} = 413mm^2$	$A_{sx} = 413mm^2$
	Provide Y10 @ $175^c/_c$	
	As prove = $449mm^2/m$	A_s prove = $449mm^2/m$
	At span	
	$A_{sx} = \dfrac{12.48 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 112.5} = 310.9mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	$\dfrac{\textit{long span}}{\textit{at edge}}$	
	Provide reinforcement as in panel 1	
	(long span)	
	At span	
	Provide reinforcement as in panel 1	
	Check deflection	
	$fs = 5/8 \times 410 \times \dfrac{413}{449} x \dfrac{1}{1} = 235.7N/mm^2$	Fs = $235.7N/mm^2$
	$mf = 0.55 + \dfrac{477 - 235.7}{120\left(0.9 + \dfrac{16.5 \ x \ 10^6}{1000 \ x \ 119^2}\right)} = 1.52$	Mf = 1.52
	$\dfrac{\textit{Limiting span}}{\textit{depth}} = \dfrac{4000}{119} = 33.61$	
	Deflection ok	

Reference	Calculation	Out put
	<u>Panel 3</u>	
		
	$\frac{ly}{lx} = \frac{8000}{400} = 2.0$	
	$B_{sx} = -0.063; B_{sx} = 0.048$	
	$B_{sy} = -0.032; B_{sy} = 0.024$	
	$M_{sx} = -0.063 \text{ x } 11.14 \text{ x } 4^2 = 11.23KN/m$	
	$M_{sx} = -0.048 \text{ x } 11.14 \text{ x } 4^2 = 6.56KN/m$	
	$M_{sy} = -0.032 \text{ x } 11.14 \text{ x } 4^2 = 5.70KN/m$	
	$M_{sy} = -0.024 \text{ x } 11.14 \text{ x } 4^2 = 4.28KN/m$	
	$\frac{\textit{short span}}{\textit{at edge}}$	
	$K = \frac{11.23 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.032$	
	$Z = 119(0.5 + \sqrt{0.25 - \frac{0.032}{0.9}} = 114.68$	$A_{sx = 274.53mm^2}$
	$A_{sx} = \frac{11.23 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.68} = 274.53mm^2$	
	Provide Y10 @ $200^c/_c$	As prove = $393mm^2/m$
	As prove = $393mm^2/m$	
	<u>At span</u>	A_{sx} = $209.25mm^2$
	$A_{sx} = \frac{8.56 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.68} = 209.25$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	

Reference	Calculation	Out put
	$K = \dfrac{5.70 \ x \ 10^6}{1000 \ x \ 107^2 \ x \ 25} = 0.020$	
	$Z = 107(0.5 + \sqrt{0.25 - \dfrac{0.02}{0.9}} = 104.59$	Z = 104.59
	$A_{sy} = \dfrac{5.7 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 104.59} = 153mm^2$	As prove = $393mm^2/m$
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sy} = \dfrac{4.28 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 104.59} = 114$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	Check deflection	
	$Fs = 5/8 \ x \ 410 \ x \ \dfrac{274.5}{393} = 178.98$	
	$mf = 0.55 + \dfrac{477 - 178.98}{120\left(0.9 + \dfrac{11.23 \, x \, 10^6}{1000 \, x \, 119^2}\right)} = 2.02$	
	$\dfrac{Limiting \ span}{depth} = 2.02 \ x \ 26 = 52.43$	
	$\dfrac{Limiting \ span}{depth} = \dfrac{4000}{119} = 33.61$	
	Deflection ok	

Reference	Calculation	Out put

Panel 4



$$\frac{ly}{lx} = \frac{8000}{400} = 2.0$$

$B_{sx} = -0.067; B_{sx} = 0.050$

$B_{sy} = -0.037; B_{sy} = 0.028$

$M_{sx} = -0.067 \text{ x } 11.14 \text{ x } 4^2 = 11.94KN/m$

$M_{sx} = -0.050 \text{ x } 11.14 \text{ x } 4^2 = 8.91KN/m$

$M_{sy} = -0.037 \text{ x } 11.14 \text{ x } 4^2 = 5.59KN/m$

$M_{sy} = -0.028 \text{ x } 11.14 \text{ x } 4^2 = 4.99KN/m$

$$\frac{\textit{short span}}{\textit{at edge}}$$

$$K = \frac{11.94 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.034$$

K = 0,034

$$Z = 119(0.5 + \sqrt{0.25 - \frac{0.034}{0.9}} = 114.29$$

$$A_{sx} = \frac{11.94 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.29} = 293mm^2$$

Provide Y10 @ $200^c/_c$

As prove = $393mm^2/m$

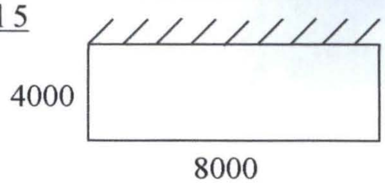
At span

$$A_{sx} = \frac{8.9 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.29} = 218.55$$

Provide Y10 @ $200^c/_c$

As prove = $393mm^2/m$

Reference	Calculation	Out put
	$\dfrac{long\ span}{at\ edge}$	
	$$K = \frac{6.59\ x\ 10^6}{1000\ x\ 107^2\ x\ 25} = 0.023$$	
	$Z = 104.48$	
	$$A_{sy} = \frac{6.59\ x\ 10^6}{0.87\ x\ 410\ x\ 104.48} = 176.82mm^2$$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	At span	
	$$A_{sy} = \frac{4.99\ x\ 10^6}{0.87\ x\ 410\ x\ 104.49} = 116.49mm^2$$	
	Provide Y10 @ $200^c/_c$	
	Check deflection	Fs = 191.05N/nm
	$$Fs = 518 \text{ x } 410 \text{ x } \frac{293}{393}\ x\ \frac{1}{1} = 191.05N/mm^2$$	
	$$mf = 0.55 + \frac{477 - 191.05}{120\left(0.9 + \dfrac{11.94\ x\ 10^6}{1000\ x\ 119^2}\right)} = 1.92$$	
	$$\frac{Limiting\ span}{depth} = 1.92\ x\ 26 = 49.84$$	
	$$\frac{Limiting\ span}{depth} = \frac{4000}{119} = 33.61$$	
	Deflection ok	

Reference	Calculation	Out put
	Panel 5	
		
	$\frac{ly}{lx} = \frac{8000}{4000} = 2.0$	
	$B_{sx} = -0.089$; $B_{sx} = 0.067$	
	$B_{sy} = -0.037$; $B_{sy} = 0.028$	
	$M_{sx} = -0.089 \times 11.14 \times 4^2 = 15.86KN/m$	
	$M_{sx} = -0.067 \times 11.14 \times 4^2 = 11.94KN/m$	
	$M_{sy} = -0.037 \times 11.14 \times 4^2 = 6.59KN/m$	
	$M_{sy} = -0.028 \times 11.14 \times 4^2 = 4.99KN/m$	
	Short span	
	At edge	
	$K = \frac{15.86 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.048$	K = 0.048
	$Z = 113.38mm$	
	$A_{sx} = \frac{15.86 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 113.38} = 392.16mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	At span	
	$A_{sx} = \frac{11.94 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 113.3} = 295.23mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = $393 \ mm^2/m$	
	Long span	

Reference	Calculation	Out put

Provide reinforcement as panel 4 (long span)

Check deflection

$$Fs = 5/8 \times 410 \times \frac{295.23}{393} = 192.50 N/mm^2$$

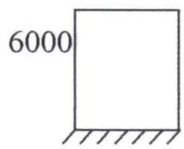
$$Mf = 0.55 + \frac{477 - 192.50}{120\left(0.9 + \frac{11.94 \times 10^6}{1000 \times 119^2}\right)} = 1.91$$

$$\frac{Limiting\ span}{depth} = 1.91 \times 26 = 49.66$$

$$\frac{Actual\ span}{depth} = \frac{4000}{119} = 33.61$$

Deflection ok

Panel 6



$$\frac{ly}{lx} = \frac{6000}{4230} = 1.41 = 1.4$$

$B_{sx} = -0.055$; $B_{sx} = 0.041$

$B_{sy} = -0.037$; $B_{sy} = 0.028$

$M_{sx} = -0.041 \times 11.14 \times 4230^2 = 10.96KNm$

$M_{sx} = 0.041 \times 11.14 \times 4230^2 = 8.17KNm$

$M_{sy} = 0.037 \times 11.14 \times 4230^2 = 7.37KNm$

$M_{sy} = 0.028 \times 11.14 \times 4230^2 = 5.58KNm$

Short span

At edge

$$K = \frac{10.86 \times 10^6}{1000 \times 119^2 \times 25} = 0.031$$

Reference	Calculation	Out put
	$Z = 119(0.5 + \sqrt{0.25 - \frac{0.031}{0.9}}\ 114.81mm$	K = 0.031
	$A_{sx} = \frac{10.96 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.81} = 267.63mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	As prove = $393mm/m^2$
	$A_{sx} = \frac{8.17 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 114.81} = 200mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	Long span	
	At edge	
	$K = \frac{7.37 \ x \ 10^6}{1000 \ x \ 107^2 \ x \ 25} = 0.026$	
	$Z = 107(0.5 + \sqrt{0.25 - \frac{0.026}{0.9}}\ 103.8mm$	
	$A_{sy} = \frac{7.37 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 103.81} = 199.05mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sy} = \frac{5.58 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 103.8} = 151mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	Check deflection	

Reference	Calculation	Out put

$$Fs = 5/8 \times 410 \times \frac{200}{393}\ \frac{1}{1} = 130.41$$

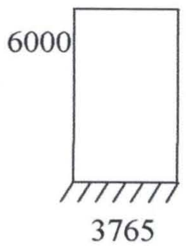
$$Mf = 0.55 + \frac{477 - 130.41}{120\left(0.9 + \frac{10.96 \, x \, 10^6}{1000 \, x \, 119^2}\right)} = 2.27$$

$$\frac{Limiting\ span}{depth} = 2.27 \, x \, 26 = 59.16$$

$$\frac{Actual\ span}{depth} = \frac{4230}{119} = 35.55$$

Deflection ok

Panel 7



$$\frac{ly}{lx} = \frac{6000}{3765} = 1.59 \text{ by interpretation}$$

= 1.855 = 1.9 w 2.0

B_{sx} = -0.067 ; B_{sx} = 0.050

B_{sy} = -0.037 ; B_{sy} = 0.028

M_{sx} = -0.067 x 11.14 x 3765^2 = 10.56KNm

M_{sx} = 0.050 x 11.14 x 3765^2 = 7.90KN/m

M_{sy} = 0.028 x 11.14 x 3765^2 = 5.84KN/m

M_{sy} = 0.028 x 11.14 x 3765^2 = 4.42KN/m

Short span

At edge

$$K = \frac{10.58 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.0299$$

Reference	Calculation	Out put
	$Z = 119(0.5 + \sqrt{0.25 - \frac{0.0299}{0.9}}\ 114.93mm$	
	$A_{sx} = \frac{10.58\ x\ 10^6}{0.87\ x\ 410\ x\ 114.93} = 192.7mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sx} = \frac{7.90\ x\ 10^6}{0.87\ x\ 410\ x\ 114.93} = 192.7mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	Long span	
	At edge	
	$K = \frac{5.84\ x\ 10^6}{1000\ x\ 107^2\ x\ 25} = 0.020$	
	$Z = 104.48mm$	
	$A_{sy} = \frac{5.84\ x\ 10^6}{0.87\ x\ 410\ x\ 104.48} = 156.7mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sy} = \frac{11.94\ x\ 10^6}{0.87\ x\ 410\ x\ 113.3} = 295.23mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	Check deflection	

Reference	Calculation	Out put

$$Fs = 5/8 \times 410 \times \frac{19.72}{393}\ \frac{1}{1} = 125.65N/mm^2$$

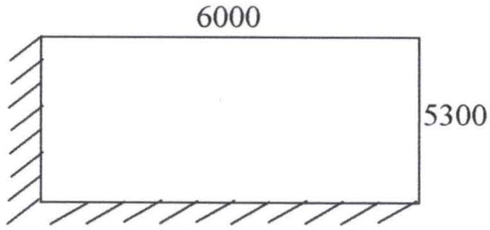
$$Mf = 0.55 + \frac{477 - 125.65}{120\left(0.9 + \dfrac{10.58 \times 10^6}{1000 \times 119^2}\right)} = 2.23$$

$$\frac{\textit{Limiting span}}{\textit{depth}} = 2.33 \times 26 = 60.58$$

$$\frac{\textit{Actual span}}{\textit{depth}} = \frac{3765}{119} = 31.64$$

Deflection ok

Panel 8



$$\frac{ly}{lx} = \frac{6000}{5300} = 1.13 = 1.1$$

$B_{sx} = -0.056$; $B_{sx} = 0.042$

$B_{sy} = -0.045$; $B_{sy} = 0.034$

$M_{sx} = -0.056 \times 11.14 \times 5.3^2 = 17.53KNm$

$M_{sx} = 0.042 \times 11.14 \times 5.3^2 = 13.14KNm$

$M_{sy} = 0.045 \times 11.14 \times 5.3^2 = 14.08KNm$

$M_{sy} = 0.034 \times 11.14 \times 5.3^2 = 10.64KNm$

$$\frac{\textit{short span}}{\textit{at edge}}$$

$$K = \frac{17.53 \times 10^6}{1000 \times 119^2 \times 25} = 0.050$$

Reference	Calculation	Out put

$$Z = 119(0.5 + \sqrt{0.25 - \frac{0.05}{0.9}}\ 112.05\text{mm}^2$$

$$A_{sx} = \frac{17.53 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 112.05} = 438.60mm^2$$

Provide Y10 @ $200^c/_c$

As prove = $566\text{mm}^2/\text{m}$

<u>At span</u>

$$A_{sx} = \frac{13.14 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 112.05} = 328.76mm^2$$

Provide Y 10 @ $200^c/_c$

As prove = $566\text{mm}^2/\text{m}$

<u>*long span*</u>

at edge

$$K = \frac{14.08 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.040$$

$$Z = 107(0.5 + \sqrt{0.25 - \frac{0.04}{0.9}} = 102.06$$

$$A_{sy} = \frac{14.08 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 102.06} = 386.76mm^2$$

Provide Y10 @ $200^c/_c$

As prove = $393\text{mm}^2/\text{m}$

<u>At span</u>

$$A_{sy} = \frac{10.64 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 102.6} = 292.27mm^2$$

Provide Y 10 @ $200^c/_c$

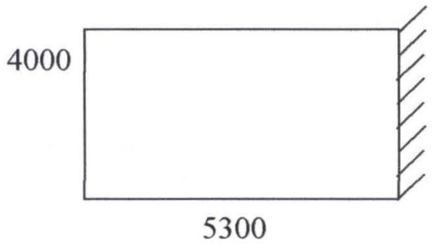
As prove = 393 mm^2/m

<u>Check deflection</u>

Reference	Calculation	Out put
	$Fs = 5/8 \times 410 \times \frac{328.76}{566} \frac{1}{1} = 148.54$	
	Mf = 2.05	
	$\frac{Limiting\ span}{depth} = 26.x\ 2.05 = 53.20$	
	$\frac{Actual\ span}{depth} = \frac{5300}{119} = 44.54$	
	Deflection ok	
	$Z = 107(0.5 + \sqrt{0.25 - \frac{0.04}{0.9}} = 102.06$	
	$A_{sy} = \frac{14.08\ x\ 10^6}{0.87\ x\ 410\ x\ 102.06} = 386.76mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sy} = \frac{10.64\ x\ 10^6}{0.87\ x\ 410\ x\ 102.6} = 292.27mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	<u>Check deflection</u>	
	$Fs = 5/8 \times 410 \times \frac{328.76}{566} \frac{1}{1} = 148.54$	
	Mf = 2.05	
	$\frac{Limiting\ span}{depth} = 26\ x\ 2.05 = 53.20$	
	$\frac{Actual\ span}{depth} = \frac{5300}{119} = 44.54$	
	Deflection ok	

Reference	Calculation	Out put

Panel 9



$$\frac{ly}{lx} = \frac{5300}{4000} = 1.32 \underline{w} 1.3$$

$B_{sx} = -0.052$; $B_{sx} = 0.039$

$B_{sy} = -0.037$; $B_{sy} = 0.028$

$M_{sx} = -0.052 \times 11.14 \times 4^2 = 9.26KNm$

$M_{sx} = 0.039 \times 11.14 \times 4^2 = 6.95KNm$

$M_{sy} = 0.037 \times 11.14 \times 4^2 = 6.59KNm$

$M_{sy} = 0.028 \times 11.14 \times 4^2 = 4.99KNm$

$$\frac{short\ span}{at\ edge}$$

$$K = \frac{9.26 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.026$$

$Z = 115.44$

$$A_{sx} = \frac{9.26 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 115.44} = 224.88mm^2$$

Provide Y10 @ $200^c/_c$

As prove = $393mm^2/m$

At span

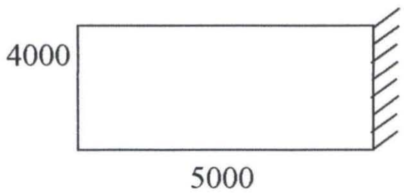
$$A_{sx} = \frac{6.95 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 115.44} = 168.84mm^2$$

Provide Y 10 @ $200^c/_c$

Reference	Calculation	Out put
	As prove = $566mm^2/m$	
	$\dfrac{\textit{long span}}{\textit{at edge}}$	
	$K = \dfrac{14.08 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.040$	
	$Z = 104.70mm$	
	$A_{sy} = \dfrac{6.59 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 104.70} = 176.46mm$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sy} = \dfrac{4.99 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 104.70} = 133.6mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	<u>Check deflection</u>	
	$Fs = 5/8 \ x \ 410 \ x \ \dfrac{168.84}{393} \ \dfrac{1}{1} = 110.09$	
	$Mf = 0.55 + \dfrac{477 - 110.09}{120\left(0.9 + \dfrac{6.95 \ x \ 10^6}{1000 \ x \ 119^2}\right)} = 2.75$	
	$\dfrac{\textit{Limiting span}}{\textit{depth}} = 2.75 \ x \ 26 = 71.46$	
	$\dfrac{\textit{Actual span}}{\textit{depth}} = \dfrac{4000}{119} = 33.61$	
	Deflection ok	

Reference	Calculation	Out put

Panel 10



$$\frac{ly}{lx} = \frac{5000}{4000} = 1.25 = 1.3$$

$B_{sx} = -0.052\ ;\ 0.039$

$B_{sy} = -0.037\ ;\ 0.028$

$M_{sx} = -0.052 \times 11.14 \times 4^2 = 9.26KNm$

$M_{sx} = 0.039 \times 11.14 \times 4^2 = 6.95KNm$

$M_{sy} = -0.037 \times 11.14 \times 4^2 = 6.59KNm$

$M_{sy} = 0.028 \times 11.14 \times 4^2 = 4.99KNm$

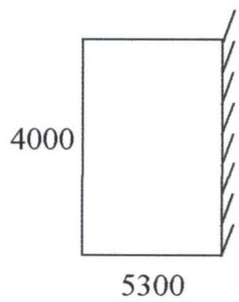
<u>Short span</u>

Provide reinforcement as in panel 9 i.e Y10 at $200^c/_c$ for both edge and span

<u>Long span</u>

Provide reinforcement as in panel 9 i.e Y10 @ 200% both edge and span.

<u>Panel 11</u>



$$\frac{ly}{lx} = 4000 = 1.05 \ \underline{w}\ 1.1$$

Reference	Calculation	Out put
	B_{sx} = -0.049 ; 0.036	
	B_{sy} = -0.037 ; 0.028	
	$M_{sx} = 0.049 \times 11.14 \times 3.8^2 = 7.88KNm$	
	$M_{sx} = 0.036 \times 11.14 \times 3.8^2 = 5.79KNm$	
	$M_{sy} = 0.037 \times 11.14 \times 3.8^2 = 5.59KNm$	
	$M_{sy} = 0.028 \times 11.14 \times 3.8^2 = 4.5KNm$	
	short span	
	at edge	
	$K = \frac{7.88 \ x \ 10^6}{1000 \ x \ 119^2 \ x \ 25} = 0.0223$	
BS 8110 3.4.4.4	$Z = 119(0.5 + \sqrt{0.25 - \frac{0.0223}{0.9}} = 115.95$	
	$A_{sx} = \frac{7.88 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 115.95} = 190.5mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	At span	
	$A_{sx} = \frac{5.79 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 115.95} = 140mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = $393 \ mm^2/m$	
	At span	
	$A_{sx} = \frac{5.79 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 115.95} = 140mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = $393mm^2/m$	

Reference	Calculation	Out put

$\frac{\textit{long span}}{\textit{at edge}}$

$$K = \frac{5.95 \ x \ 10^6}{1000 \ x \ 107^2 \ x \ 25} = 0.0208$$

$Z = 104.48mm$

$$A_{sy} = \frac{5.59 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 104.48} = 59.65mm$$

Provide Y10 @ $200^c/_c$

As prove = $393mm^2/m$

<u>Check deflection</u>

Bs 8110 3.4.6.7

$$Fs = 5/8 \times 410 \times \frac{140}{393} \ \frac{1}{1} = 91.28$$

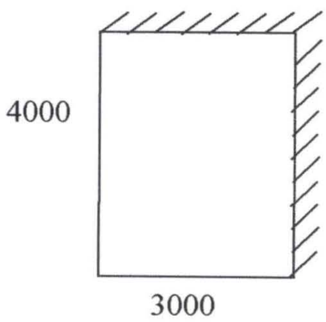
$$Mf = 0.55 + \frac{477 - 91.25}{120\left(0.9 + \frac{7.88 \, x \, 10^6}{1000 \, x \, 119^2}\right)} = 2.76$$

$$\frac{\textit{Limiting span}}{\textit{depth}} = 2.76 \, x \, 26 = 71.68$$

$$\frac{\textit{Actual span}}{\textit{depth}} = \frac{3800}{119} = 31.93$$

Deflection ok

Panel 12



$$Ly/lx = \frac{4000}{3000} = 1.33 = 1.3$$

Reference	Calculation	Out put
Bs 8110 part 1 1985 Table 3	B_{sx} = - 0.069 ; = 0.051 B_{sy} = - 0.045 ; = 0.034 $M_{sx} = -\ 0.069 \times 11.14 \times 3^2 = 6.92$KN/m $M_{sx} = -\ 0.051 \times 11.14 \times 3^2 = 5.11$KN/m $M_{sy} = -\ 0.045 \times 11.14 \times 3^2 = 4.51$KN/m $M_{sy} = -\ 0.034 \times 11.14 \times 3^2 = 3.40$KN/m^2 *short span* *At edge* $K = \dfrac{6.92 \ x \ 10^6}{0.00 \ x \ 119^2 \ x \ 25} = 0.0195$ $Z = 116.32$ $A_{sx} = \dfrac{6.92 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 116.3.2} = 166.78mm^2$ Provide Y10 @ 175$^c/_c$ As prove = 393mm^2/m At span $A_{sx} = \dfrac{5.11 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 11.32} = 123.16mm^2$ Provide Y 10 @ 200$^c/_c$ As prove = 393 mm^2/m Long span $K = \dfrac{4.51 \ x \ 10^6}{1000 \ x \ 107^2 \ x \ 25} = 0.016$ $Z = 105.04$ At edge	

Reference	Calculation	Out put

Terrazzo tiles $= 0.025 \times 22 = 0.55KN/m^2$

Cement mortar $= 0.0125 \times 20 = 0.25KN/m^2$

Partition wall (say) $= 2.5KN/m^2$

Total loading gk $=$ $= 7.5KN/m^2$

Imposed load $qk = 2.5KN/m^2$ for general office

Design load $(n) = 1.4gk + 1.6qk$

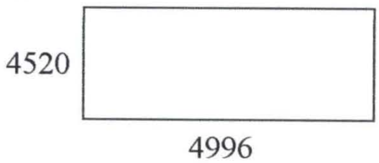
$$= 1.4 \times 7.5 + 1.6 \times 2.5$$

$$n = 14.5KN/m^2$$

short span depth $dx = 175 - 25 - \frac{12}{2} = 144mm$

Long span depth $dy = 175 - 25 - 12 - 6 = 138mm$

<u>Panel 4</u>



$\frac{ly}{lx} = 4996 = 1.10$

$B_{sx} = 0.037; = 0.028$

$B_{sy} = -0.032; = 0.024$

$M_{sx} = -0.037 \times 14.5 \times 4520^2 = 10.97KNm$

$M_{sx} = 0.028 \times 14.5 \times 4520^2 = 8.29KNm$

$M_{sy} = -0.032 \times 14.5 \times 4520^2 = 9.48KNm$

$M_{sy} = 0.024 \times 14.5 \times 4520^2 = 7.11KNm$

$\frac{\textit{short span}}{\textit{at edge}}$

Reference	Calculation	Out put
	$\frac{ly}{lx} = \frac{4996}{4056} = 1.23 = 1.2$	
	$B_{sx} = -0.056; B_{sx} = 0.042$	
	$B_{sy} = -0.037; B_{sy} = 0.028$	
	$M_{sx} = -0.056 \text{ x } 14.5 \text{ x } 4.056^2 = 13.36\text{KNm}$	
	$M_{sx} = -0.042 \text{ x } 14.5 \text{ x } 4.056^2 = 10.02\text{KNm}$	
	$M_{sy} = 0.037 \text{ x } 14.5 \text{ x } 4.056^2 = 8.83\text{KNm}$	
	$M_{sy} = 0.028 \text{ x } 14.5 \text{ x } 4.056^2 = 6.68\text{KNm}$	
	$\frac{\textit{short span}}{\textit{at edge}}$	
	$K = \frac{13.36 \ x \ 10^6}{1000 \ x \ 114^2 \ x \ 25} = 0.028$	
	$Z = 144(0.5 + \sqrt{0.25 - \frac{0.026}{0.9}} = 139.70$	
	$A_{sx} = \frac{13.36 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 139.70} = 268.11mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393\text{mm}^2/\text{m}$	
	<u>At span</u>	
	$A_{sx} = \frac{10.02 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 139.7} = 261.28mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	$\frac{\textit{longspan}}{\textit{at edge}}$	
	$K = \frac{8.83 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 139.7} = 261.28mm^2$	

Reference	Calculation	Out put
	$Z = 135.04$	
	$A_{sy} = \dfrac{8.83 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 135.04} = 183.31mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	At span	
	$A_{sy} = \dfrac{6.68 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 135.04} = 183.31mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>Check deflection</u>	
	$Fs = 5/8 \ x \ 410 \ x \ \dfrac{261.28}{393} \ x \ \dfrac{1}{1} = 170.36$	
	$mf = 0.55 + \dfrac{477 - 170.36}{120\left(0.9 + \dfrac{m}{bd^2}\right)} = 2.98$	
	$\dfrac{Limiting \ span}{depth} = 2.4 \ x \ 26 = 62.33$	
	$\dfrac{Actual \ span}{depth} = \dfrac{4056}{144} = 28.17$	
	Deflection ok	
	TYPICAL FLOOR SLAB	
	Assume slab thickness = 175mm	
	Concrete cover = 25mm	
	LOADINGS	
	Self weight of slab = 0.175 x 24 = $4.25KN/m^2$	

Reference	Calculation	Out put
	$K = \dfrac{10.96 \ x \ 10^6}{1000 \ x \ 114^2 \ x \ 25} = 0.0211$	
	$Z = 140.91mm$	
	$A_{sx} = \dfrac{10.96 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 140.99} = 218.07mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>At span</u>	
	$A_{sx} = \dfrac{8.29 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 140.91} = 164.93mm^2$	
	Provide Y 10 @ $200^c/_c$	
	As prove = 393 mm^2/m	
	<u>*long span*</u>	
	at edge	
	$K = \dfrac{10.44 \ x \ 10^6}{1000 \ x \ 138^2 \ x \ 25} =$	
	Z = 134.89	
	At edge	
	$A_{sy} = \dfrac{10.4 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 134.89} = 216.98mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	As span	
	$A_{sy} = \dfrac{7.89 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 134.89} = 142.66mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	

Reference	Calculation	Out put

Check deflection

$$Fs = 5/8 \times 410 \times \frac{332.23}{393} \, x \, \frac{1}{1} = 150.87$$

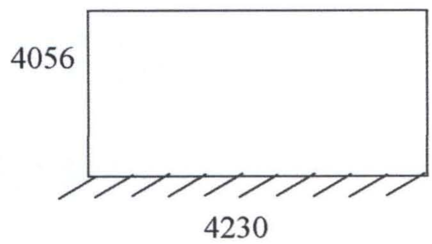
$$mf = 0.55 + \frac{477 - 150.87}{120\left(0.9 + \frac{m}{bd^2}\right)} = 2.16$$

$$\frac{Limiting\ span}{depth} = 2.16 \, x \, 26 = 56.27$$

$$\frac{Actual\ span}{depth} = \frac{4000}{144} = 27.78$$

Deflection ok

Panel 5



$$\frac{ly}{lx} = \frac{4230}{4056} = 1.04 = 1.0$$

$B_{sx} = 0.039$; $= 0.030$

$B_{sy} = -0.037$; $= 0.028$

$M_{sx} = -0.039 \times 14.5 \times 4.056^2 = 9.30KNm$

$M_{sx} = 0.028 \times 14.5 \times 4.056^2 = 7.17KNm$

$M_{sy} = -0.032 \times 14.5 \times 4.056^2 = 8.83KNm$

$M_{sy} = 0.024 \times 14.5 \times 4.056^2 = 6.68KNm$

$$\frac{short\ span}{at\ edge}$$

$$K = \frac{9.30 \, x \, 10^6}{1000 \, x \, 114^2 \, x \, 25} = 0.018$$

Reference	Calculation	Out put

$Z = 141.06$

$$A_{sx} = \frac{9.30 \; x \; 10^6}{0.87 \; x \; 410 \; x \; 141.06} = 184.83mm^2$$

Provide Y10 @ $200^c/_c$

As prove = $393mm^2/m$

<u>At span</u>

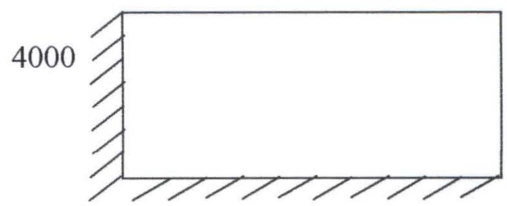
$$A_{sx} = \frac{7.16 \; x \; 10^6}{0.87 \; x \; 410 \; x \; 141.06} = 184.30mm^2$$

Reinforcement as in edge span

LONG SPAN

Reinforcement as in short span

Panel 8



$$\frac{ly}{lx} = \frac{8000}{4000} = 0.2$$

$B_{sx} = 0.093 \; ; = 0.070$

$B_{sy} = -0.045 \; ; = 0.034$

$M_{sx} = -0.093 \text{ x } 14.5 \text{ x } 4^2 = 21.5KNm$

$M_{sx} = 0.070 \text{ x } 14.5 \text{ x } 4^2 = 16.24KNm$

$M_{sy} = -0.045 \text{ x } 14.5 \text{ x } 4^2 = 10.44KNm$

$M_{sy} = 0.034 \text{ x } 14.5 \text{ x } 4^2 = 7.89KNm$

$$\frac{\textit{short span}}{\textit{at edge}}$$

Reference	Calculation	Out put

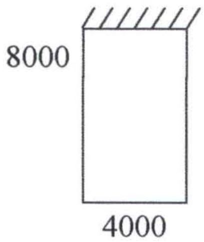
$$mf = 0.55 + \frac{477 - 146.09}{120\left(0.9 + \frac{m}{bd^2}\right)} = 2.49$$

$$\frac{Limiting\ span}{depth} = 2.47\ x\ 26 = 64.19$$

$$\frac{Actual\ span}{depth} = \frac{4000}{144} = 27.78$$

Deflection ok

Panel 8



$$\frac{ly}{lx} = \frac{8000}{4000} = 2.0$$

$B_{sx} = 0.063$; $= 0.048$

$B_{sy} = -0.032$; $= 0.024$

$M_{sx} = -0.063 \text{ x } 14.5 \text{ x } 4^2 = 14.62\text{KNm}$

$M_{sx} = 0.048 \text{ x } 14.5 \text{ x } 4^2 = 11.14\text{KN/m}$

$M_{sy} = -0.032 \text{ x } 14.5 \text{ x } 4^2 = 7.42\text{KN/m}$

$M_{sy} = 0.024 \text{ x } 14.5 \text{ x } 4^2 = 6.50\text{KN/m}$

$$\frac{short\ span}{at\ edge}$$

$$K = \frac{114.62\ x\ 10^6}{1000\ x\ 114^2\ x\ 25} = 0.028$$

$Z = 139.39\text{mm}$

$$A_{sx} = \frac{14.62\ x\ 10^6}{0.87\ x\ 410^2\ x\ 139.39} = 294.04mm^2$$

Provide Y10 @ $200^c/_c$

Reference	Calculation	Out put
	As prove = 393mm²/m	

As prove $= 393mm^2/m$

<u>At span</u>

$$A_{sx} = \frac{11.14 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 139.39} = 224.05mm^2$$

Provide Y 10 @ $200^c/_c$

As prove $= 393 \ mm^2/m$

$$\frac{\textit{long span}}{\textit{at edge}}$$

$$K = \frac{7.42 \ x \ 10^6}{1000 \ x \ 138^2 \ x \ 25} = 0.016$$

Z = 135.61

$$A_{sy} = \frac{7.42 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 135.61} = 153.39mm^2$$

Provide Y10 @ $200^c/_c$

As prove $= 393mm^2/m$

As span

$$A_{sy} = \frac{6.50 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 135.61} = 134.38mm^2$$

Provide Y10 @ $200^c/_c$

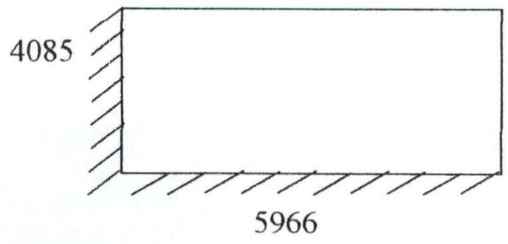
As prove $= 393mm^2/m$

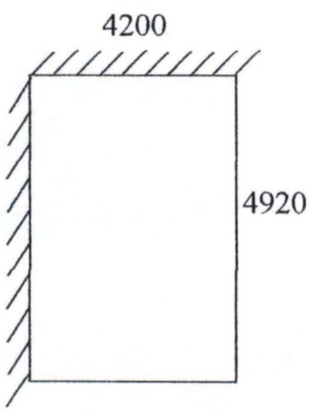
<u>Check deflection</u>

$$Fs = 5/8 \ x \ 410 \ x \ \frac{224.05}{393} \ x \ \frac{1}{1} = 146.09$$

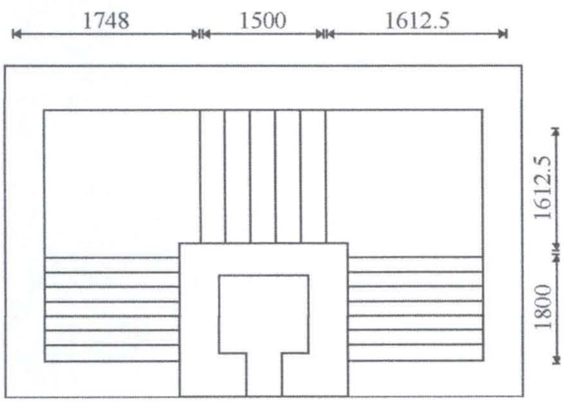
$$mf = 0.55 + \frac{477 - 146.09}{120\left(0.9 + \frac{11.14 \times 10^6}{bd^2}\right)} = 2.47$$

$$\frac{\textit{Limiting span}}{\textit{depth}} = 2.47 \ x \ 26 = 64.18$$

Reference	Calculation	Out put
	Panel 11	
		
	$\frac{ly}{lx} = \frac{5966}{4088} = 1.46 = 1.5$	
Bs 8110 3.5.3.3	$B_{sx} = -0.078\ ;\ 0.059$ $B_{sy} = -0.045\ ;\ 0.034$	
	$M_{sx} = -0.078 \text{ x } 14.5 \text{ x } 4.085^2 = 18.87\text{KN/m}$	
	$M_{sx} = 0.059 \text{ x } 14.5 \text{ x } 4.085^2 = 14.28\text{KN/m}$	
	$M_{sy} = -0.045 \text{ x } 14.5 \text{ x } 4.085^2 = 10.89\text{KN/m}$	
	$M_{sy} = 0.034 \text{ x } 14.5 \text{ x } 4.085^2 = 8.23\text{KN/m}$	
	$\frac{\textit{short span}}{\textit{at edge}}$	
	$K = \frac{18.87 \ x \ 10^6}{1000 \ x \ 144^2 \ x \ 25} = 0.06$	
Bs 8110 3.4.4.4	$Z = 144(0.5 + \sqrt{0.25 - \frac{0.036}{0.9}}\ 137.83\text{mm}$	
	$A_{sx} = \frac{18.87 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 137.83} = 383.8mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393\text{mm}^2/\text{m}$	
	$A_{sx} = \frac{14.28 \ x \ 10^6}{0.87 \ x \ 410 \ x \ 137.83} = 290.46mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393\text{mm}^2/\text{m}$	
	$Z = 134.46\text{mm}$	

Reference	Calculation	Out put
	$A_{sx} = \dfrac{10.89\ x\ 10^6}{0.87\ x\ 410\ x\ 134.46} = 227.06mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>Check deflection</u>	
	$Fs = 5/8 \text{ x } fy \text{ x } \dfrac{Area\ Required}{Area\ Provided}\ x\ \dfrac{1}{\beta}$	
Bs 8110 3.4.6.7	$Fs = 5/8 \text{ x } 410 \text{ x } \dfrac{290.46}{393}\ x\ \dfrac{1}{1} = 189.39$	
Bs 8110 3.4.6.7	$Mf = 0.55 + \dfrac{477 - 189.39}{120\left(0.9 + \dfrac{18.83 \times 10^6}{1000 \times 144^2}\right)} = 1.87$	
	$Limiting\ span = 1.87\ x\ 26 = 48.71$	
	$Actual\ span = \dfrac{4085}{144} = 28.37$	
	Deflection ok	
	Panel 12	
		
	$ly/lx = \dfrac{4920}{4200} = 1.17 = 1.2$	
	B_{sx} = -0.063 ; 0.047	

Reference	Calculation	Out put
	B_{sy} = -0.045 ; 0.034	
	$M_{sx} = -0.063 \times 14.5 \times 4.2^2 = 16.11KNm$	
	$M_{sx} = 0.047 \times 14.5 \times 4.2^2 = 12.02KNm$	
	$M_{sy} = -0.045 \times 14.5 \times 4.2^2 = 11.51KNm$	
	$M_{sy} = 0.034 \times 14.5 \times 4.2^2 = 8.70KNm$	
	short span / *at edge*	
	$K = \dfrac{16.11 \times 10^6}{1000 \times 144^2 \times 25} = 0.031$	
	$Z = 144(0.5 + \sqrt{0.25 - \dfrac{0.031}{0.9}} = 138.77mm$	
	$A_{SX} = \dfrac{16.11 \times 10^6}{0.87 \times 410 \times 138.77} = 325.46mm^2$	
	Provide Y10 @ $200^c/_c$	
	As provide = $393mm^2/m$	
	At Spam	
	$A_{sx} = \dfrac{12.02\ x\ 10^6}{0.87\ x\ 410\ x\ 138.77} = 242.83mm^2$	
	Provide Y 10 @ $200^c/_c$	

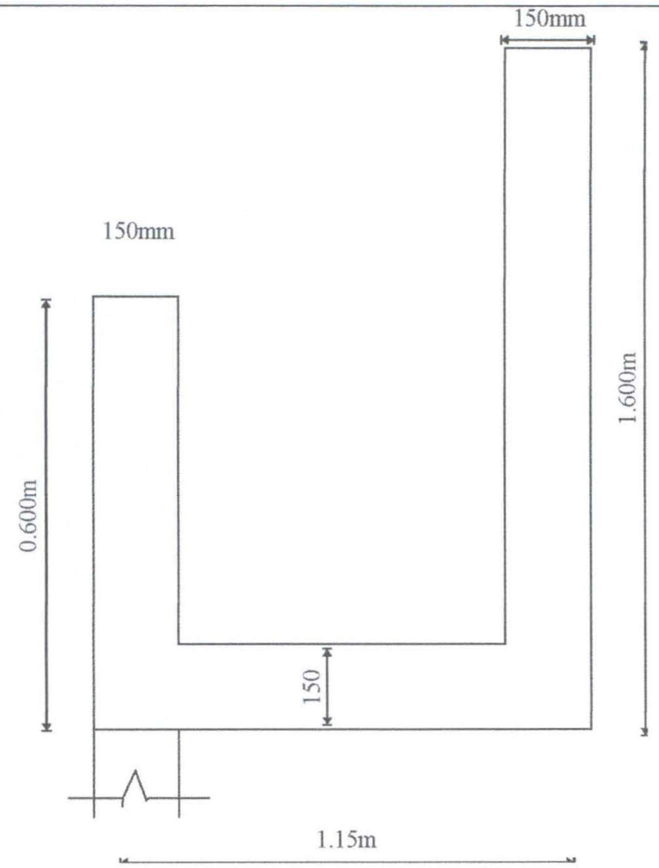
Reference	Calculation	Out put
	STAIR CASE DESIGN	
	Rise = 1.57.5 (R)	
	Tread = 300mm (T)	
	Waist = 200mm	
	Slope factor = $\frac{\sqrt{R^2 \quad T^2}}{T} + \frac{\sqrt{157.5^2 + 300^2}}{300} = 1.29$	
	d = h – c – ½ Φ = 150 – 20 – 6 = 124mm	
		
Flights	Loading	
	Self weight = 0.15 x 24 = 3.6KN/m^2 Finishing (say) 1.6KN/m^2 4.6KN/m^2	
Bs6399: Part 1 1984 Table 5	Steps = 0.5 x 0.1575 x 24 = 1.89KN/m^2 Live load = 3.0KN/m^2	
	F (4.8 x 129 + 1.89) 1.4 + 1.6 x 3.0	
	= (5.42 + 1.89) x 1.4 + 4.8	
	F = 15.03KN/m/m run	
	As prove = 393 mm^2/m	
	$\frac{\textit{long span}}{\textit{at edge}}$	
	Z = 134.17	

Reference	Calculation	Out put
	$A_{sy} = \dfrac{11.51\ x\ 10^6}{0.87\ x\ 410\ x\ 134.17} = 240.50mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	As span	
	$A_{sy} = \dfrac{8.7\ x\ 10^6}{0.87\ x\ 410\ x\ 134.17} = 181.79mm^2$	
	Provide Y10 @ $200^c/_c$	
	As prove = $393mm^2/m$	
	<u>Check deflection</u>	
	$Fs = 5/8 \text{ x } 410 \text{ x } \dfrac{224.83}{393}\ x\ \dfrac{1}{1} = 158.33$	
	$mf = 0.55 + \dfrac{477 - 158.33}{120\left(0.9 + \dfrac{12.02 \times 10^6}{1000 \times 144^2}\right)} = 2.34$	
	$Limiting\ span = 2.34\ x\ 26 = 60.84$	
	$Actual\ span = \dfrac{4200}{144} = 29.17$	
	Deflection ok	
	1^{st} FLIGHT	
	Span = 6 x 300 + 1800/2 = 2.43m	
	$M = \dfrac{fl^2}{8} = 0.125 \text{ x } 15.03 \text{ x } 2.43^2 = 11.09KNm$	
	Design	
	$K = \dfrac{11.09 \times 10^6}{1000 \times 144^2 \times 25} = 0.0147$	
	Z = 121.98mm	

Reference	Calculation	Out put
	$A_{SX} = \dfrac{11.09\times10^6}{0.87\times410\times121.98} = 254.88mm^2$	
	Provide Y10 @ 200$^c/_c$	
	As provide = 566mm^2/m	
	<u>Check deflection</u>	
	Fs = 5/8 x 410 x $\dfrac{254.88}{542}$ x $=144.50$	
	mf $= 0.55 + \dfrac{477-144.50}{120\left(0.9+\dfrac{11.09\times10^6}{1000\times124^2}\right)} = 2.25$	
	Simply supported = 20	
BS8110 Table3.10	Limiting span = 20 x 2.25 = 45.18	
	Actual span = $\dfrac{2.430}{124}$ = = 19.97	
	Deflection ok	
	Transverse distribution bars =	
Mosley and Bungey page 230 2nd edition	$\dfrac{0.2bh}{100} = \dfrac{0.24\times1000\times2000}{100} = 480$	
	Provide Y12@200	
	As provide = 566mm^2/m	
	FLIGHT 2	
	Span = 5 x 300 + (1612.5) = 3226.5mm	
	M = 0.125 x 12.63 x 3.2265^2	
	= 16.44KNm	
	Reinforcement	
	$K = \dfrac{16.44\times10^6}{1000\times124^2\times25} = 0.043$	

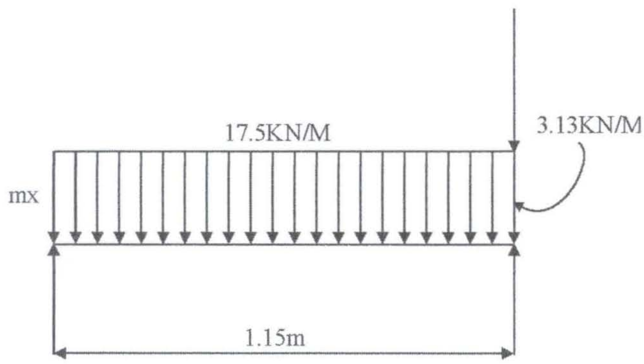
Reference	Calculation	Out put
	Z = 117.73	
	$A_{SX} = \dfrac{16.44 \times 10^6}{0.87 \times 410 \times 117.73} = 391.48mm^2$	
	Provide Y12 @ $200^c/_c$	
	As provide = $566mm^2/m$	
	Distribution bar	
	$\dfrac{0.24bh}{100} = \dfrac{0.24 \times 1000 \times 200}{100}$	
	FLIGHT 3	
	Reinforcement as in flight i.e	
	Provide Y12 @ $200^c/_c$ bottom and Y @$200^c/_c$	
	As distribution	
	Calculation	
	PROOF – GUTTER	
	LOADING	
	Self weight of slab = 0.15 x 1.15 x 24 = 4.14KN/m	
	Load from wlls = 1.6 x 0.15 x 24 = 5.76KN/m	
	0.6 x 0.15 x 24 = 2.16KN/m	
	Finishes say = 2.16KN/m	
	gk = 12.28KN/m	
	= 1.4gk + 1.6qk	
	= 1.4 x 12.28 + 1.6 x 0.25	
	= 17.19 + 0.4	
	= 17.5KN/m	
	Convert wall load to point load	
	5.76KN/m	

Reference	Calculation	Out put



To point load

$$5.76 \text{ x } \frac{1.15}{2} = 3.31KN$$

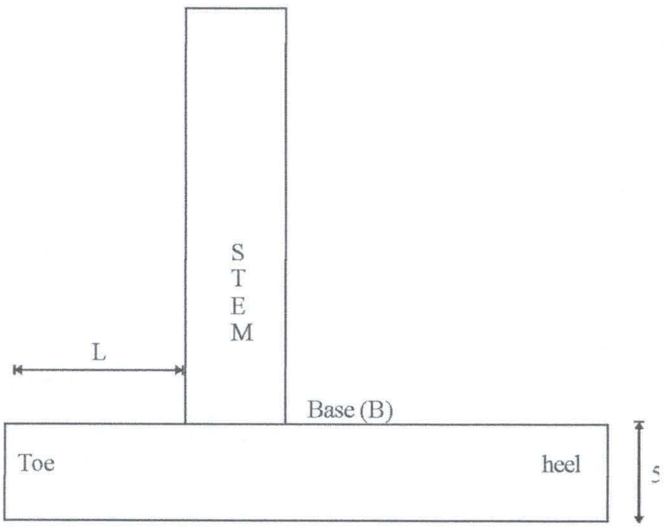


$$Mx = 3.31 \text{ x } 1.15 + 17.57 \text{ x } 1.15 \text{ x } \frac{1.15}{2} = 15.44 \text{KN/m}$$

Reinforcement

$$K = \frac{M}{Fcubd^2}$$

Reference	Calculation	Out put
	$D = h - c - \frac{1}{2} \times \theta$	
	$= 150 - 20 - \frac{1}{2}$ x $10 = 124mm$	
	$K = \dfrac{15.44 \times 10^6}{25 \times 1000 \times 124^2} = 0.0402$	
	$Z = d(0.5 + \sqrt{0.25 - \frac{K}{0.9}}$	
	$= 124(0.5 + \sqrt{0.25 - \frac{0.0402}{0.9}} = 118.18mm$	
	$A_S = \dfrac{15.44 \times 10^6}{0.87 \times 410 \times 118.18} = 366.27mm^2$	
	Provide Y12 @ $250^c/_c$	
	As provide = $452mm^2/m$	
	<u>Check deflection</u>	
	$Fs = 5/8\ fy\ \dfrac{A_s reqd}{A_s prov}$	
	$= 5/8$ x 410 x $\dfrac{366.27}{452}\ x\ \dfrac{1}{1} = 207.65N/mm$	
	$mf = 0.55 + \dfrac{477 - Fs}{120\left(0.9 + \dfrac{m}{bd^2}\right)} = 2.25$	
	$= 0.55 + \dfrac{477 - 207.65}{120\left(0.9 + \dfrac{15.44 \times 10^6}{1000 \times 124^2}\right)} = 1.73$	
	$\dfrac{Limiting\ span}{Depth} = 1.73 \times 7$	

Reference	Calculation	Out put
BS 8110 Table3.10	$\frac{Actual\ span}{Depth} = 12.1$ Deflection ok Distribution bar $\frac{0.13bh}{100} = \frac{0.13 \times 1000 \times 150}{100} = 195mm^2$ Provide Y10 @ $300^c/_c$ As provide = $262mm^2/m$ <u>Reinforcement for concrete walls:</u> Provide reinforcement as in the Slab i.e Y 12 @ 250mm and Y 10 @ 300mm as distribution bars. Retailing wall  **CALCULATION**	
Basic and applied soil mech. By copal Rajan	(i) Projection of toe from base (L) = 0.2B to 0.4B (ii) Base width (B) = 0.4H to 0.7H (iii) Thickness of base slab (b) = $\frac{H}{12}$ to $\frac{H}{8}$	

Reference	Calculation	Out put
	ANALYSIS AND DESIGN	

H = 3.450m

(i) Base width (B) = 0.7 x 3.45 = 2.415m

(ii) Projection of toe (L) = 0.4B

= 0.4 x 2.415 = 0.966m

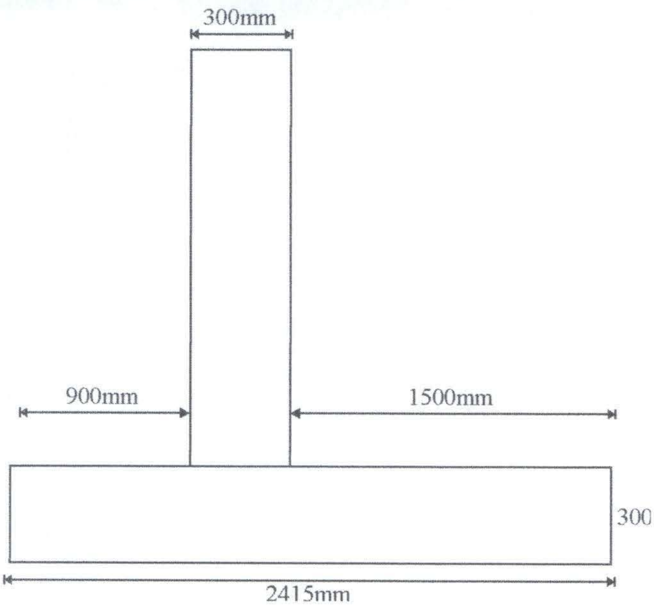
Say 0.9m

$\therefore$ Heel projection = 2.415 – 0.3 – 0.9 = 1.215

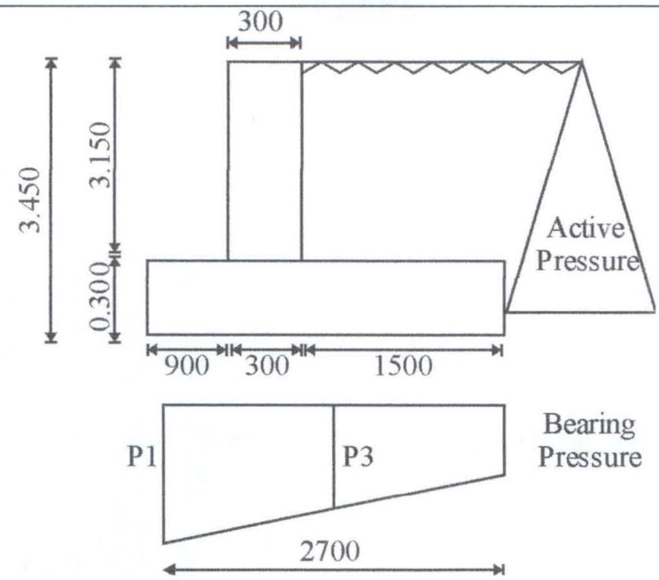
Say (1.500m)

(iii) Thickness of base slab = $\frac{H}{12}$

$$\frac{3.450}{12} = 288mm \ \ say \ \ 300mm$$



Reference	Calculation	Out put



For a granular materials

Saturated density = 18KN/m^3 (∞)

Allowable bearing pressure = 120KN/m^2

Value of coefficient of earth pressure Ko = 0.40

STABILITY

Horizontal force; The earth pressure P

P = Ko ∞ H

= 0.4 x 18 x 3.450

= 24.84KN/m^2

Therefore horizontal force on 1m length of wall is

Hk = 0.5pH

= 0.5 x 24.84 x 3.45 = 42.85KN

Vertical loads

Wall = ½ (0.3 + 0.3) x 3.45 x 24 = 24.84KN

Base = 0.3 x 2.7 x 24 = 19.44

Earth = 1.5 x 3.45 x 18 = 93.15

Total = 137. 43KN

Reference	Calculation	Out put
	For stability calculation a partial factor of safety of 1.6 is applied while 1.4 is applied for strength Calculations.	
	(i) <u>SLIDING</u>	
	∞ (1.0Gk + 1.0Vk)	
	∞ is taken as 0.45	
	Frictional resisting force = 0.45 x 1.0 x 137.43	
	= 61.84KN	
	Sliding force = $1.6H_K$	
	= 1.6 x 42.85 = 68.56KN	
	Since sliding force is greater than frictional resisting force therefore there will be need to provide passive earth pressure to act against heel beam is given by	
	$H_p = \infty_f \times 0.5 \times K_p \infty h^2$	
	H = depth of heel beam	
	∞f = 1.0 for granular	
	$K_p = \tan^2 (45 - {}^{\Theta}/_{2})$	
	K_p = is taken as 3.0 for granular materials	
	H = 500mm	
	$Hp = 1.0 \times 0.5 \times 3.0 \times 18 \times 0.5^2$	
	= 6.75 KN	
	Therefore total resisting force = frictional resisting force + passive pressure	
	= 61.84 + 6.75 = 68.58KN	
	Greater than sliding force	

Reference	Calculation	Out put
	Overturning: Taking moment above B at edge of toe	
	Overturning moment = $\infty\ \delta v\ H_k\ {}^{H}/_{3}$	
	δv_1 = factor of safety = 1.6	
	= 1.6 x 42.85 x 3.45/3	
	= 78.84KNm	
	Restraining moment = 1.0 (wall load x 1.0 + base)	
	Load x $\frac{2.7}{2}$ + earth load $\left(0.9+0.3+\frac{1.5}{2}\right)$	
	= 1.0 (24.84 x 1.0 + 19.44 x 1.35 + 93.15 x1.95)	
	Restraining M = 232.72KNm	
	BEARING PRESSURE	
	Bearing pressure = p $\frac{N}{B} \pm \frac{6m}{B^2}$	
	Moment about base centre line	
	$=H_k\ {}^{x}/_{3}$ wall load $({}^{B}/_{2} - 1.0)$ + earth $\left(\frac{B}{2} - 0.9 + 3 + \frac{1.5}{2}\right)$	
	$= 42.84 \times \frac{3.45}{2} + 24.84 \frac{(2..7-1.0)}{2} + 93.15\left(\frac{2.7-1.95}{2}\right)$	
	= 2.07KNm	
	P1 $\frac{N}{B} + \frac{6m}{B^2} = \frac{137.43}{2.7} + \frac{6 \times 2.07}{2.7^2} = 52.70\text{KN/m}^2$	
	Wall:	
	Horizontal force = $\delta v_1\ 0.5\ ka\ \infty\ H^2$	
	= 1,4 x 0.5 x 0.45 x 18 x 3.45^2	
	= 67.49KN	
	Maximum Moment $= 67.49\left(\frac{1}{2} \times 0.3 + \frac{3.45}{3}\right)$	

Reference	Calculation	Out put
	$= 87.74KNm$	
	Reinforcement (wall)	
	$D = h - c \dfrac{1\theta}{2}$	
	$= 300 - 20 - \dfrac{16}{2} = 272mm$	
	$K = \dfrac{87.74 \times 10^6}{1000 \times 272^2 \times 25} = 0.047$	
	$Z = 272(0.5 + \sqrt{0.25 - \dfrac{0.047}{0.9}} = 256.73mm$	
	$A_S = \dfrac{87.74 \times 10^6}{0.87 \times 410 \times 256.73} = 958.11 mm^2$	
	Provide Y16 @ $200^c/_c$	
	A_s prov = 1010mm	
	BASE REINFORCEMENT	
	Factors of safety:	
	$\delta v_1 = 1.4$; $\delta v_2 - \delta v_3 = 0$	
	Moment $M = \delta v_1 H_k y + \delta v2 \text{ x } G_k ({}^B/_2 - X) - \delta v_3 Vk ({}^B/_2 - q)$	
	$q_0 = \dfrac{B}{2} - \dfrac{\text{heel lenght}}{2} + \text{toe} + \text{width of all}$	
	$M = 1.4 \text{ x } 49.28 + 1.0 \text{ x } 8.69 - 1.0$	
	$= 68.99 + 8.69 - 55.89$	
	$= 21.79KNm$	
	$N = \delta v_2$ (wall load + base load) $+ \delta v_3 \text{ x } V_k$	
	$= 1.0 (24.84 + 19.44) + 1.0 \text{ x } 93.15$	

Reference	Calculation	Out put
	$= 137.4KN$	
	Pressure $P_1 = \frac{137.4}{3.4} + \frac{6 \times 21.79}{3.4^2}$	
	$= 40.41 + 11.31 = 51.72KN/m^2$	
	$P_2 = 40.41 - 11.31 = 29.1KN/m^2$	
	$P_3 = 29.1 + (51.72 - 29.1)\ 1.5 = 28.73KN/m^2$	
	Heel and Toe Reinforcement	
	Moment = $1.0 \times 19.44\ (1.05) + 1.0 \times 93.15\ (1.05)$	
	$- 29.1 \times 1.5 \times 1.05 - (28.73 - 29.1) \times \frac{1.5}{2} \times 0.45$	
	$M = 20.41 + 97.81 - 45.83 + 0.12$	
	Reinforcement	
	$A_S = \frac{72.27 \times 10^6}{0.87 \times 410 \times 256.73} = 849.35mm^2$	
	Provide Y16 @ $200^c/_c$	
	A_s prov = $1010mm^2/_m$	
	Toe reinforcement:	
	Provide reinforcement as in heel	
	i.e. Y16 @ $150^c/_c$	
	Distribution bar	
	$\frac{0.13 \times 1000 \times 300}{100} = 390mm^2/_m$	
	Provide Y12 @ $200^c/_c$	
	A_s provide = $566mm^2/_m$	

Reference	Calculation	Out put

CHAPTER FOUR

ROOF BEAM

Spacing of truss = 1.5m

Corrugated alluminium roofing shhet + norma; laps

= 2.44kg/m^2

Timber is mahogany ; density = 627kgm^2

Timber is mahogany ; density = 627kg/m^3

Purlin: 50mm x 75mm @ 900mm spacing

Weight of aluminium roofing sheet =

$$\frac{2.44 \times 9.81}{1000} = 0.024\text{KN/m}$$

Weight of purlin $\frac{672 \times 9.81}{1000} \times 0.05 \times 0.075$

= 0.025KN/m

Total dead load = 0.049

Imposed load

Roof without access except for maintenance =

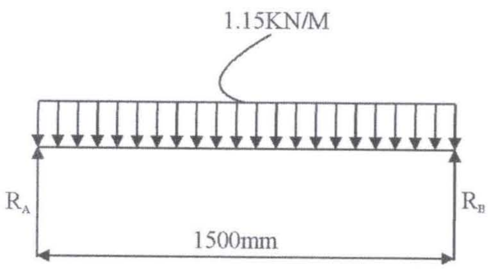
0.75KN/m^2

Total imposed load = 0.75 x 0.9 = 0.675KN/m

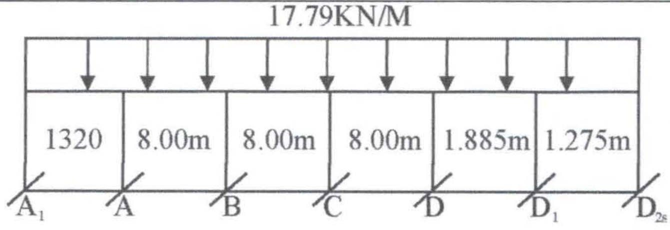
Design load = 1.4gk + 1.6qk

= 1.4 x 0.049 + 1.6 x 0.675

= 0.0686 + 1.08 = 1.148KN/m $\underline{w}$1.15KN/m



Reference	Calculation	Out put
	$R_A = R_B = \dfrac{WL}{2} = \dfrac{1.15 \times 1.5}{2} = 0.8625KN/m$	
	For external node, $R_A = {}^{p}/_{2} = 0.8625$	
	i.e p/2 = 0.8625	
	for internal nodes p = 2 x 0.8625	
	= 1.725KN	
	Reaction from roof truss = 6½ p	
	= 6.5 x 1.725 = 11.213KN	
	No of truss on the roof beam 2 – 2	
	Lenght of the beam 28.48	
	Spacing = 1.5m	
	No of truss = 18.97	
	Uniform load on the beam =	
	$\dfrac{18.97 \times 11.213}{28.48} = 7.47KN/m$	
	Beam size = 0.25 x 0.600	
	Self wt of beam = 0.225 x 0.6 x 24	
	= 3.22KN/m	
	From roof = 7.47KN/m	
	Finishes (say) = 0.28KN/m	
	Total dead load gk = 10.99KN/m	
	Imposed load = 1.5KN/m	
	Max design load = 1.4gk + 1.6gk	
	n = 1.4 x 10.99 + 1.6 x 1.5	
	n = 17.79KN/m	
	roof beam 1 – 1	

Reference	Calculation	Out put
		
Table 3.0 of Bs 8110	Bending Movement (using simplified method) at 1st	
	interior support M_A = -0.11FL	
	Where f = nl	
	= 17.79 x 1.32 = 23.48KN	
	M_A = -0.11 x 23.48 x 1.32 = 341KN/m	
	Interior support =	
	$M_B = M_C = M_D$	
	M_B = -0.11 x FL	
	F = 17.79 x 8.0 = 142.32KN	
	M_B = -0.11 x 142.32 x 8.0 = 125.24KN/m	
	At mid span	
	M_{A1-A} = 0.09FL	
	0.09 x 23.48 x 1.32 = 2.79KNm	
	M_{A-B} = 0.07 x 142.32 x 80 = 79.70KNm	
	M_{B-C} = 0.09 x 142.32 x 80 = 102.47KNm	
	Main reinforcement	
	Overall depth h = 600mm	
	Web width (bw) = 22.5mm	
	Effective depth, $d = 600 - 25 - 10 - \frac{16}{2} = 557mm$	
	Fcu = 25N/mm_2	
	Fy = 410N/mm^2	

Reference	Calculation	Out put
	At support B	
	$M_B = 125.24KNm$	
	$K = \frac{m}{Fcubd^2} \leq 0.156$	
	$= \frac{125.24 \times 10^6}{225 \times 557^2 \times 25} = 0.072$	
	Compressed bar not required	
	$Z = 557(0.5 + \sqrt{0.25 - \frac{0.072}{0.9}}$ 508.16mm	
	$A_S = \frac{125.24 \times 10^6}{0.87 \times 410 \times 508.16} = 690.94mm^2$	
	Provide 4Y16 bottom	
	A_s prov = 804mm^2/m	
	At span	
	M = 102.47	
	$\frac{M}{bd^2fcu} = \frac{102.47 \text{ x } 10^6}{225 \text{ x } 55^2\ 25} = 0.059$	
	Z = 518.07mm	
	$A_S = \frac{102.47 \times 10^6}{0.87 \times 4 \times 10518.07} = 554.50mm^2$	
	Provide 4Y16 bottom	
	A_s prov = 804mm^2/m	
	Provide 2Y16 Top (As prov = 402mm^2)	
	SHARE FORCES	
	V_{A1} = 0.45f = 0.45 x 23 .48 = 10.57KN	
	V_A to V_D = 0.6f = 0.6 x 142.32 = 85.39KN	
	V_{D1} = 0.6f = 0.6 x 33 .53 = 20.12KN	

Reference	Calculation	Out put
	$V_{D2} = 0.45f$ if $= 17.79 \times 1.275 = 22.68KN$	
	$V_{D2} = 0.45 \times 22.68 = 10.20KN$	
	Check deflection	
	$Fs = 5/8fy \dfrac{\text{Area required}}{\text{Area prov}} \times \dfrac{1}{1}$	
	$= 5/8 \times 410 \times \dfrac{554}{804} \times \dfrac{1}{1} = 176.57$	
	$Mf = 0.55 + \dfrac{477 - fs}{120\left(0.9\, x\, \dfrac{M}{Bd^2}\right)}$	
	$\dfrac{M}{bd^2} = \dfrac{102.47\, x\, 10^6}{1000\, x\, 557^2} = 3.03$	
	$Mf = 0.55\, x\, \dfrac{477 - 176.57}{121.23}$	
	$\dfrac{Limiting\ span}{Depth} = 26 \times 3.03 = 78.73$	
	$\dfrac{Actual\ span}{Depth} = \dfrac{8000}{557} = 14.36$	
	Deflection ok	
	Check max shear stress	
	$Vs = 0.6f - wu \times \dfrac{\text{support width}}{2}$	
	$= 0.6 \times 85.39 - 17.79 \times \dfrac{0.225}{2}$	
	$Vs = 49.23KN$	
	$V = Vs = \dfrac{49.23\, x\, 10^3}{255.559} = 0.39 N/mm^2$	
	$0.39Nmm^2 < 0.8\sqrt{fcu} = 4N/mm^2$	
	End supports	

Reference	Calculation	Out put
	Shear at distance d from support face	
	$V_d = 0.45f - wu\ (d \times \dfrac{\text{support width}}{2})$	
	$= 0.45 \times 85.39 - 17.79\ (0.557 \times \dfrac{0.225}{2})$	
	$= 38.43 - 11.91$	
	$= 26.62KN.$	
	$V = \dfrac{26.52\ x\ 10^3}{255.557} = 0.211 N/mm^2$	
	Normal a links	
	$\dfrac{Asv}{Sv} = \dfrac{0.4b}{0.87 fyv} = \dfrac{0.4\ x\ 225}{0.87\ x\ 200} = 0.41$	
	Provide Y10 Links @ $300^c/_c$	
	Check shear	
	$V = 85.39KN$	
	$V = \dfrac{v}{bd} = \dfrac{85.39\ x\ 10^3}{225\ x\ 557} = 0.68$	
Table 3.9 BS8110	$\dfrac{100As}{bd} = \dfrac{100\ x\ 804}{225\ x\ 557} = 0.64$	
	$Vc = \dfrac{0.79\left(\dfrac{100As}{bd}\right)^{1/3}\left(\dfrac{400}{d}\right)^{1/4}}{\partial m} =$	
	$= \dfrac{0.79\left(\dfrac{100\ x\ 804}{225\ x\ 557}\right)^{1/3}\left(\dfrac{400}{557}\right)^{1/4}}{1.25}$	
	Where $\partial m = 125$	
	$V_c = 0.15 N/mm^2$	
	$\dfrac{ASV}{Sv} = \dfrac{b(V - V_c)}{0.87 fyv} = \dfrac{225(0.64 - 0.15)}{0.87 \times 250} = 0.51s$	

Reference	Calculation	Out put
	Provide Y10 @ $300^c/_c$	
	GROUND FLOOR BEAMS	
	BEAM ON GRID LINE 2 – 2	
	Loading	
	Beam size 300 x 750mm	
	Self weight of the beam = 5.4KN/m	
	Wall = 7.6KN/m	
	Finishes = 0.25KN/m	
	Total = 13.30KN/m	
	Load on spans	
	Span AB = BC = CD	
	Triangular loading from slab	
	$= \frac{1}{2} n\ lx \left[1 - \frac{1}{3 \times k^2} \right]$; k = ly/lx	
	$= \frac{1}{2} \times 11.14 \times 4 \left[1 - \frac{1}{3 \times 2^2} \right]$	
	= 20.42 x 2 sides	
	= 40.84KN/m	
	Load on span DD1	
	Trapezoidal loading = 1/3nlx	
	= 1/3 x 11.14 x 3.8	
	= 14.09KN/m	
	= 14.09 x 2 sides = 28.19KN/m	
	<u>Design load on spans</u>	
	<u>Span AB, BC and CD</u>	
	Imposed load = 4.4KN/m	

Reference	Calculation	Out put
	Total dead load don span AB, BC and CD	

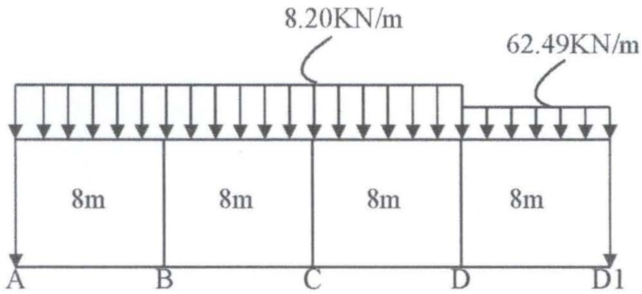
= 40.84 + 13.3 = 54.14KN/m

Design load = 14 x 54.14 + 4.4 = 80.20KN/m

Span DD1

Dead load = 28.19 + 13.3 = 41.49KN/m

Design load = 1.4 x 41.49 + 4.4 = 62.49KN/m



Analysis

$$FEMAB = FEMBC = FEMCD = \frac{WL^2}{12}$$

$$= \frac{80.026\ x\ 8^2}{12} = 428.05KN/m$$

$$FEMDD1 = \frac{62.49\ x\ 3.8^2}{12} = 75.19KN/m$$

Stiffness factor

$$KBC = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$$

$$KCB = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KCD = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KDC = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KDD1 = \frac{3EI}{L} = \frac{3EI}{3.8} = 0.79EI$$

Reference	Calculation	Out put
	Distribution factor	
	DFAB = $\dfrac{KAB}{KAB+KBC}=\dfrac{0.38EI}{(0.38+0.5)EI}=0.43$	
	DFBC = 1 – 0.43 = 0.57	
	DFCB= $\dfrac{KCB}{KCB+KCD}=\dfrac{0.5EI}{(0.5+0.5)EI}=0.5$	
	DFBC = 1 – 0.5 = 0.5	
	DFDC= $\dfrac{KDC}{KDC+KDD1}=\dfrac{0.5EI}{(0.5+0.79)EI}=0.39$	
	DFBC = 1 – 0.39 = 0.61	

A	B		C		D		D1
	0.43	0.57	0.5	0.5	0.39	0.61	
+428.05	-428.05	+428.05	-428.05	+428.05	-428.05	+75.19	-75.19
	-214.03	0	0	214.03	0	37.60	0
	-642.08	428.05	-428.05	642.08	-428.05	112.79	-75.19
	92.03	121.997	-107.02	-107.02	-122.95	192.31	
		-53.51	60.995		-53.51		
	23.01	30.50	-30.50	-30.50	20.87	20.87	
		-15.25	15.25		-15.25		
	6.56	8.69	-7.63	-7.63	5.95	9.30	
		-3.82	3.82		-3.82		
	1.64	2.18	-1.64	-1.64	1.50	2.33	
		-0.82	1.09		-0.82		
	0.35	0.47	0.55	0.55	0.32	0.50	
	-518.72	518.48	-494.78	494.78	-349.86	349.1	

Reference	Calculation	Out put
	Support moments	
	Support B = 518.72KNm	
	Support C = 494.78KNm	
	Support D = 349.86KNm	
	Span Moment	
	Span AB = $0.125WL^2 - \frac{1}{2} = (MB\ MC)$	
	$= 0.125 \times 80.26 \times 8^2\ \Phi - \frac{1}{2} = (518.72 + 494.78)$	
	= 315.92KNm	
	Span BC	
	$= 0.125 \times 80.26 \times 8^2 - \frac{1}{2} = (494.78)$	
	= 575.28KNm	
	Support CD	
	$= 0.125 \times 80.26 \times 8^2 - \frac{1}{2} = (494.78 + 349.56)$	
	= 400.35KNm	
	Support DD1	
	$= 0.125 \times 62.49 \times 8^2 - \frac{1}{2} = (349.86)$	
	= 62.14KNm	
	Reinforcement	
	Support B	
	$K = \frac{518.72 \times 10^6}{25 \times 300 \times 710^2} 0.0137$	
	Z = 577.00mm	

Reference	Calculation	Out put
	$A_s = \dfrac{518.72 \times 10^6}{087 \times 410 \times 577} = 252031.83mm^2$	
	Provide 3 + 3 Y25 (As prov = 2950 top)	
	Support C	
	$K = \dfrac{494.78 \text{ x } 10^6}{25 \text{ x } 300 \text{ x } 710^2} 0.131$	
	Z = 584.52mm	
	$A_s = \dfrac{494.78 \times 10^6}{087 \times 410 \times 584.52} = 2393.06mm^2$	
	Provide 3 + 2 Y25 (as provide = 2450mm^2/m top)	
	Support D	
	$K = \dfrac{394.86 \text{ x } 10^6}{25 \text{ x } 300 \text{ x } 710^2} 0.0925$	
	Z = 627.31mm	
	$A_s = \dfrac{349.86 \times 10^6}{087 \times 410 \times 627.31} = 1{,}563.54mm^2$	
	Provide 3 + 3 Y20 (as provide = 1870mm^2/m top)	
	Span reinforcement	
	Span AB	
	$bf = bw + \dfrac{0.7L}{10} = 300 + \dfrac{0.7 \, x \, 800}{10} \; 860mm$	
	$K = \dfrac{315.92 \text{ x } 10^6}{25 \text{ x } 860 \text{ x } 710^2} = 0.02914$	
	Z = 686.20mm	
	$A_s = \dfrac{315.92 \times 10^6}{087 \times 410 \times 686.20} = 1290.07mm^2$	

Reference	Calculation	Out put
	Proved 3Y25 (As prov = 1470mm^2/m bottom)	
	Provide 3Y16 top	
	<u>Span BC</u>	
	$K = \dfrac{575.28 \times 10^6}{25 \times 860 \times 710^2} = 0.0530$	
	Z = 665.29mm	
	$A_s = \dfrac{575.28 \times 10^6}{087 \times 410 \times 665.29} = 2424.18mm^2$	
	Proved 3 + 3Y25 (As prov = 2950mm^2/m bottom)	
	Provide 3Y16 top	
	<u>Span CD</u>	
	$K = \dfrac{400.38 \times 10^6}{25 \times 860 \times 710^2} 0.03694$	
	Z = 679.51mm	
	$A_s = \dfrac{400.38 \times 10^6}{087 \times 410 \times 679.51} = 1651.86mm^2$	
	Proved 2 + 2Y25 (As prov = 1960mm^2/m bottom)	
	Provide 3Y16 top	
	<u>Span DD1</u>	
	$bf = 300 + \dfrac{0.7\, x\, 3800}{10}$ 566mm	
	$K = \dfrac{62.14 \times 10^6}{25 \times 566 \times 710^2} 0.0087116$	
	Z = 703.04mm	
	$A_s = \dfrac{62.14 \times 10^6}{087 \times 410 \times 703.04} = 247.79mm^2$	

Reference	Calculation	Out put
	Proved 3Y16 (As prov = $603mm^2/m$ bottom)	
	Provide 3Y16 top	
	<u>Check deflection</u>	
	$= 5/8 \ \ x\ 410\ x \dfrac{2424.18}{2950} x\ \dfrac{1}{1} = 209.86N/mm$	
	$Mf = 0.55 + \dfrac{477 - 209.86}{120\left(0.9 + \dfrac{575 \text{ x } 10^6}{860 \text{ x } 710^2}\right)} = 1.55$	
	$\dfrac{Limiting\ span}{depth} = 1.55\ x\ 26 = 40.29$	
	$\dfrac{Actual\ span}{depth} = \dfrac{8000}{710} = 11.27$	
	Shear VAB = $\dfrac{WL}{2} + \dfrac{0 - MB3}{L}$	
	$= \dfrac{80.20 \text{ x } 8}{2} + \dfrac{0 - 518.72}{8} = 255.96KN$	
	VBA= $\dfrac{80.2 \text{ x } 8}{2} + \dfrac{518.72}{8} = 385.64KN$	
	VBC $\dfrac{80.2 \text{ x } 8}{2} + \dfrac{518.72 - 494.78}{8} = 323.79KN$	
	VCB= $\dfrac{80 \text{ x } 8}{2} + \dfrac{494.78 - 518.72}{8} = 317.81KN$	
	VCD = $\dfrac{80.2 \text{ x } 8}{2} + \dfrac{494.78 - 349.86}{8} = 338.92KN$	
	VDC = $\dfrac{80.2\ x\ 8}{2} + \dfrac{349.86 - 494.86}{8} = 302.692KN$	
	VDD1 = $\dfrac{62.49 \text{ x } 3.8}{2} + \dfrac{349.86 - 0}{8} = 210.80KN$	
	VD1D = $\dfrac{62.49 \text{ x } 3.8}{2} + \dfrac{0 - 349.86}{3.8} = 26.66KN$	
	Check shearer	

Reference	Calculation	Out put
	V = 338.92KN	
	$V = \frac{338.92 \ x \ 10^3}{300 \ x \ 710} = 1.591$	
	$\frac{100As}{bd} = \frac{100 \ x \ 2950}{300 \ x \ 710} = 1.385$	
	$Vc = \frac{0.79 \ (1.385)^{1/3} \ (\frac{400}{710})^{1/4}}{1.25} = \mathbf{0.610}$	
	$\frac{b(v - vc)}{0.87 fyv} \frac{300(1.591 - 0610)}{0.87 \ x \ 250} = 1.453$	
	Provide Y12 @ 150	
	Check masx shear	
	$Vs = 0.6f - \frac{wu}{2}$ x support width	
	$= 0.6 \ x \ 338.92 - 80.2 \ x \ \frac{0.3}{2} = 191.32KN$	
	$V = \frac{vs}{bd} = \frac{191.32 \ x \ 10^3}{300 \ x \ 710} = 0.898$	
	Shear at distance from support face	
	$Vd = 0.45F - wu \left[d + \frac{0.3}{2} \right]$	
	$= 0.45 \ x \ 338.92 - 80.2 \ (0.710 + 0.15)$	
	= 83.54KN	
	$V = \frac{83.5 \ x \ 10^3}{300 \ x \ 710} = 0.392$	
	Normal links	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.87 fyv} = \frac{0.4 \ x \ 300}{0.87 \ x \ 250} = 0.552$	
	Provide Y10 @ 250$^c/_c$	

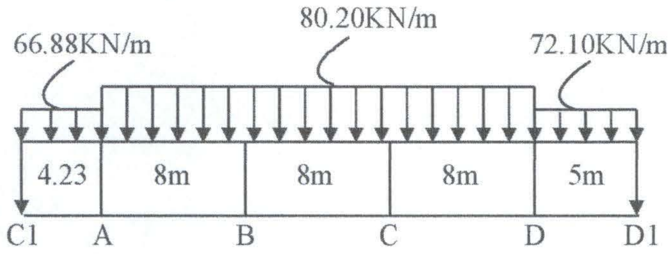
Reference	Calculation	Out put
	BEAM ON GRID LINE 3 – 3	
	Beam size 300 x 750	
	Self weight = 5.4KN/m	
	Wall = 7.65KN/m	
	Finishes = 0.25KN/m	
	Total = 13.30KN/m	
	Load on spans	
	Span C_1A	
	$= ½\, n\, lx \left[1 - \frac{1}{3\,x\,2k^2}\right]$; K = Ly/Lx	
	$= \frac{1}{2}\,x\,11.14\,x\,4\,x\left[1 - \frac{1}{3\,x\,1.06^2}\right];$	
	= 15.57KN/m x 2 sides = 31.33KN/m	
	$= \frac{1}{2}\,x\,11.14\,x\,4\,x\left[1 - \frac{1}{3\,x\,2^2}\right];$	
	= 20.42KN/m x 2 sides = 40.84KN/m	
	Load on span DD1	
	$= \frac{1}{2}\,x\,11.14\,x\,4\,x\left[1 - \frac{1}{3\,x\,1.25^2}\right];$	
	= 17.53KN/m x 2 sides = 35.06 KN/m	
	Design of spans	
	Total dead load on span C_1A	
	gk = 13.3 x 31.33 = 44.63KN/m	
	Design load = 1.4 x 44.63 x 4.4 = 66.88KN/m	
	Design load on span AB, BC and CD	
	gk = 13.3 x 40.84 = 54.14KN/m	

Reference	Calculation	Out put

design load = 1.4 x 54.14 x 4.4 = 80.20KN/m

SPAN DD1

Design Load = 13.3 x 35.06 x 1.4 x 4 = 72.10KN/m



Analysis

$$FEMC1A = \frac{WL^2}{12} = \frac{66.88\ x\ 4.23^2}{12} = 99.72KNm$$

FEMAB = FEMBC = FEMCD =

$$\frac{80.20\ x\ 8^2}{12} = 428.05KNm$$

$$FEMDD1 = \frac{72.10\ x\ 5^2}{12} = 150.21KNm$$

Stiffness factors

$$KCIA = \frac{3EI}{L} = \frac{3EI}{4.23} = 0.71EI$$

$$KAB = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KBA = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KBC = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KCB = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$KCD = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

Reference	Calculation	Out put

$$\text{KDC} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$\text{KDD1} = \frac{3EI}{L} = \frac{3EI}{5} = 0.6EI$$

Distribution factors

$$\text{DFC1A} = \frac{KC1A}{KC1A + KAB} = \frac{0.71EI}{(0.71 + 0.5)EI} = 0.59$$

$$\text{DFAB} = 1 - 0.59 = 0.41$$

$$\text{DFBA} = \frac{KBA}{KBA + KBC} = \frac{0.5EI}{(0.5 + 0.5)EI} = 0.5$$

$$\text{DFBC} = 1 - 0.5 = \text{-}0.5$$

$$\text{DFBA} = \frac{KCB}{KCB + KCD} = \frac{0.5}{0.5 + 0.5} = 0.5$$

$$\text{DFCD} = 1 - 0.5 = 0.5$$

$$\text{DFDC} = \frac{KDC}{KDC + KDD1} = \frac{0.5EI}{(0.5 + 0.6)EI} = 0.46$$

$$\text{DFCD} = 1 - 0.46 = 0.54$$

0	0.59	0.41	0.5	0.5	0.5	0.5	0.46	0.54	0
+99.72	-99.72	428.5	-428.05	428.05	-428.05	428.05	-428.05	150.21	-150.21
	-49.86	0	0	214.03	0	214.03	0	72.11	
	-149.58	428.05	-428.05	642.08	-428.05	642.08	-428.05	222.32	
	-164.30	-114.17	-107.02	-107.02	-107.02	-107.02	+94.64	111.10	
		-53.51	-57.09		-53.51		53.51		
	31.57	21.94	28.55	28.5	26.76	26.76	24.62	28.90	
		14.28	10.97		14.28		13.38		
	-8.43	-5.86	-5.49	-5.49	-7.14	-7.14	-6.16	-7.23	
		-2.75	-2.93		-2.75		-3.57		
	1.62	1.13	1.47	1.47	1.38	1.38	1.64	1.93	
		0.74	0.57		0.74		0.69		
	0.44	0.30	0.29	0.29	0.37	1.38	0.32	0.37	
	-288.68	289.15	-558.73	559	-554.94	556.43	-356.00	356.40	

Reference	Calculation	Out put
	MOMENT	
	Support moment	
	Support A = 288.68KNm	
	Support B = 559.88KNm	
	Support C = 556.43KNm	
	Support D = 356.40KNm	
	Span momments	
	Span C1A = $0.125WL^2 - \frac{1}{2}(MA + MB)$	
	$0.125 \text{ x } 66.88 \text{ x } 4.23^2 - \frac{1}{2}(288.68 + 559.58)$	
	= 274.69KNm	
	Span AB = $0.125 \text{ x } 80.20 \text{ x } 8^2 - \frac{1}{2}(559.88)$	
	= 361.66KNm	
	Span BC = $0.125 \text{ x } 80.2 \text{ x } 8^2 - \frac{1}{2}(559.88 + 556.43)$	
	= 83.44KNm	
	Span CD = $0.125 \text{ x } 80.2 \text{ x } 8^2 - \frac{1}{2}(556.43)$	
	= 363.38KNm	
	Span DD1 = $0.125 \text{ x } 72.10 \text{ x } 5^2 - \frac{1}{2}(556.43 + 356.40)$	
	= 231.11KNm	
	Reinforcemnt	
	Supports A	
	$K = \frac{288.68 \times 10^6}{25 \times 300 \times 710^2} = 0.0764$	

Reference	Calculation	Out put
	$Z = 643.50mm$	
	As = $\dfrac{288.68 \times 10^6}{0.87 \times 410 \times 643.5} = 1257.67mm^2$	
	Provide 3Y25 (As Prov = 1470mm^2/m Top)	
	SUPPORT B	
	K = $\dfrac{559.88 \times 10^6}{25 \times 300 \times 710^2} = 0.149$	
	$Z = 561.32mm$	
	As = $\dfrac{559.88 \times 10^6}{0.87 \times 410 \times 561.32} = 2796.28mm^2$	
	Provide 3Y25 (As Prov = 2950mm^2/m Top)	
	SUPPORT C	
	K = $\dfrac{556.43 \times 10^6}{25 \times 300 \times 710^2} = 1.1472$	
	$Z = 563.78mm$	
	As = $\dfrac{556.43 \times 10^6}{0.87 \times 410 \times 563.78} = 2766.92mm^2$	
	Provide 6 Y25 (As Prov = 290mm^2/m Top)	
	SUPPORT D	
	K = $\dfrac{356.40 \times 10^6}{25 \times 300 \times 710^2} = 0.0943$	
	$Z = 625.64mm$	
	As = $\dfrac{356.40 \times 10^6}{0.87 \times 410 \times 625.64} = 1597.02mm^2$	
	Provide 6Y20 (As Prov = 1890mm^2/m Top)	

Reference	Calculation	Out put
	SPAN REINFORCEMENT **SPAN C1A** $bf = 300 + \frac{0.7 \text{ x } 4230}{10} \quad 596.10\text{mm}$ $K = \frac{274.69 \text{ x } 10^6}{25 \text{ x } 596.1 \text{ x } 710^2} 0.03656$ $Z = 679.89\text{mm}$ $A_s = \frac{274.69 \times 69 \times 10^6}{087 \times 410 \times 679.89} = 1132.66mm^2$ Provide 3 Y 20 (As Prov = 943mm^2/m Bottom) 2Y16 Top **SPAN AB** $bf = 300 + \frac{0.7 \text{ x } 8000}{10} \quad 860\text{mm}$ $K = \frac{361.66 \text{ x } 10^6}{25 \text{ x } 860 \text{ x } 710^2} 0.0333$ $Z = 682.60\text{mm}$ $A_s = \frac{361.66 \times 10^6}{087 \times 410 \times 682.60} = 1485.30mm^2$ Provide 4 Y 25 (As Prov = 1960mm^2/m Bottom) Provide 3 Y16 Top **SPAN BC** $K = \frac{83.44 \text{ x } 10^6}{25 \text{ x } 860 \text{ x } 710^2} 0.0076987$ $Z = 703.84\text{mm}$	

Reference	Calculation	Out put
	$A_s = \dfrac{83.44 \times 10^6}{087 \times 410 \times 703.84} = 332.35mm^2$	
	Provide 2 Y 20 (As Prov = 628mm^2/m Bottom)	
	PROV 3 Y 16 Top	
	SPAN CD	
	$K = \dfrac{363.38 \text{ x } 10^6}{25 \text{ x } 860 \text{ x } 710^2} 0.0335$	
	Z = 682.45mm	
	Provide 4 Y 25 (As Prov = 1492.75mm^2/m Bottom)	
	PROV 3 Y 16 Top	
	SPAN DD1	
	$bf = 300 + \dfrac{0.7 \text{ x } 5000}{10} \quad 650mm$	
	$K = \dfrac{231.11 \text{ x } 10^6}{25 \text{ x } 650 \text{ x } 710^2} 0.02821$	
	Z = 686.96mm	
	$A_s = \dfrac{231.11 \times 10^6}{087 \times 410 \times 686.98} = 939.06mm^2$	
	Provide 3 Y 20 (As Prov = 943mm^2/m)	
	PROV 3Y16 Top	
	Check deflection	
	$Fs = 5/8 \quad \text{x } 410 \text{ x } \dfrac{1492.75}{1960} \text{ x } \dfrac{1}{1} = 195.16Nlmm$	
	$Mf = 0.55 + \dfrac{477 - 195.16}{120\left(0.9 + \dfrac{363.38 \text{ x } 10^6}{682.45 \text{ x } 710^2}\right)}$	
	$\dfrac{Limiting\ span}{depth} = 1.75\ x\ 26 = 45.51$	

Reference	Calculation	Out put
	$\frac{Actual\ span}{depth} = \frac{8000}{720} = 11.27$	
	SHEAR	
	$VC1A = \frac{WL}{2} = \frac{0 - M_A}{L}$	
	$= \frac{66.88 \text{ x } 4.23}{2} + \frac{0 - 288.68}{4.23} = 73.20KN$	
	$VAC1 = \frac{66.88 \text{ x } 4.23}{2} + \frac{288.68 - 0}{4.23} = 209.7KN$	
	$VAB = \frac{80.20 \text{ x } 8}{2} + \frac{288.68 - 559.88}{8} = 286.9KN$	
	$VBA = \frac{80.20 \text{ x } 8}{2} + \frac{559.88 - 288.68}{8} = 354.7KN$	
	$VBC = \frac{80.20 \text{ x } 8}{2} + \frac{556.88 - 556.43}{8} = 320.37KN$	
	$VCB = \frac{80.20\ x\ 8}{2} + \frac{556.43 - 559.88}{8} = 321.23KN$	
	$VCD = \frac{80.20 \text{ x } 8}{2} + \frac{556.43 - 356.40}{8} = 345.8KN$	
	$VDC = \frac{80.20 \text{ x } 8}{2} + \frac{356.40 - 556.40}{8} = 295.8KN$	
	$VDD1 = \frac{72.10 \text{ x } 5}{2} + \frac{356.40 - 40}{5} = 142.56KN$	
	$VD1D = \frac{72.10 \text{ x } 5}{2} + \frac{0 - 356.40}{5} = 108.97KN$	
	CHECH SHEAR:	
	$V = 345.8KN$	
	$V = \frac{345.8 \text{ x } 10^3}{300 \text{ x } 710} = 1.623$	
	$\frac{100AS}{bd} = \frac{100\ x\ 2950}{3000\ x\ 710} = 1.384$	

Reference	Calculation	Out put
	$Vc = \dfrac{0.79\,(1.384)^{1/3}\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.610$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87fyv} = \dfrac{0.4\,x\,300}{0.87\,x\,250}0.552$	
	Provide Y10 @ $250^{c}/_{c}$	
	Check Max Shear	
	$Vs = 0.6f - wu \text{ x } \dfrac{\text{support}}{2}$	
	$= 0.6 \text{ x } 345.8 - 80.20 \text{ x } \dfrac{0.3}{2}$	
	Vs = 195.45KN	
	$V = \dfrac{Vs}{bd} = \dfrac{195.45 \text{ x } 10^{3}}{300 \text{ x } 710} = 0.918$	
	End support	
	Shear at distance d from support face	
	$Vd = 0.45f - wu \left(d\,x\,\dfrac{300}{2}\right)$	
	= 0.45 x 345.8 – 80.20 (0.710 x 0.15)	
	= 86.64KN	
	$V = \dfrac{86.64 \text{ x } 10^{3}}{300 \text{ x } 710} = 0.407$	
	Normal Links	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87fyv} = \dfrac{0.4\,x\,300}{0.87\,x\,250}0.552$	
	Provide Y10 @ $250^{c}/_{c}$	

Reference	Calculation	Out put
	$FEMCD = \frac{WL^2}{12} + \frac{Pab^2}{L^2} = \frac{80.2\ x\ 8^2}{12} + \frac{224.30\ x\ 4\ x\ 4^2}{12}$	
	= 652.03KNm	
	$FEMDD1 = \frac{WL^2}{12} = \frac{58.29\ x\ 5.3^2}{12} = 136.4KNm$	
	<u>Stiffness factor</u>	
	$KAB = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$	
	$KBC1 = \frac{4EI}{3.770} = 1.38EI$	
	$KCIB = 1.06EI$	
	$KC1C = \frac{4EI}{L} = \frac{4EI}{4.23} = 0.95EI$	
	$KCC1 = \frac{4EI}{L} = \frac{4EI}{8} = 0.95EI$	
	$KCD = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	
	$KDC = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	
	$KDD1 = \frac{3EI}{L} = \frac{3EI}{8} = 0.57EI$	
	Distribution factors	
	$DFAB = \frac{KAB}{KAB + KBC1} = \frac{0.38}{(0.38 + 1.06)} = 0.26$	
	$DFBC1 = 1 - 0.26 = 0.74$	
	$DFC1B = \frac{KC1B}{KC1B + KC1C} = \frac{1.06}{1.06 + 0.95} = 0.53$	
	DFC1C $1 - 0.53\ 0.47$	

Reference	Calculation	Out put
	$$\text{DFCC1} = \frac{KCC1}{KCC1 + KCD} = \frac{0.95}{0.95 + 0.5} = 0.66$$	
	DFCD $1 - 0.66$ 0.34	
	$$\text{DFDC} = \frac{KDC}{KDC + KDD1} = \frac{0.5}{0.5 + 0.57} = 0.47$$	
	DFCD $1 - 0.47 = 0.53$	

A	B		C1		C		D		D1
	0.26	0.74	0.53	0.47	0.66	0.34	0.47	0.53	
+4666.86	-466.86	+94.99	-94.99	119.58	-199.58	652.03	-652.03	136.45	-136.45
-466.86	-233.83	0	0	59.79	0	326.02	0	68.23	+136.45
0	-7000.29	94.99	-94.99	179.37	-199.58	978.05	-652.03	204.68	
	157.38	447.92	-44.72	-39.66	-566.59	-291.88	210.25	237.10	
		-22.36	223.96		-19.83		-14594		
	5.81	16.55	-118.70	-105.26	13.88	6.74	68.59	77.35	
		-59.35	8.22		-52.63		3.37		
	15.43	43.92	-4.36	-3.86	34.74	17.89	-1.58	-1.79	
		-2.18	21.96		-1.93		8.95		
	0.57	1.61	-11.64	-10.32	1.27	0.66	-4.21	-4.74	
		-5.82	0.81		-5.15		0.33		
	1.51	4.31	-0.43	-0.38	1.27	1.75	0.16	0.14	
		-0.22	2.16		-0.19		0.86		
	0.06	0.16	-1.14	-1.02	0.13	0.06	-0.40	-0.46	
		-0.57	0.08		-0.51		0.03		
	0.15	0.42	-0.04	0.04	0.34	0.17	-0.01	-0.02	
	-519.38	519.38	-18.83	18.91	-712.65	712.44	-51163	511.29	

Reference	Calculation	Out put
	BEAM ON GRID LINE 4 – 4 Beam size 300 x 750 Self weight = 0.3 x 0.75 x 24KN/m = 5.4KN/m Wall = 7.65KN/m Finishes = <u>0.25KN/m</u> Total = <u>13.3KN/m</u> Load on spans Span AB1 $= \frac{1}{2} x\, 11.14\, x\, 14\, x \left[1 - \frac{1}{3\, x\, 2^2}\right] = 20.42 KNm$ Span BC1 $= \frac{1}{2} x\, 11.14\, x\, 14\, x \left[1 - \frac{1}{3\, x\, 2^2}\right] = 20.42\, x\, 2\, Span$ = 40.84KN/m Span C1C $= \frac{1}{2} x\, 11.14\, x\, 14\, x \left[1 - \frac{1}{3\, x\, 2^2}\right] = 20.42\, x\, 2$ = 40.84KN/m Span CD = as in span C1C = 40.84KN/m SPAN DD1 $= \frac{1}{2} x\, 11.14\, x\, 4\, x \left[1 - \frac{1}{3\, x\, 33^3}\right] = 12.60\, x\, 2\, Swen$ = 25.19KN/m Design load on spans Span AB, GC, and Span CD	

Reference	Calculation	Out put
	Supports momments.	
	MB = 519.38KNm	
	MC1 = 18.81	
	MC = 712.65KNm	
	MD = 511.63KNm	
	SPANS MOMMENTS	
	Span AB = 0.125 $WL^2 + \frac{PL}{2} -$ ½ (MB x MC1)	
	$= 0.125 \times 80.2 \times \frac{8^2 + 115.26}{2} x \frac{1}{2}[(5190.381) - (-18.81)]$	
	= 852.64KNm	
	Span BC1	
	= 0.125 x 80.2 x 3.770^2 – ½ (-18.81 + 712.65).	
	= 204.44KNm	
	Span C1C	
	= 0.125 x 80.28^2 – ½ (712.65)	
	= 176.95KNm	
	Span CD	
	= 0.125 x 80.28^2 – ½ (712.65 + 511.63) + $\frac{224.3 x 8}{2}$	
	= 926.66KNm	
	Span DD1	
	= 0.125 x 58.29 x 5.3^2 – ½ (511.63)	
	= 51.15KNm.	
	Reinforcement	
	Supports	

Reference	Calculation	Out put

Support B.

$$K = \frac{519.38 \times 10^6}{25 \times 300 \times 710^2} = 0.1374$$

Z = 576.54mm

$$As = \frac{519.38 \times 10^6}{0.87 \times 410 \times 576.54} = 2525.53 mm^2$$

Provide 3 + 3 Y25 (As Provide = 2950mm^2/m)

Support C1

$$K = \frac{18.81 \times 10^6}{25 \times 300 \times 710^2} = 0.004975$$

Z = 706.00mm

$$As = \frac{18.81 \times 10^6}{0.87 \times 410 \times 7106} = 74.69 mm^2$$

Provide 2Y16 (As Provide = 402mm^2/m Top)

Support C

$$K = \frac{712.65 \times 10^6}{25 \times 300 \times 710^2} = 0.1885 > 0.156$$

Compression reinforcement required

Z = 551.50mm

Steel compressed

MU = 0.156 FCubd^2

= 0.156 x 25 x 300 x 710^2 = 589.80 x 10^6 KNm

$$As = \frac{(712.65 - 589.80) \times 10^6}{0.95 \times 410 \times (d - d^1)}$$

$$= \frac{122.85 \times 10^6}{0.95 \times 410 \times 710 - 20} = 463.83 mm^2$$

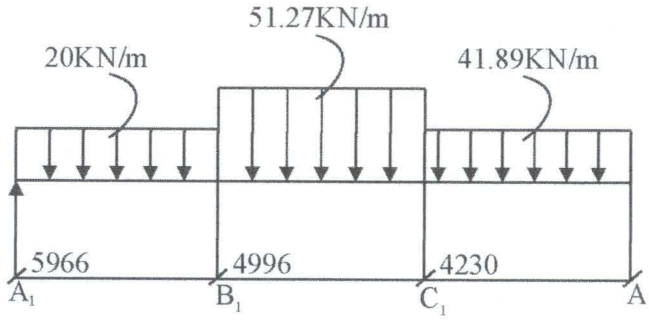
Reference	Calculation	Out put
	Provide 2 Y20 (As Prov = 628mm^2/m bottom)	
	Tension steel	
	As = $\dfrac{589.80 \times 10^6}{0.87 \times 410 \times 551.50} = 2998.17mm^2$	
	Provide 3 + 4 Y25 (As Prov = 3440mm^2/m)	
	Support D	
	K = $\dfrac{51.63 \times 10^6}{25 \times 300 \times 710^2} = 0.1353$	
	Z = 579.11mm	
	As = $\dfrac{511.63 \times 10^6}{0.87 \times 410 \times 579.11} = 2476.81mm^2$	
	Provide 5 Y25 (As Prov = 2450mm^2/m Top)	
	span reinforcement	
	Span AB	
	bf = bw x $\dfrac{0.7L}{10}$	
	$= 300\ x\ \dfrac{0.7\ x\ 8000}{10} = 860mm$	
	K = $\dfrac{852.64 \times 10^6}{300 \times 860 \times 710^2} = 0.006555$	
	Z = 704.78mm	
	As = $\dfrac{852.64 \times 10^6}{0.87 \times 410 \times 704.28} = 3391.63mm^2$	
	Provide 3 + 4 Y25 (As Prov = 3440mm^2/m bottom)	
	Provide 3 Y top.	

Reference	Calculation	Out put
	Span BC1	
	bf= $300\ x\ \frac{0.7\ x\ 3770}{10} = 563.9mm$	
	K = $\frac{204.44 \times 10^6}{25 \times 300 \times 710^2} = 0.05407$	
	Z = 664.45mm	
	As = $\frac{204.44 \times 10^6}{0.87 \times 410 \times 664.4} = 862.65mm^2$	
	Provide 3 Y20 (As Prov = 943mm^2/m)	
	Provide 2 Y 16 Top.	
	Span C_1C	
	bf= $300\ x\ \frac{0.7\ x\ 4230}{10} = 596.10mm$	
	K = $\frac{176.95 \times 10^6}{25 \times 596.1 \times 710^2} = 0.04680$	
	Z = 670.85mm	
	As = $\frac{176.95 \times 10^6}{0.87 \times 410 \times 670.85} = 739.47mm^2$	
	Provide 3 Y20 (As Prov = 943mm^2/m bottom)	
	Provide 2 Y 16 Top	
	Span CD	
	K = $\frac{926.66 \times 10^6}{25 \times 410 \times 710^2} = 0.085499$	
	Z = 573.84mm	
	As = $\frac{926.66 \times 10^6}{0.75 \times 410 \times 573.84} = 452717mm^2$	

Reference	Calculation	Out put
	Provide 5 + 5 Y25 (As Provide = 4910mm^2/m bottom)	
	Provide 3 Y 16 Top	
	Span DD1	
	bf = $300\ x\ \dfrac{0.7\ x\ 5300}{10} = 611mm$	
	K = $\dfrac{51.15 \times 10^6}{25 \times 671 \times 710^2} = 0.00604488$	
	Z = 705.21mm	
	As = $\dfrac{51.15 \times 10^6}{0.87 \times 410 \times 705.21} = 203.34mm^2$	
	Provide 3 Y16 (As Prov = 603mm^2/m bottom)	
	Check deflection	
	Fs = 5/8 x 410 x $\dfrac{4527.17}{4910}$ x $\dfrac{1}{1} = 236.67$N/mm	
	Mf = $0.55 + \dfrac{477 - 236.27}{120\left(0.9 + \dfrac{926.66\ \text{x}\ 10^6}{860\ \text{x}\ 710^2}\right)} = 1.21$	
	$\dfrac{Limiting\ span}{depth} = 1.21\ x\ 26 = 31.47$	
	$\dfrac{Actual\ span}{depth} = \dfrac{8000}{710} = 11.27$	
	Shear	
	VAB = $\dfrac{\text{WL}}{2} + \dfrac{PL}{L} = \left(\dfrac{0 - \text{MB3}}{\text{L}}\right)$	
	$= \dfrac{80.28\ \text{x}\ 8}{2} + \dfrac{115.26\ \text{x}\ 8}{8} + \left(\dfrac{0 - 519.38}{8}\right) = 371.14$KN	
	VBA = $\dfrac{80.2\ \text{x}\ 8}{2} + \dfrac{115.26\ \text{x}\ 8}{8} + \left(\dfrac{519.38 - 0}{8}\right) = 500.98$KN	

Reference	Calculation	Out put
	$Vc = \dfrac{0.79\,(1.615)^{1/3}\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.642$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87fyv} = \dfrac{0.4 \ x\ 300}{0.87 \ x\ 250} 0.551$	
	Provide Y10 @ $250^{c}/_{c}$	
	Check Max Shear	
	$Vs = 0.6f - wu \ x\ \dfrac{0.3}{2}$	
	$= 0.6 \ x\ 570.23 - 80.2 \ x\ 0.15 = 330.11KN$	
	Check Max Shearer	
	$V = \dfrac{300.11 \ x\ 10^{3}}{300 \ x\ 710} = 1.549$	
	End support	
	Shear at distance d from support face	
	$Vd = 0.45 \ x\ 570.23 - 80.2\ (0.710 \ x\ 0.15) = 187.63KN$	
	$V = \dfrac{187.63 \ x\ 10^{3}}{300 \ x\ 710} = 0.888$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87fyv} = \dfrac{0.4 \ x\ 300}{0.87 \ x\ 250} 0.552$	
	Provide Y10 @ 250mm	
	$Fs = 5/8 \ \ x\ 410 \ x\ \dfrac{825.13}{943} \ x\ \dfrac{1}{1} = 224.22N/mm^{2}$	
	$Mf = 0.55 + \dfrac{477 - 224.22}{120\left(0.9 + \dfrac{204.4 \ x\ 10^{6}}{840.79 \ x\ 710^{2}}\right)} = 2.07$	
	$\dfrac{Limiting\ span}{depth} = \ 2.07 \ x\ 20 = 41.47$	

Reference	Calculation	Out put
	$\frac{Actual\ span}{depth} = \frac{8797}{710} = 12.39$	
	Shear	
	F = wu x spam	
	= 21.13 x 8797 = 18588KN	
	At face of support	
	Shear Vs = $\frac{f}{2} - \frac{\text{wu x supportw with}}{2}$	
	$= \frac{185.88}{2} - 21.13\, x\, \frac{0.225}{2} = 90.56KN$	
	Shear stress	
	$V = \frac{Vs}{bd} = \frac{90.56 \text{ x } 10^3}{225 \text{ x } 710} = 0.567$	
	Shear from distance d from the face of the support	
	Vd = Vs – wud	
	= 90.56 – 21.13 x 0.710	
	= 75.56KN	
	$V = \frac{75.56 \text{ x } 10^3}{225 \text{ x } 710} = 0.473$	
	$\frac{100AS}{bd} = \frac{100\, x\, 943}{225\, x\, 710} = 0.590$	
	$Vc = \frac{0.79\,(1.590)^{1/3}\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.459$	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.87fyv} = \frac{0.4\, x\, 225}{0.87\, x\, 250} 0.414$	
	Provide Y10 @ 250$^c/_c$	

Reference	Calculation	Out put
	BEAM GRID LINE $2A_1 - 2A_1$	
	LOADINGS	
	Beam size 225 x 600	
	Self weight = 0.225 x 0.225 x 0.6 x 24 = 3.24KN/m	
	Wall = 7.65KN/m	
	Finishes = 0.25KN/m	
	Total = 11.4KN/m	
	Load on spans	
	Span BICI	
	Load from stair (flight) = 15.03KN/m	
	$= \frac{1}{2} \times 15.03 \times 3.595 (1 - 1/3 \times 1.39^2$	
	= 22.34KN/m	
	Total dead load = 11.14 x 22.34 = 33.48KN/m	
	Design load = 33.48 x 1.4 x 4.4 = 51.27KN/m	
	Span C1A	
	$= \frac{1}{2} \times 11.14 \times 4 \left(1 - \frac{1}{3\, x\, 1.06^2}\right) = 15.64 KN/m$	
	Total dead load = 11.14 x 15.64 = 26.78KN/m	
	Design load = 26.76 x 1.4 + 4.4 = 41.89KN/m	
	Design load span A1B1	
	1.4 x 11.14 + 4.4 = 20.0KN/m	
		

Reference	Calculation	Out put
	Analysis	

$$\text{FEMA1B1} = \frac{WL^2}{12} = \frac{20\ x\ 5.966^2}{12} = 59.32KN/m$$

$$\text{FEMB1C1} = \frac{WL^2}{12} = \frac{51.27\ x\ 4.966^2}{12} = 105.36KN/m$$

$$\text{FEMC1A} = \frac{WL^2}{12} = \frac{41.89\ x\ 4.23^2}{12} = 62.46KN/m$$

Stiffness factors

$$\text{KAB1} = \frac{3EI}{L} = \frac{3EI}{5.966} = 0.50EI$$

$$\text{KB1C1} = \frac{4EI}{L} = \frac{4EI}{4.996} = 0.80EI$$

$$\text{KC1A} = \frac{3EI}{L} = \frac{3EI}{4.23} = 0.71EI$$

DISTRIBUTION FACTOR

$$\text{DFAB1} = \frac{KAB1}{KAB1 + KB1C1} = \frac{0.5}{0.5 + 0.8} = 0.39$$

$$\text{DFB1C1} = 1 - 0.39 = 0.61$$

$$\text{DFC1B1} = \frac{KC1B1}{KC1B1 + K1A} = \frac{0.8}{0.8 + 0.71} = 0.66$$

$$\text{DFB1C1} = 1 - 0.66 = 0.34$$

Reference	Calculation	Out put

A		B1		C1	A
+59.32	0.39	0.61	0.66	0.34	
	-59.3	+105.36	-105.36	+62.46	-62.46
	-29.66	0	0	31.23	
	- 88.98	105.36	-105.36	93.69	0
	- 6.39	-9.99	7.70	3.97	
		3.85	-4.995		
	- 1.50	-2.35	2.30	1.70	
		1.15	-1.18		
	-0.45	-0.70	0.78	0.40	
		0.39	-0.23		
	-0.15	-0.24	0.23	0.12	
	- 97.47	97.47	-100.88	100.00	

Support Moments

Support B1 = 97.47KNM

Support C1 = 100.88KNM

Span Moment

Span AB $= 0.125WL^2 - \frac{1}{2}(MB1 + MC1)$

$= 0.125 \times 20 \times 5.966 - \frac{1}{2}(97.47 + 100.88)$

= 84.26KN/m

MB1C1 $= 0.125 \times 41.89 \times 4.23 - \frac{1}{2}(100.88)$

= 18.44KN/m

MC1A $= 0.125 \times 41.89 \times 4.23 - \frac{1}{2}(100.88 + 97.47)$

= 77.03KN/m

Reinforcement

Supports

Support B

$$K = \frac{97.47 \times 10^6}{25 \times 255 \times 130^2} = 0.0553$$

Z = 523.20mm

Reference	Calculation	Out put
	$\text{As} = \dfrac{97.47 \times 10^6}{0.87 \times 410 \times 523.20} = 522.28mm^2$	
	Provide 3 Y20 (As Provide = 943mm^2/m Top)	
	Provide 2 Y 16	
	Support C1	
	$\text{K} = \dfrac{100.88 \times 10^6}{25 \times 255 \times 560^2} = 0.0572$	
	Z = 521.77mm	
	$\text{As} = \dfrac{100.88 \times 10^6}{0.87 \times 410 \times 521.77} = 542.03mm^2$	
	Provide 3 Y 20 (As Provide = 943mm^2/m Top)	
	Span reinforcement	
	Span A1B1	
	$\text{bf} = bw + \dfrac{0.7L}{10} = 225 + \dfrac{0.7\ x\ 5966}{10} = 642.6^2\,mm$	
	$\text{K} = \dfrac{84.26 \times 10^6}{25 \times 642.62 \times 560^2} = 0.01672$	
	Z = 549.38mm	
	$\text{As} = \dfrac{84.26 \times 10^6}{0.87 \times 410 \times 549.38} = 429.98mm^2$	
	Provide 2 Y20 (As Provide = 628mm^2/m bottom)	
	Provide 2 Y 16 Top	
	Span B1C1	
	$\text{bf} = 225 + \dfrac{0.7\ x\ 4996}{10} = 574.72mm$	

Reference	Calculation	Out put
	$K = \dfrac{18.44 \times 10^6}{25 \times 574.72 \times 560^2} = 0.004092$	
	$Z = 557.41mm$	
	$As = \dfrac{18.44 \times 10^6}{0.87 \times 410 \times 557.41} = 414.41mm^2$	
	Provide 2 Y20 (As Provide = 626mm^2/m bottom)	
	Provide 2 Y 12 Top	
	Span C1A	
	$bf = 225 + \dfrac{0.7\ x\ 4.230}{10} = 521.1mm$	
	$K = \dfrac{77.03 \times 10^6}{0.87 \times 410 \times 521.1} = 414.32mm^2$	
	Provide 2 Y20 (As Prove = 628mm^2/m)	
	Provide 1 Y 16 Top	
	Check deflection	
	$Fs = 5/8 \quad x\ 410\ x \dfrac{429.28}{628}\ x\ \dfrac{1}{1} = 175.45mm$	
	$Mf = 0.55 + \dfrac{477 - 175.45}{120\left(0.9 + \dfrac{84.26\ x\ 10^6}{642.6\ x\ 560^2}\right)} = 2.46$	
	$\dfrac{Limiting\ span}{depth} = 2.46\ x\ 26 = 63.87$	
	$\dfrac{Actual\ span}{depth} = \dfrac{5966}{560} = 10.65$	
	Shear	
	$VA1B1 = \dfrac{WL}{2} = \dfrac{0 - MB1}{L}$	
	$= \dfrac{20\ x\ 5.966}{2} + \dfrac{0 - 97.47}{5.966} = 43.32KN$	

Reference	Calculation	Out put
	$VB1A1 = \frac{20\ x\ 5.966}{2} + \frac{97.47 - 0}{5.966} = 76KN$	
	$VB1C1 = \frac{51.27\ x\ 4.996}{2} + \frac{97.47 - 100.88}{4.996} = 127.39KN$	
	$VC1B1 = \frac{51.27\ x\ 4.996}{2} + \frac{100.88 - 97.47}{4.996} = 128.75KN$	
	$VC1A = \frac{41.89\ x\ 4.23}{2} + \frac{100.88 - 0}{4.230} = 112.45KN$	
	$VAC1 = \frac{41.89\ x\ 4.23}{2} + \frac{0 - 100.88}{4.230} = 64.75KN$	
	Check Shear	
	V = 128.75KN	
	$V = \frac{128.75\ x\ 10^3}{22.5\ x\ 560} = 1.02$	
	$\frac{100AS}{bd} = \frac{100\ x\ 943}{225\ x\ 560} = 1.02$	
	$Vc = \frac{0.79\ (0.784)^{1/3}\ \left(\frac{400}{560}\right)^{1/4}}{1.25} = 0.536$	
	$\frac{ASV}{Sv} = \frac{b(v - vc)}{0.87 fyv} = \frac{255(1.02 - 0.536)}{0.87\ x\ 250} = 0.501$	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.87 fyv} = \frac{0.4\ x\ 225}{0.87\ x\ 250} 0.414$	
	Provide Y10 @ $250^c/_c$	
	Check Max Shear	
	$Vs = 0.6f - wu = \frac{\text{support width}}{2}$	
	$= 0.6\ x\ 197.1 - 55.47\ x\ \frac{0.225}{2} = 112.02KN$	

Reference	Calculation	Out put
	$V = \dfrac{112.02\ x\ 10^3}{225\ x\ 560} = 0.889$ End support Shear at distance d from support face $Vd = 0.45f - wu\ \dfrac{(d + 0.225)}{2}$ $= 0.45 \text{ x } 197.1 - 55.47\ (0.7 + 0.115)$ = 43.49KN $V = \dfrac{43.49\ x\ 10^3}{22.5\ x\ 560} = 0.345$ $\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87 fyv} = \dfrac{0.4\ x\ 225}{0.87\ x\ 250} 0.414$ Provide Y 10 @ 250mm	

Reference	Calculation	Out put

BEAM ON GRID LINE B – B

Loading

Beam size 600 x 750

Self weight of the beam = 0.6 x 0.75 x 24

= 10.8KN/m

Wall = 7.65KN/m

Finishes = 0.25KN/m

Total = 18.7KN/m

Load on span 1 – 2 = 2 – 3 and 3 – 4

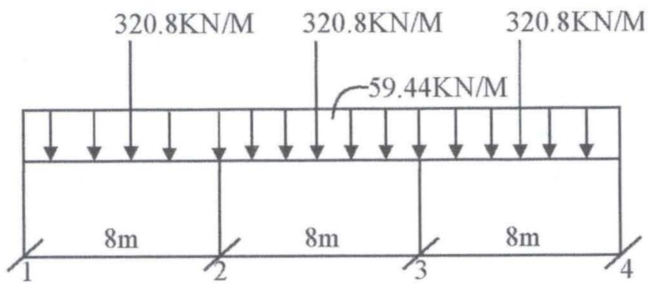
= ½ x 11.14 x 8 (1 – 1/3 x 1^2)

= 29.72 x 2 sides = 59.44KN/m

Load from BM on grid line

1A – 1A, 2A – 2A and 3A – 3A

$$\frac{WL}{2} = \frac{80.20\,x\,8}{2} = 320.8KN/m$$



Analysis

FEM 12 = FEM 23 = FEM4

$$= \frac{WL^2}{12} + \frac{pab^2}{L^2} = \frac{80.20\,x\,8^2}{12} + \frac{320.8\,x\,4\,x\,4^2}{8^2}$$

= 748.53KN/m

Stiffness factors

Reference	Calculation	Out put
	$K_{12} = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$	
	$K_{23} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	
	$K_{32} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	
	$K_{34} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$	
	$K_{43} = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$	
	Distribution factors	
	$DF_{12} = \frac{K12}{K_{12} + K_{23}} = \frac{0.38}{(0.38 \, x \, 0.5)EI} = 0.43$	
	$DF_{12} = 1 - 0.43 = 0.57$	
	$DF_{32} \; \frac{K_{23}}{K_{23} + K_{32}} = \frac{0.5EI}{(0.5 \, x \, 0.5)EI} = 0.5$	
	$DF_{34} = 1 - 0.5 = 0.5$	
	$= 0.125 \text{ x } 59.44 \text{ x } 8^2 \text{ x } \frac{320.8 \, x \, 8}{2} - \frac{1}{2}(906.30)$	
	= 1305.57KN/m	
	Span 23	
	$= 0.125 \text{ x } 59.44 \text{ x } 8^2 \text{ x } \frac{320.8 \, x \, 8}{2} - \frac{1}{2}(906.30 \, x 863.98)$	
	= 813.58KN/m	
	Span 34	
	$= 0.125 \text{ x } 59.44 \text{ x } 8^2 \text{ x } \frac{320.8 \, x \, 8}{2} - \frac{1}{2}(863.98)$	
	= 1326.73KN/m	

Reference	Calculation	Out put

Reinforcement

Supports

Support 2

$$K = \frac{906.30 \times 10^6}{25 \times 600 \times 710^2} = 0.1198$$

Z = 597.65mm

$$As = \frac{906.3 \times 10^6}{0.87 \times 410 \times 597.65} = 4251.3mm^2$$

Provide 5 + 5 Y25 (As Provided 491mm^2/m Top)

Support 3

$$K = \frac{863.98 \times 10^6}{25 \times 600 \times 710^2} = 0.11426$$

Provide 4 + 5 Y25 (As Provided 4420mm^2/m Top)

1		2		3	4
0	0.43	0.57	0.5	0.54	
743.53	-748.53	784.53	-784.53	748.53	-748.53
-748.53	- 374.26	0	0	374.26	+748.53
0	- 1122.79	748.53	748.53	1122.79	0
	160.93	213.33	187.13	-187.13	
		-93.56	106.66		
	40.23	53.33	-53.33	-53.33	
		-26.66	26.66		
	11.46	15.19	- 13.33	-53.33	
		-26.66	26.66		
	2.86	3.79	-3.79	-3.79	
		-1.89	1.89		
	0.81	1.07	0.94	-0.94	
		-0.47	0.53		
	0.20	0.26	-0.26	-0.26	
	-906.3	906.30	-863.98	863.01	

Reference	Calculation	Out put
	Support Moments	
	$M_2 = 906.30KNM$	
	$M_3 = 863.98KNM$	
	Span Moment	
	Span 1 – 2	
	$= 0.125 \text{ x } WL^2 \text{ x } \frac{PL}{2} - ½ (M_2)$	
	Span Reinforcement	
	Span 1 – 2	
	$bf = bw + \frac{0.7L}{10} = 600 \text{ x } \frac{0.7 \, x \, 8000}{10} = 1160mm$	
	$K = \frac{1305.57 \, x \, 10^6}{25 \, x \, 1160 \, x \, 710^2} 0.08930$	
	$Z = 630.07mm$	
	$A_s = \frac{1305.57 \times 10^6}{087 \times 410 \times 630.07} = 5809.09mm^2$	
	Provide 6 + 6Y 125 (As Prov = $5900mm^2/m$ Bottom)	
	Prov 3Y126 Top	
	Span 2 – 3	
	$K = \frac{873.58 \, x \, 10^6}{25 \, x \, 1160 \, x \, 710^2} 0.05975$	
	$Z = 659.22mm$	
	$A_s = \frac{873.58 \times 10^6}{087 \times 410 \times 659.22} = 3715.09mm^2$	
	Provide 4 + 4 Y25 (As Prov = $3930mm^2/m$)	
	Prov 3Y16 Top	

Reference	Calculation	Out put

Span 3 – 4

$$K = \frac{1326.73\ x\ 10^6}{25\ x\ 1160\ x\ 710^2} 0.09075$$

Z = 629.25mm

$$A_s = \frac{1326.73 \times 16^6}{25 \times 1160 \times 710^2} = 5910.94mm^2$$

Z = 629.25mm

$$A_s = \frac{1326.73 \times 10^6}{087 \times 410 \times 629.25} = 5910.94mm^2$$

Provide 6 + 6 Y25 (As Prov = 5900mm^2/m Bottom)

Prov 3Y16 Top

Check deflection

$$Fs = 5/8 \quad x\ 410\ x\ \frac{5910.94}{5900}\ x\ \frac{1}{1} = 256.72N/MM$$

$$Mf = 0.55 + \frac{477 - 256.72}{120\left(0.9 + \frac{1326.73\ x\ 10^6}{11.60\ x\ 710^2}\right)} = 1.13$$

$$\frac{Limiting\ span}{depth} = 1.13\ x\ 26 = 29.36$$

$$\frac{Actual\ span}{depth} = \frac{8000}{710} = 11.27$$

Shear

$$V_{12} = \frac{WL}{2} + \frac{PL}{L} + \frac{0 - M_2}{L}$$

$$= \frac{59.44\ x\ 8}{2} + \frac{320.8\ x\ 8}{8} + \frac{0 - 906.3}{8} = 444.47KN$$

$$V_{21} = \frac{59.44\ x\ 8}{2} + \frac{320.8\ x\ 8}{8} + \frac{906.3 - 0}{8} = 671.8KN$$

Reference	Calculation	Out put

$$V_{23} = \frac{59.44\,x\,8}{2} + \frac{320.8\ \ x\ \ 8}{8} + \frac{906.3 - 863.98}{8} = 563.05KN$$

$$V_{32} = \frac{59.44\,x\,8}{2} + \frac{320.8\ \ x\ \ 8}{8} + \frac{863.98 - 906.3}{8} = 553.27KN$$

$$V_{34} = \frac{59.44\,x\,8}{2} + \frac{320.8\ \ x\ \ 8}{8} + \frac{863.93 - 0}{8} = 666.55KN$$

$$V_{43}\ \frac{59.44\,x\,8}{2} + \frac{320.8\ \ x\ \ 8}{8} + \frac{0 - 863.93}{8} = 450.57KN$$

Check Shear

V = 666.55KN

$$V = \frac{666.55\,x\,10^3}{300\,x\,710} = 3.13$$

$$\frac{100As}{bd} = \frac{100\,x\,4910}{300\,x\,710} = 2.31$$

$$Vc = \frac{0.79\,(2.231)^{1/3}\,\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.723$$

$$\frac{ASV}{Sv} = \frac{0.4b}{0.87fyv} = \frac{0.3\,x\,300}{0.87\ 250}0.552$$

Provide Y10 @ $250^c/_c$

CHECK MAX SHEAR:

$$Vs = 0.6F - wu\ x\ \frac{0.3}{2}$$

= 0.6 x 666.55 – 59.44 x 0.15

= 391.01KN

$$V = \frac{391.01\,x\,10^2}{300\,x\,710} = 1.836$$

End support

Shear at Distance d from support face

Reference	Calculation	Out put
	Vd = 0.45 x 666.55 – 59.44 (0.710 + 0.15)	
	= 248.83KN	
	$V = \dfrac{248.83\,x\,10^{2}}{300\,x\,710} = 1.168$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87\,fyv} = \dfrac{0.4\,x\,300}{0.87\,x\,250} = 0.552$	
	Provide Y10 @ 250mm $^{c}/_{c}$	

Reference	Calculation	Out put

BEAM ON GRID LINE B1 B1

Loadings

Beam size 225 x 600

Self weight = 0.225 x 0.6 x 24 = 3.24KN

Wall = 7.65KN/m

Finishes = 0.25KN/m

Total = 11.KN/m

D = 600 – 30 ½ x 20 = 560mm

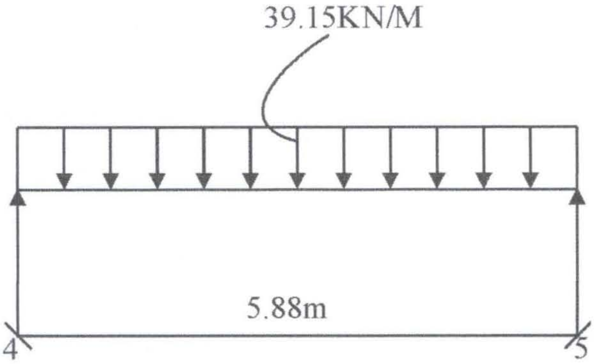
Load from star (flight)

= ½ x 15.03 x 1.885 (1 – 1/3 x 3.12^2)

= 13.68KN/m

Total dead load = 1.14 x 13.68 = 24.82KN/m

Design Load = 25.9 x 1.4 x 4.4 = 39.15KN/m



M = $0.125wl^2$

= 0.125 x 39.15 x 5.888^2

= 169.65KN/m

Reinforcement

Bf = 225 x $\frac{0.7 x 5888}{10}$ = 637.16mm

Reference	Calculation	Out put
	$K = \dfrac{169.65\, x\, 10^6}{25\, x\, 637.16\, x\, 560} 0.0339$	
	$Z = 538.03mm$	
	$A_s = \dfrac{169.65 \times 10^6}{087 \times 410 \times 538.03} = 88.398mm^2$	
	Provide 3 Y 120 (As Prov = 943mm^2/m Bottom)	
	Prov 2Y16 Top	
	Check deflection	
	$Fs = 5/8 \quad x\ 410\ x\ \dfrac{883.98}{943}\ x\ \dfrac{1}{1} = 240.21N/mm$	
	$Mf = 0.55 + \dfrac{477 - 240.21}{120\left(0.9 + \dfrac{169.65\ x\ 10^6}{637.16\ x\ 710^2}\right)}$	
	$\dfrac{Limiting\ span}{depth} = 1.68\, x\, 20 = 3356$	
	$\dfrac{Actual\ span}{depth} = \dfrac{5888}{560} = 10.51$	
	Deflection Ok	
	Shear	
	F = wu x span	
	= 39.15 x 5888	
	= 230.52KN	
	At face of support	
	Shear Vs = f/s – wu x $\dfrac{\text{support width}}{2}$	
	= 110.86KN	
	Shear stress $V = \dfrac{Vs}{bd} = \dfrac{100.86\, x\, 10^3}{250\, x\, 560} = 0.879$	

Reference	Calculation	Out put
	Shear from distance	
	Vd = Vs – wud	
	= 110.86 – 39.15 x 0.56 = 88.94KN	
	$V = \dfrac{88.94\ x\ 10^3}{225\ x\ 260} = 0.706$	
	$\dfrac{100As}{bd} = \dfrac{100\ x\ 943}{250\ x\ 560} = 0.784$	
	$Vc = \dfrac{0.79\,(0.784)^{1/3}\,\left(\frac{400}{560}\right)^{1/4}}{1.25} = 0.528$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.87fyv} = \dfrac{0.4\ x\ 225}{0.87\ 250}0.414$	
	Provide Y10 @ $275^{c}/_{c}$	

Reference	Calculation	Out put

BEAM ON GRID LINE C1 – C1

Loading size = 300 x 750

Self weight = 0.3 x 0.75 x 24

= 5.4KN/m

Wall = 7.65KN/m

Finishes = 0.25KN/m

Total = 13.3KN/m

Load from panel 1

$= ½ \text{ n/x} \left[1 - \frac{1}{3k^2}\right]; k = ly/lx$

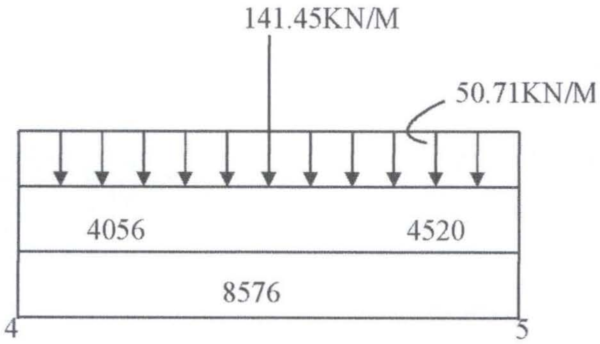
$= ½ \text{ x } 11.14 \text{ x } 4.23 \left[1 - \frac{1}{3\,x\,2.8^2}\right]; 19.78KN/m$

Total dead load = 13.3 + 19.78 = 33.18

Design load = 1.4 x 33.08 * 4.4 = 50.71KN/m

Load from beam span CA convert to point load

$= \frac{WL}{2} = \frac{66.88\,x\,4.23}{2} = 141.45KN$



Take moment above 3A

M = 50.71 x 8.576 x 8.576 + 141.45 x 4056

= 2438.52KNm

Reinforcemnt

Reference	Calculation	Out put
	$bf = bw\ x\ \frac{0.7L}{10} = 300 + \frac{0.7\ x\ 8576}{10} = 900.32$	
	$K = \frac{2438.52 \times 10^6}{25 \times 900 \times 710^2} = 0.21499 > 0.156$	
	$Z\ =\ d(0.5 + \sqrt{0.25 - \frac{0.156}{0.9}}$	
	$Z\ =\ 710(0.5 + \sqrt{0.25 - \frac{0.156}{0.9}}\ 551.59mm$	
	Compressor bar	
	$Mu = 0.156\ x\ 25\ x\ 900\ x\ 32\ x\ 710^2 = 1700.02$	
	$As^1 = \left(\frac{2438.52 - 1770.02}{0.95\ x\ 410\ x\ 680}\right) x\ 10^6 = 2523.97mm^2$	
	Prove 3 + 3 Y 25 (As prov top = 2950mm^2/m	
	Tension bar	
	$As\ \frac{1770.02\ x\ 10^6}{0.87\ x\ 410\ x\ 551.59} + 252397 = 11520.16mm^2$	
	Provide 7 + 7 Y 32 + 2 Y 16	
	(As prove = 11662mm^2/m bottom)	
	Check deflection	
	$= 5/8\ \ x\ 410\ x\ \frac{11520}{11662}\ x\ \frac{1}{1} = 253.13$	
	$Mf = 0.55 + \frac{477 - 253.13}{120\left(0.9 + \frac{2438.52\ x\ 10^6}{785\ x\ 710^2}\right)} = 0.847$	
	$\frac{Limiting\ span}{depth} =\ 0.847\ x\ 20 = 16.95$	
	$\frac{Actual\ span}{depth} = \frac{8576}{710}\ =\ 12.08$	

Reference	Calculation	Out put

Shear

F = wu x spam

= 50.71 x 8576 = 434.89KN

At face of support

$$\text{Shear } V_5 = f/2 - wu \times \frac{\text{support with}}{2}$$

$$= \frac{434.89}{2} - 50.71\,x\,\frac{0.3}{2} = 209.84KN$$

Shear stress

$$V = \frac{Vs}{bd} = \frac{209.84\,x\,10^3}{300\,x\,710} = 0.985$$

Shear from distance d from the face of the support

Vd = Vs – wud = 2309.84 – 50.71 x 0.71 = 173.84KN

$$V = \frac{173.84\,x\,10^3}{300\,x\,710} = 0.816$$

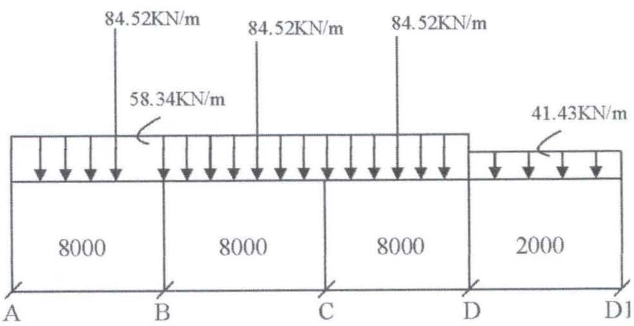
$$\frac{100As}{bd} = \frac{100\,x\,11520}{300\,x\,710} = 5.408$$

$$Vc = \frac{0.79\,(5.408)^{1/3}\left(\frac{400}{710}\right)^{1/4}}{1.25} = \mathbf{0.}961$$

$$\frac{ASV}{Sv} = \frac{0.4b}{0.87fyv} = \frac{0.4\,x\,300}{0.87\,x\,250} = 0.552$$

Provide Y10 @ 250mm

Reference	Calculation	Out put
	TYPICAL FLOOR BEAMS	
	D h – c – ½ Φ	
	h = 750mm	
	b = 225mm	
	c = 30mm	
	d = 750 – 30 – ½ x 0	
	d = 710mm	
	BEAM ON GRID LINE 1 – 1	
	Loading:	
	Self weight of the beam = 0.225 x 7.65KN/m	
	= 4.05KN/m	
	Wall = 2.55 x 3.0 = 7.65KN/m	
	Finishes = 0.0125 x 20 = 0.25KN/m	
	Total = 11.95KN/m	
	TRIANGULAR LOAD FROM PANELS (SLAB) ON AB, BC AND CD:	
	½ nl x $\left(1-\frac{1}{3k^2}\right)$	
	K = ly/lx	
	ly/lx = $\frac{8}{4}$ = 2.0	
	= ½ x 14.5 x 4000 $\left(1-\frac{1}{3\,x\,2^2}\right)$	
	= 26.58KN/m	
	Total Dead Load gk = 26.58 + 11.95 = 38.53KN/m	
	Design Load = 1.4gk + 1.69k	

Reference	Calculation	Out put
	$qk = ½ \times 1.5 \times \left(1 - \frac{1}{3x\,2^2}\right)$	
	= 2.75KN/m	
	Design load = 1.4 x 38.53 + 1.6 x 2.75	
	= 58.34KN/m	
	LOAD ON SPAN DD1:	
	Self weight = 4.05KN/m	
	Wall = 7.65KN/m	
	Finishes = 0.25KN/m	
	Total = 11.9KN/m	
	Load from slab (panel 16)	
	14.5KN/m	
	Total dead load = 11.95 = 14.5 + 26.45KN/m	
	Design load = 1.4 x 26.45 + 1.6 x 2.75	
	= 41.43KN/m	
		
	ANALYSIS:	
	$FEMAB = BC = CD = \frac{WL^2}{12} + \frac{pab^2}{L^2}$	
	$= \frac{58.52\,x\,8^2}{12} + \frac{54.52\,x\,4\,x\,4^2}{8^2}$	
	$FEMDD1 = \frac{WL^2}{12} + \frac{41.43\,x\,2^2}{12}$	

Reference	Calculation	Out put
	Stiffness Factor $KAB = \frac{3E1}{L} = \frac{3E1}{8} = 0.38EI$ $KBC = \frac{4E1}{L} = \frac{4E1}{8} = 0.5EI$ $KCB = \frac{4E1}{8} = 0.5EI$ $KCD = \frac{4E1}{L} = \frac{4E1}{8} = 0.5EI$ $KDC = \frac{4E1}{8} = 0.5EI$ $KDD1 = \frac{3E1}{L} = \frac{34E1}{2} = 0.5EI$ DISTRIBUTION FACTOR $DFAB = \frac{KAB}{KAB + KBC} = \frac{0.38EI}{0.38 + 0.5(EI)} = 0.43$ DFBC = 1 – 0.43 = 0.57 $DFCB = \frac{KBC}{KCB + KCD} = \frac{0.5EI}{0.5 + 0.5(EI)} = 0.5$ DFBC = 1 – 0.5EI = 0.5 $DFCD = \frac{KDC}{KDC + KDD1} = \frac{0.5EI}{0.5 + 0.5(EI)} = 0.5$ DFDD1 = 1 – 0.5 = 0.5	

Reference	Calculation	Out put

	0.43	0.57	0.5	0.5	0.5	0.5	
22.59	-227.51	227.59	-227.59	227.59	-227.59	13.81	-13.81
	-113.80	0	0	113.80	0	6.91	
	-341.39	227.59	-227.59	341.39	-227.59	20.72	
	48.93	64.87	-56.9	-56.9	103.44	103.44	
		-28.45	32.44		-28.45		
	12.33	16.22	16.22	16.22	14.23	14.23	
		-8.11	8.11		-8.11		
	3.49	16.22	4.06	4.06	4.06	4.06	
		-2.03	2.31		-2.03		
	0.87	1.16	-1.16	4.06	4.06	4.06	
		-0.58	0.58		-0.58		
	0.25	0.33	-0.29	-0.29		0.29	
				0.29			
	-275.62	275.62	-262.78	+262.78	-143.72	143.76	

MOMENTS:

Support Moments

Support B = 275.62KNm

Support C = 262.78KNm

Support D = 143.72KNm

SPAN MOMENT:

Span AB = $0.125WL^2 + \frac{PL}{2} - ½ (MB + MC)$

$= 0.125 \times 58.34 \times 8^2 + 84.52 \times 8 - ½ (275.62 + 262.78)$

= 536.6KNm

Reference	Calculation	Out put
	SPAN BC:	
	$= 0.125 \times 58.34 \times 8^2 - ½ (262.78)$	
	$= 335.33KNm + \dfrac{84.52\ x\ 8}{2} = 673.41KNm$	
	SPAN CD:	
	$= 0.125 \times 58.34 \times 8^2 + \dfrac{84.52\ x\ 8}{2} - ½ (262.78 + 143.72)$	
	$= 601.55KNm$	
	SPAN DD1:	
	$= 0.125 \times 58.34 \times 8^2 - ½ (143.72)$	
	$= 394.86KNm$	
	REINFORCEMENT	
	Support:	
	Support B	
	$K = \dfrac{275.62 \times 10^6}{25 \times 225 \times 710^2} = 0.097$	
	$Z = d(0.5 + \sqrt{0.25 - \dfrac{k}{0.9}}$	
	$Z = 622.55mm$	
	As $\dfrac{275.62 \times 10^6}{0.87 \times 410 \times 622.55} = 1241.18mm^2$	
	Provide 4 Y 20 (As Prov = 1260mm^2/m Top)	
	Support D	
	$K = \dfrac{143.72 \times 10^6}{25 \times 225 \times 710^2} = 0.051$	
	$Z = 667.4mm$	

Reference	Calculation	Out put
	As $\dfrac{143.72 \times 10^6}{0.87 \times 410 \times 667.4} = 603.71mm^2$	
	Provide 4 Y 16 (As Prov = 804mm^2/m Top)	
	SPAN REINFORCEMENT	
	Span AB	
	$Bf = 225 + \dfrac{0.7L}{10}$	
	$225 + \dfrac{0.7\ x\ 8000}{10} = 785mm$	
	$K = \dfrac{535.6\ x\ 10^6}{25\ x\ 785\ x\ 710^2} 0.0541$	
	Z = 664.32mm	
	$A_s = \dfrac{535.6 \times 10^6}{087 \times 410 \times 664.32} = 22602.27mm^2$	
	Provide 5 Y 25 (As Prov = 2450mm^2/m Bottom)	
	Span BC	
	$K = \dfrac{673.41\ x\ 10^6}{25\ x\ 785\ x\ 710^2} 0.0681$	
	Z = 651.42mm	
	$A_s = \dfrac{673.41 \times 10^6}{087 \times 410 \times 651.42} = 2898.11mm^2$	
	Provide 6 Y 25 (As Prov = 2950mm^2/m Bottom)	
	Span CD	
	$K = \dfrac{601.55\ x\ 10^6}{25\ x\ 785\ x\ 710^2} 0.061$	
	Z = 701.93mm	

Reference	Calculation	Out put
	$A_s = \dfrac{601.55 \times 10^6}{087 \times 410 \times 701.93} = 2402.56 mm^2$	
	Provide 5 Y 25 (As Prov = 2450mm^2/m Bottom)	
	Span DD1	
	$K = \dfrac{394.86\ x\ 10^6}{25\ x\ 785\ x\ 710^2} 0.0399$	
	Z = 676.90mm	
	$A_s = \dfrac{394.86 \times 10^6}{087 \times 410 \times 6769} = 1635.37 mm^2$	
	Provide 6 Y 20 (As Prov = 1890mm^2/m Bottom)	
	Check Deflection	
	$Fs = 5/8\ \ x\ 410\ x\ \dfrac{2898}{2950}\ x\ \ \dfrac{1}{1} = 251.73 mm^2/m$	
	$Mf = 0.55 + \dfrac{477 - 251.73}{120\left(0.9 + \dfrac{673.41\ x\ 10^6}{785\ x\ 710^2}\right)} = 1.27$	
	$\dfrac{Limiting\ span}{depth} = 1.27\ x\ 26 = 33.06$	
	$\dfrac{Actual\ span}{depth} = \dfrac{8000}{710} = 11.27$	
	Deflection Ok	
	Shear	
	$V_{AB} = \dfrac{WL}{2} + \dfrac{PL}{L} + \dfrac{0 - MB}{L}$	
	$= \dfrac{58.34\ x\ 8}{2} + \dfrac{84.52\ \ x\ \ 8}{8} + \dfrac{0 - 75.62}{8} = 283.43 KN$	
	$V_{BA} = \dfrac{58.34\ x\ 8}{2} + \dfrac{84.52\ \ x\ \ 8}{8} + \dfrac{275.62 - 0}{8} = 352.33 KN$	

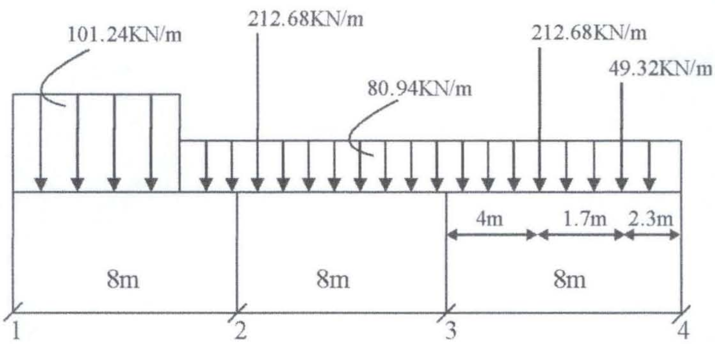
Reference	Calculation	Out put
	$V_{BC} = \frac{58.34\ x\ 8}{2} + \frac{84.52\ \ x\ \ 8}{8} + \frac{275.62 - 262.78}{8} = 319.49KN$	
	$V_{CB} = \frac{58.34\ x\ 8}{2} + \frac{84.52\ \ x\ \ 8}{8} + \frac{262.78 - 275.62}{8} = 316.28KN$	
	$V_{CD} = \frac{58.34\ x\ 8}{2} + \frac{84.52\ \ x\ \ 8}{8} + \frac{143.72 - 262.78}{8} = 302.99KN$	
	$V_{DC} = \frac{58.34\ x\ 8}{2} + \frac{84.52\ \ x\ \ 8}{8} + \frac{262.78 - 143.72}{8} = 332.76KN$	
	$V_{DD1} = \frac{41.42\ \ x\ \ 2}{2} + \frac{143.72 - 0}{2} = 113.28KN$	
	$V_{D1D} = \frac{41.42\ \ x\ \ 2}{2} + \frac{0 - 143.72}{2} = 30.44KN$	
	CHECK SHEAR	
	V = 352.33KN	
	$V = \frac{352.33 \times 10^3}{225 \times 710} = 2.21$	
	$\frac{100As}{bd} = \frac{100\ x\ 1260}{255\ x\ 710} = 0.789$	
	$Vc = \frac{0.79\ (0.789)^{1/3}\ \left(\frac{400}{710}\right)^{1/4}}{1.25} = \mathbf{0.}506$	
	$Vc < 0.5 + \sqrt{25}\ OK$	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.87\,fyv} = \frac{0.4\ x\ 250}{0.87\ x\ 250} = 0.41$	
	Provide Y10 @ $250^{c}/_{c}$	

Reference	Calculation	Out put
	BEAM ON GRID LINE B – B	
	Beam size = 450 x 750	
	Wall = 7.65KN/m	
	Finishes = 0.25KN/m	
	Total = 16.0KN/m	
	Load on spans:	
	Spans 12 = ½ x 14.5 x 4 $\left(1-\frac{1}{3\,x\,2^2}\right)$; K = $\frac{8}{2}$ = 2	
	= 26.58 x 2 = 53.17KN/m	
	Span 23.1 x $\frac{14.5}{2}$ x 8 $\left(1-\frac{1}{3\,x\,1^2}\right)$ K = $\frac{8}{8}$ = 1.0	
	= 38.67KN/m	
	Span 34 = 38.67KN/m	
	Load from beams 26, 24 and 28, convert to point	
	Load beams 24 and 26	
	$= \frac{WL}{2}$; w = 53.17KN/m	
	$= \frac{53.17}{2}\,x\,8 = 212.68KN$	
	Beam 28	
	W = 15.91KN/m	
	$\frac{15.91\,x\,6.2}{2} = 49.32KN$	
	Design Loads	
	Span 12	
	Dead Loads = 16 + 38.67 = 54.67KN/m	
	Design Load = 1.4 x 54.67 + 4.4 = 80.94KN/m	

Reference	Calculation	Out put

Span 34

Dead Loads = 80.94KN/m



Fixed Moment

$$\text{FEM}_{12} = \pm\frac{WL^2}{12} = \frac{101.24\ x\ 8^2}{12} = 539.95KNm$$

$$\text{FEM}_{23} = \frac{WL^2}{12} + \frac{pab^2}{L^2} = \frac{80.94\ x\ 8^2}{12} + \frac{212.68\ x\ 4^0\ x\ 4^2}{8^2}$$

= 1282.4KN/m

Structural Analysis By E. Nebo

$$\text{FEM}_{34} = \frac{WL^2}{12} + \frac{(P_1 + P_2)L}{9} = \frac{80.94\ x\ 8^2}{12} + \frac{(212.68 + 49.32)^2}{8^2}$$

= 688.20KN/m

STIFFNESS FACTOR:

$$K_{12} = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$$

$$K_{21} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$K_{23} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$K_{32} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$K_{12} = \frac{3EI}{L} = \frac{3EI}{8} = 0.38EI$$

Reference	Calculation	Out put

DISTRIBUTION FACTORS:

$$DF_{12} = \frac{K_{12}}{K_{12} + K_{12}} = \frac{0.38}{(0.38 + 0.5)EI} = 0.43$$

$DF_{23} = 1 - 0.43 = 0.57$

$$DF_{32} = \frac{K_{23}}{K_{23} + K_{32}} = \frac{0.5EI}{(0.5 + 0.5)EI} = 0.5$$

$DF_{34} = 1 - 0.5 = 0.5$

1	2		3		4
	0.43	0.57	0.5	0.54	
+539.95	-539.95	1282.4	-784.53	688.20	-688.20
-539.95	- 269.98	0	0	344.1	+688.20
0	-809.93	1282.4	-1282.4	1032.3	0
	-203.16	-269.31	125.05	-125.05	
		62.53	-134.66		
	-26.89	-35.64	-67.33	67.33	
		-33.67	17.82		
	-14.48	19.19	-8.91	-8.91	
		-4.46	9.60		
	1.92	2.54	-4.8	-4.8	
		-2.4	1.27		
	1.03	1.37	-0.64	0.64	
		-0.32	0.67		
	0.14	0.18	-0.34	-0.34	
	-1051.37	1051.373	1210.01	1209.99	

SUPPORT MOMENTS

Support 2 = 1051.37KNm

Support 3 = 1210.01KNm

Span Moments

$M_{12} = 0.12wl^2 - ½ (m2)$

$= 0.125 \times 101.24 \times 8^2 - ½ (1051.37) = 284.69$KN/m

$$M_{23} = 0.125wl^2 - \frac{Pl}{2} - ½ (M2 + M3)$$

Reference	Calculation	Out put
	$= 0.125 \times 80.94 \times 8^2 - 212.68 \times 8 - ½$	
	$(1051.37 + 1210.01) = 1333.89$KN/m	
	$M_{34} = 0.125wl^2 - \frac{PI}{2} - \frac{PI}{2} - \frac{1}{2}(M3)$	
	$= 0.125 \times 80.94 \times 8^2 - \frac{212.68 \ x \ 8}{2} - \frac{49.32 \ x \ 8}{2} - \frac{1}{2}$	
	$(1210.01) = 1005.49$KN/m	
	REINFORCEMENT	
	Support 2:	
	$K = \frac{1051.37 \times 10^6}{25 \times 450 \times 710^2} = 0.1853$	
	Compressed Bars Required	
	$As^1 = \frac{M - MU}{0.87 fy \ (d - d^1)}$	
	d = 710mm	
	d^1 = 30mm	
	$As^1 = \frac{1051.37 - 0.156 fcu \ bd^2}{0.87 \ x \ 410 \ (710 - 30)}$	
	$= \frac{(1051.37 - 0.156 \ x \ 25 \ x \ 450 \ 710^2) \ x 10^6}{0.87 \ x \ 410 \ (680)}$	
	$= \frac{(1051.37 - 884.7070) \ x 10^6}{242556} = 687.14 mm^2$	
	Provide 3 Y 20 (As Prov = 943mm^2/m Bootm)	
	Tension Bar (Top):	
	$As = \frac{0.156 \ x \ fcubd^2}{0.87 fyz} + AS^1$	
	Z = 0.775d	

Reference	Calculation	Out put
	$= 0.775 \times 710 = 550.25mm$	
	$As = \dfrac{0.156 \times 25 \times 450 \times 710^2}{0.87 \times 410 \times 550.25} = 687.14mm^2$	
	$= 5194.47mm^2$	
	Provide 8 + 3 Y 25 (As Prov = $5400mm^2/m$ Top)	
	Support 3:	
	$K = \dfrac{1210.01 \times 10^6}{25 \times 450 \times 710^2} = 0.22134$	
	$As = \dfrac{0.156 \times 25 \times 450 \times 710^2}{0.87 \times 410 \times 550.25} + 1341.17 = 5848.61mm^2$	
	Provide 8 + 4 Y 25 (As Prov = $5890mm^2/m$ Top)	
	SPAN REINFORCEMENT:	
	Span 12	
	M = 284.69KN/m	
	$bf = bw + \dfrac{0.7L}{10} = 450 + \dfrac{0.7\,x\,8000}{10} = 1010mm$	
	$K = \dfrac{284.69\,x\,10^6}{25\,x\,1010\,x\,710^2} 0.02237$	
	Z = 691.86mm	
	$A_s = \dfrac{284.69 \times 10^6}{087 \times 410 \times 691.86} = 1153.59mm^2$	
	Provide 3 + 2 Y 20 (As Prov = $1570mm^2/m$ Bottom)	
	Provide 3 Y 16 Top	
	Span 23	
	M = 1333.89KNm	
	$K = \dfrac{1333.89\,x\,10^6}{25\,x\,1010\,x\,710^2} 0.10479$	

Reference	Calculation	Out put

Z = 614.48mm

$$A_s = \frac{1.333.89 \times 10^6}{087 \times 410 \times 614.48} = 6085.68mm^2$$

Provide 8 + 5 Y 25 (As Prov = 6380mm^2/m Bottom)

Provide 3 Y 16 Top

Span 34

M = 1005.49 KNm

$$K = \frac{1005.49\ x\ 10^6}{25\ x\ 1010\ x\ 710^2} 0.07899$$

Z = 640.96mm

$$A_s = \frac{1005.49 \times 10^6}{087 \times 410 \times 640.96} = 4397.88mm^2$$

Provide 5 + 4 Y 25 (As Prov = 4420mm^2/m Bottom)

Provide 3 Y 16 Top

Check Deflection:

$$Fs = 5/8 \quad x\ 410\ x\ \frac{6085.68}{6380}\ x\ \frac{1}{1} = 244.43mm^2$$

$$Mf = 0.55 + \frac{477 - fs}{120\left(0.9 + \frac{M}{bd^2}\right)}$$

$$0.55 + \frac{477 - 244.43}{120\left(0.9 + \frac{1333.89\ x\ 10^6}{1010\ x\ 710^2}\right)} = 1.101$$

$$\frac{Limiting\ span}{depth} = 1.101\ x\ 26 = 28.63$$

$$\frac{Actual\ span}{depth} = \frac{8000}{710} = 11.27$$

Deflection Ok

Reference	Calculation	Out put
	Shear:	

$$V_{12} = \frac{WL^2}{2} + \frac{0-M2}{L}$$

$$= \frac{101.24\,x\,8}{2} + \frac{0-1051.37}{8} = 273.54KN$$

$$V_{21} = \frac{101.24\,x\,8}{2} + \frac{1051.37-0}{8} = 536.38KN$$

$$V_{23} = \frac{80.94\,x\,8}{2} + \frac{212.68\,x\,8}{8} + \frac{1}{2}(1051.37-1210.01)$$

$$= 456.62KN$$

$$V_{32} = \frac{80.94\,x\,8}{2} + \frac{212.68\,x\,8}{8} + \frac{1}{2}(1210.01-1051.37)$$

$$= 616.26KN$$

$$V_{34} = \frac{80.94\,x\,8}{2} + \frac{(212.68+49.32)\,x\,8}{8} + \frac{1}{2}(1210.01)$$

$$= 1141.454KN$$

$$V_{43} = \frac{80.94\,x\,8}{2} + \frac{(212.68+49.32)\,x\,8}{8} + \frac{1}{2}(0-1210.01)$$

$$= 68.57KN$$

Check Shear

$$V = 1141.454KN$$

$$V = \frac{1141.454 \times 10^3}{225 \times 710} = 7.15$$

$$\frac{100As}{bd} = \frac{100As}{255\,x\,710}\,x\,\frac{1000\,x\,6380}{255\,x\,710} = 3.99$$

$$Vc = \frac{0.79\,(3.99)^{1/3}\left(\frac{400}{710}\right)^{1/4}}{1.25} = 1.08$$

$$Vc < 0.8 + \sqrt{fcu}$$

Reference	Calculation	Out put
	$= 0.8 + \sqrt{25} = 4.0$	
	Vc < = 4.0	
	Shear Ok	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.87fyv} = \frac{0.4 \, x \, 225}{0.87 \, x \, 250} = 0.414$	
	Provide Y10 @ $250^c/_c$	
	<u>Check Max Shear</u>	
	V = 0.6f – wu x $\frac{\text{support with}}{2}$	
	= 0.6 x 1141.45 – 101.24 x $\frac{0.225}{2}$	
	= 673.48KN	
	Vs = $\frac{673.48 \, x \, 10^2}{225 \, x \, 710} = 4.22$	
	End Support	
	Shear at Distance d from support face	
	Vd = 0.45 – wu $\frac{(d + 0.225)}{2}$	
	= 0.45 x 1141.45 – 101.24 (0.710 + 0.115)	
	= 430.13KN	
	V = $\frac{430.13 \, x \, 10^3}{225 \, x \, 710} = 4.69$	
	$\frac{Asv}{Sv} = \frac{0.46}{0.87fyv} = \frac{0.4 \, x \, 255}{0.87 \, x \, 250} = 0.414$	
	Provide Y10@ $250^c/_c$	

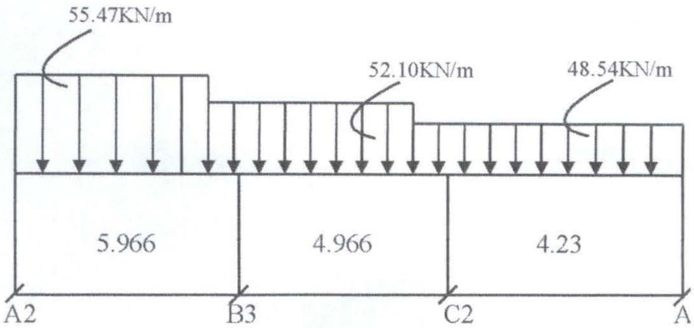
Reference	Calculation	Out put
	BEAM ON GRID LINE $3A_2 - 3A_2$	
	Loading	
	Beam size 225 x 600mm	
	Self weight of the beam = 0.225 x 0.6 x 24 = 3.2KN/m	
	Wall = 7.65KN/m	
	Finishes = 0.25KN/m	
	Total = 11.14KN/m	
	Lead on spans	
	Span A2 – B2	
	Triangular loading from slab = ½ n lx $\left[1 - \frac{1}{3\,x\,2^2}\right]$	
	; K = ly/lx	
	= ½ x 14.5 x 4.085 $\left[1 - \frac{1}{3\,x\,1.5^2}\right]$	
	= 25.34KN/m	
	Design load	
	Total dead load = 11.14 + 25.34 = 36.48KN/m	
	Design load = 36.48 x 1.4 + 4.4 = 55.47KN/m	
	Span B3 – C2	
	Load from slab (p3)	
	= ½ x 14 x 4.056 $\left[1 - \frac{1}{3\,x\,1.23^2}\right]$ = 22.93KN/m	
	Dead load = 11.14 + 22.93 = 34.07KN/m	
	Design load = 34.07 x 1.4 + 4.4 = 52.10KN/m	
	SPAN C2A	

Reference	Calculation	Out put
	Load from p5 = ½ x 14.5 x 4.056 $\left[\dfrac{1-1}{3\,x\,1.04^2}\right]$	

= 20.39KN/m

Dead load = 11.14 + 20.39 = 31.53KN/m

Design load = 31.53 x 1.4 + 4.4 = 48.34KN/m



Analysis

$$FEMA2B3 = \frac{wL^2}{12} = \frac{55.47\,x\,5.966^2}{12} = \pm 164.53KN/m$$

$$FEMB3C2 = \frac{wL^2}{12} = \frac{52.1\,x\,4.966^2}{12} = \pm 108.37KN/m$$

$$FEMC2A = \frac{wL^2}{12} = \frac{48.54\,x\,4.23^2}{12} = \pm 72.38KN/m$$

Stiffness factor

$$KA2B3 = \frac{3EI}{L} = \frac{3EI}{5.966} = 0.5EI$$

$$KC2B3 = \frac{4EI}{L} = \frac{4EI}{4.996} = 0.8EI$$

$$KC2A = \frac{4EI}{L} = \frac{4EI}{4.23} = 0.95EI$$

$$KAC2 = \frac{3EI}{L} = \frac{3EI}{4.23} = 0.71EI$$

Distribution factor

$$DFA2B3 = \frac{KA2B3}{KA2B3 + KB3C3} = \frac{0.5EI}{(0.5 + 0.8)EI} = 0.39$$

DFB3C2 = 1 – 0.39 = 0.61

Reference	Calculation	Out put

$$\text{DFC2B3} = \frac{KC2B3}{KC2B3 + KC2A} = \frac{0.8EI}{(0.8 + 0.95)EI} = 0.46$$

DFB3C2 = 1 – 0.46 = 0.54

$$\text{DFC2A} = \frac{KC2A}{KC2A + KAC2} = \frac{0.95EI}{(0.95 + 0.71)EI} = 0.57$$

DFB3C2 = 1 – 0.57 = 0.43

A2		B3		C4	A
	0.39	0.61	0.46	0.54	
+164.53	-164.53	108.37	-1089.370	72.38	-72.38
- 164.53	- 82.27	0	0	36.19	+72.38
0	- 246.8	108.37	-108.37	108.57	0
	- 53.99	84.44	-0.092	0.11	
		-0.05	42.22		
	0.02	0.03	-19.42	-0.01	
		-9.71	0.02		
	3.79	5.92	- 0.0092	-0.01	
		-0.0046	2.96		
	0.0018	0.0028	- 1.36	-1.50	
		-0.68	0.0014		
	0.27	0.41	0.000644	0.000756	
	- 188.73	188.73	- 84.05	84.03	

Support Moments

Support B3 = 188.73KNM

Support C2 = 84.05KNM

Span Moment

$$\text{Span A2B2} = 0.125\text{WL}^2 - \frac{1}{2}(188.72) = 68.19\text{KNM}$$

Span B3C2 =

$$0.125 \text{ x } 52.10 \text{ x } 4.996^2 \ \frac{1}{2}(188.73 + 84.05) = 110.4\text{KNM}$$

$$\text{Span C2A} = 0.125 \text{ x } 48.54 \text{ x } 4.23^2 - \frac{1}{2}(84.05) =$$

Reference	Calculation	Out put
	66.54KNM	
	Support C2	
	$K = \dfrac{84.05 \times 10^6}{25 \times 225 \times 560^2} = 0.02964$	
	Z = 685.52mm	
	$As = \dfrac{84.05 \times 10^6}{0.87 \times 410 \times 685.82} = 343.58mm^2$	
	Provide 2 Y 16 (As prov = 403 mm^2/m Top)	
	Span reinforcement	
	Span A2 B2	
	$Bf = bw \text{ x } \dfrac{0.76}{10} - 25 \text{ x } \dfrac{0.77 \text{ x } 5966}{10} = 642.62mm$	
	$K = \dfrac{68.19 \times 10^6}{25 \times 642.62 \times 560^2} = 0.005419$	
	Z = 703.26mm	
	$As = \dfrac{68.19 \times 10^6}{0.87 \times 410 \times 703.26} = 271.83mm^2$	
	Provide 2 Y 16 (As prov = 403 mm^2/m bottom)	
	$Bf = 225 \text{ x } \dfrac{0.7 \text{ x } 4996}{10} = 574.72mm$	
	$K = \dfrac{110.4 \times 10^6}{25 \times 574 \times 560^2} = 0.01524$	
	Z = 697.79mm	
	$As = \dfrac{110.4 \times 10^6}{0.87 \times 410 \times 697.79} = 443.55mm^2$	
	Provide 3 Y 16 (603 mm^2/m bottom)	
	Provide 2 Y 16 Top	

Reference	Calculation	Out put
	Span C2A $Bf = 225 \text{ x } \frac{0.7 \text{ x } 4230}{10} = 521.1\text{mm}$ $K = \frac{66.54 \times 10^6}{25 \times 52101 \times 560^2} = 0.0101$ Z = 701.94mm $As = \frac{66.54 \times 10^6}{0.87 \times 410 \times 701.94} = 265.75mm^2$ Provide 3 Y 16 (As prov = 403 mm^2/m bottom) Provide 2 Y 16 Top CHECK DEFLECTION $Fs = 5/8 \text{ x } 410 \text{ x } \frac{443.55}{603} \text{ x } \frac{1}{1} = 188.49\text{N/MM}$ $Mf = 0.55 + \frac{477 - 188.49}{120\left(0.9 \times \frac{110.4 \text{ x } 10^6}{574.72 \text{ x } 710^2}\right)}$ $\frac{Limiting\ span}{depth} = 2.43\ x\ 26 = 63.18$ $\frac{Actual\ span}{depth} = \frac{5966}{710} = 8.45$ Shear $VA2B3 = \frac{WL}{2} + \frac{0 - MB3}{L}$ $= \frac{55.47 \text{ x } 5.966}{2} + \frac{0 - 188.73}{5.966} = 133.89\text{KN}$ $VB3A2 = \frac{55.47 \text{ x } 5.966}{2} + \frac{188.73 - 0}{5.966} = 197.10\text{KN}$ $VC2B3 = \frac{52.10 \text{ x } 4.996}{2} + \frac{84.05 - 188.05}{4.996} = 109.2\text{KN}$	

Reference	Calculation	Out put
	$VC2A= \dfrac{48.54 \times 4.23}{2} + \dfrac{0-84.05}{4.23} = 82.79KN$	
	$VAC2 = \dfrac{48.5 \times 4.23}{2} + \dfrac{84.05-0}{4.23} = 122.53KN$	
	CHECH SHEAR:	
	V = 197.10KN	
	$V = \dfrac{197.1 \times 10^3}{225 \times 560} = 1.564$	
	$\dfrac{100As}{bd} = \dfrac{100 x 1010}{225 x 560} = 0.802$	
	$Vc = \dfrac{0.79\,(0.802)^{1/3}\left(\frac{400}{560}\right)^{1/4}}{1.25} = 0.534$	
	Provide Y10 @ 250	
	CHECK MAX SHEAR:	
	$Vs = 0.6F - WU \times \dfrac{Support\ Width}{2}$	
	$= 0.6 \times 128.75 - 51.27 \times 0.225$	
	Vs = 71.48KN	
	$V = \dfrac{Vs}{bd} = \dfrac{71.48 x 10^3}{225 x 560} = 0.567$	
	End support	
	Shear at Distance d from support face	
	$Vd = 0.45f - wu\left(d + \dfrac{0.225}{2}\right)$	
	$= 0.45 \times 128.75 - 51.27\,(0.560 + 0.115)$	
	Vd = 23.46KN	
	$V = \dfrac{23.46 x 10^3}{255 x 560} = 0.019$	

Reference	Calculation	Out put
	Norminal link $Asv = \frac{0.4b}{0.87fyv} = \frac{0.4\,x\,250}{0.87\,x\,250} = 0.414$ Provide Y10 @ 250mm$^{c}/_{c}$	

Reference	Calculation	Out put

BEAM ON GRID LINE B2 – B2

Loading:
b = 450mm
h = 750mm
d = h – c – ½ θ
d = 750 – 30 – 10 = 710mm

self weight of the beam = 0.45 x 75.24

= 8.1KN/m

Wall = 7.65KN/m

Finishes = 0.25/KN/m

Total = 16.0KN/m

Slab Load = ½ nl x (1 – 1/3K^2) Panel 4

$$= \frac{1}{2} \times 14.5 \times 4996 \left(1 - \frac{1}{3 \times 1.7^2}\right)$$

= 32.09 KN/m

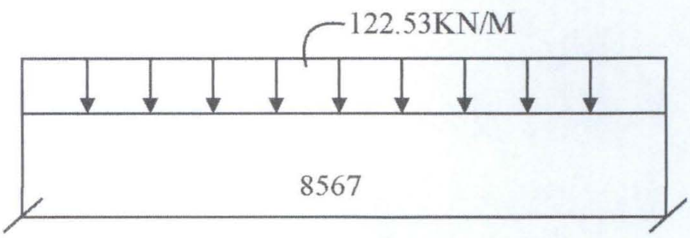
$$\text{Panel} = \frac{1}{2} \times 14.5 \times 5996 \left(1 - \frac{1}{3 \times 1.7^2}\right)$$

= 36.29 KN/m

Total load = 16.0 + 32.09 + 1.6 x 36.29

= 84.38KN/m

Design load = 1.4 x 84.38 + 4.4 = 122.53KN/m



M = 0.125WL^2 = 0.125 x 21.13 x 8^2

= 1126.49KNm

Reference	Calculation	Out put
	Share stress $= \dfrac{49784 \times 10^6}{450 \times 710^2} = 1.56$ Shear from distance Vd = Vs – Wud = 497.84 – 122.53 x 0.710 = 410.84KN $Vd = \dfrac{410.84 \times 10^3}{450 \times 710} = 1.29$ $\dfrac{100As}{bd} = \dfrac{100 \; x \; 5400}{450 \; x \; 710} = 1.69$ $Vc = \dfrac{0.79\,(1.6)^{1/3}\,\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.652$ $Vc < 0.8\ \sqrt{fcu}$ Shear OK. $\dfrac{ASV}{Sv} = \dfrac{04b}{0.87 \; x \; 250} = \dfrac{0.4 \; x \; 450}{0.87 \; x \; 250} = 0.827$ Provide Y10@ 175250 $^c/_c$ Links	

Reference	Calculation	Out put

BEAM ON GRID LINE C – C

Beam size = 450mm x 750mm

= 70mm

Self weight of the beam = 0.450 x 0.75 x 24

= 8.1KN/m

Wall = 7.65KN/m

Finishes = 0.25KN/m

Total = 16.0KN/m

Load from span 1 – 2

$$= \frac{1}{2} \times nLx\left(1 - \frac{1}{3k^2}\right)$$

$$\frac{1}{2} \times 14.5 \times 8\left(1 - \frac{1}{3\,1^2}\right)$$

= 38.63KN/m x 2 sides = 77.26KN/m

Load from span 2 – 3 and 3 – 4

$$= \frac{1}{2} \times 14.5 \times 8\left(1 - \frac{1}{3 \times 1^2}\right)$$

= 13.593 x 38.63 = 52.22KN/m

Load from span 4 – 5

$$= \frac{1}{2} \times 14.5 \times 4.235\left(1 - \frac{1}{3 \times 1.42^2}\right)$$

= 25.59 x 2 sides = 51.17Kn/m

SPAN 1 – 2

Total Dead Load gk = 16.0 + 77.26 = 93.26

Design Load = 93.2 x 1.4 x 4.4 = 134.96KN/m

SPAN 2 – 3 AND 3 – 4

Reference	Calculation	Out put
	$K3-2=\frac{4EI}{L}=\frac{4EI}{8}=0.5EI$	
	$K3-2=\frac{4EI}{L}=\frac{4EI}{8}=0.5EI$	
	$K4-3=\frac{4EI}{L}=\frac{4EI}{8}=0.5EI$	
	$K4-5=\frac{4EI}{L}=\frac{4EI}{6}=0.5EI$	
	DISTRIBUTION FACTOR	
	$DFI=\frac{K12}{K12+K23}=\frac{0.38EI}{0.38EI+0.5}=0.43$	
	$DF23=1-0.443=0.57$	
	$DF32=\frac{K32}{K32+K34}=\frac{0.38EI}{(0.5+0.5)EI}=0.5$	
	$DF34=1-0.5=0.5$	
	$DF43=\frac{K43}{K43+K45}=\frac{0.5EI}{(0.5+0.5)EI}=0.5$	
	DF3-4 $=1-0.5=0.5$	
	$DF45=\frac{K45}{K45+K43}=\frac{0.5EI}{(0.5+0.5)EI}=0.5$	
	DF5-4 $=1-0.5=0.5$	

Reference	Calculation	Out put

	0.43	0.57	0.5	0.5	0.5	0.5	
+719.79	-719.79	629.25	-629.25	629.25	-629.25	295.32	-295.32
-719.79	-359.90	0	0	147.63	0	147.66	-295.32
0	-079.69	629.25	-629.25	945.88	-629.25	442.98	
	193.69	265.75	-157.32	-157.32	93.14	93.14	
		-78.66	128.38		-78.66		
	33.82	44.84	-64.19	-64.19	39.33	39.33	
		-32.19	22.42		-32.10		
	13.80	18.30	-11.21	-11.21	16.05	16.05	
		-5.61	9.15		-5.61		
	2.41	3.20	-4.58	-4.58	2.81	2.81	
		-2.29	1.6		-2.29		
	098	1.31	-0.8	-0.8	1.15	1.15	
		-0.4	0.66		-0.5		
	0.17	0.23	-0.33		0.2	0.2	
	-834.82	834.31	-705.47	705.47	-595.63	595.66	

Support 2 = 834.82KNm

Support 3 = 705.47KNm

Support 4 = 595.66KNm

SPAN MOMENT

Span 12 = $0.125m^2 \ 0 - \frac{1}{2}(m_2 - m_3)$

$= 0.125 \text{ x } 134.96 \text{ x } 8^2 - \frac{1}{2}(834.82 + 705.47)$

M = 309.53KNm

SPAN 23

$M = 0.125 \text{ x } 9g.91 \text{ x } 8^2 \ - \frac{1}{2}(705.47) + \frac{pl}{2}$

Reference	Calculation	Out put
	$= 0.125 \text{ x } 96) \text{ } 91 \text{ x } 8^2 \quad -\frac{1}{2}(705.47) + \frac{96.4\ x\ 4}{2}$	
	$= 832.14KNm$	
	SPAN 34	
	$M = 0.125 \text{ x } 96). \text{ } 91 \text{ x } 8^2 \quad -\frac{1}{2}(705.47)\, 595.66 + \frac{96.4\ x\ 8}{2}$	
	$= 534.31KNm$	
	SPAN 45	
	$M = 0.125 \text{ x } 98.44 \text{ x } 62 \quad -\frac{1}{2}(594.66)$	
	$= 145.15KNm$	
	Reinforcement	
	SUPPORTS	
	SUPPORTS 2	
	$K = \frac{834.82\ x\ 10^6}{25\ x\ 450\ x\ \mathbf{710^2}} = 0.1472 < 0.156$	
	$Z = 563.75MM$	
	$As = \frac{834.82 \times 10^6}{0.87 \times 410 \times 563.75} = 4151.48mm^2$	
	Provide 9 Y 25(As prov = $4420mm^2/m$)	
	SUPPORT 3	
	$K = \frac{705.47\ x\ 10^6}{25\ x\ 450\ x\ \mathbf{710^2}} = 0.12439$	
	$Z = 59238mm$	
	$As = \frac{705.47 \times 10^6}{0.87 \times 410 \times 592.38} = 3338.68mm^2$	
	Provide 9 Y 25(As prov = $3930mm^2/m$ Top)	

Reference	Calculation	Out put
	SUPPORT 4	
	$K = \dfrac{7595.66\, x\, 10^6}{25\, x\, 450\, x\, \mathbf{710^2}} = 0.10533$	
	Z = 614.22 MM	
	$As = \dfrac{595.66 \times 10^6}{0.87 \times 410 \times 614.22} = 2718.76 mm^2$	
	Provide 9 Y 25(As prov = 2930mm^2/m Top)	
	SPAN REINFORCEMENT	
	SPAN 12	
	$bf = bw + \dfrac{0.7L}{10} = 450 + \dfrac{0.7\, x\, 8000}{10} = 1010 mm$	
	$K = \dfrac{309.53\, x\, 10^6}{25\, x\, 1010\, x\, \mathbf{710^2}} = 0.024317$	
	Z = 690.21MM	
	$As = \dfrac{309.53 \times 10^6}{0.87 \times 410 \times 690.21} = 1257.24 mm^2$	
	Provide 5 Y 20(As prov = 1570mm^2/m bottom)	
	Provide 3 Y 16 Top.	
	SPAN 23	
	$K = \dfrac{832.14\, x\, 10^6}{25\, x\, 1010\, x\, \mathbf{710^2}} = 0.065376$	
	Z = 654.04MM	
	$As = \dfrac{832.14 \times 10^6}{0.87 \times 410 \times 650.04} = 1257.24 mm^2$	
	Provide 8 Y 25(As prov = 1570mm^2/m)	
	SUPPORT 34	

Reference	Calculation	Out put

$$K = \frac{534.31\, x\, 10^6}{25\, x\, 1010\, x\, \mathbf{710^2}} = 0.041977$$

Z = 675.21MM

$$As = \frac{534.31 \times 10^6}{0.87 \times 410 \times 655.21} = 2218.46mm^2$$

Provide 8 Y 20(As prov = 2510mm^2/m)

Provide 3 Y 16 Top.

SPAN 45

$$bf = bw + \frac{0.7L}{10} = 450 + \frac{0.7\, x\, 6000}{10} = 870mm$$

$$K = \frac{145.15\, x\, 10^6}{25\, x\, 870\, x\, \mathbf{710^2}} = 0.01323$$

Z = 699.40MM

$$As = \frac{145.15 \times 10^6}{0.87 \times 410 \times 699.4} = 581.82mm^2$$

Provide 3 Y 20(As prov = 9430mm^2/m bottom)

Provide 3 Y 16 Top.

CHECK DEFLECTION

$$Fs = 5/8 \quad x\ 410\ x \frac{3566.88}{3930}\ x\ \frac{1}{1} = 232.57N/mm$$

$$Mf = 0.55 + \frac{477 - 232.57}{120\left(0.9 \times \frac{832.14\ x\ 10^6}{1010\ x\ 710^2}\right)} = 1.35$$

$$\frac{Limiting\ span}{depth} = 1.35\ x\ 26 = 35.20$$

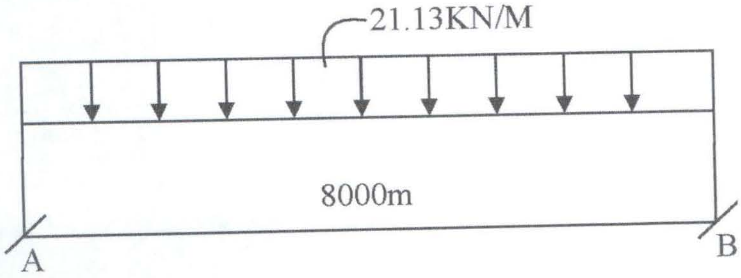
$$\frac{Actual\ span}{depth} = \frac{800}{710} = 11.27$$

<u>SHEAR</u>

Reference	Calculation	Out put
	$V_{12} = \frac{WL}{2} + \frac{0-M2}{L}$	
	$= \frac{134.96\,x\,8}{2} + \frac{0-834.82}{8} = 434.49KN$	
	$V_{21} = \frac{134.96\,x\,8}{2} + \frac{834.82-0}{8} = 643.19KN$	
	$V_{23} = \frac{99.19\,x\,8}{2} + \frac{834.82-705.47}{8} + \frac{96.4\,x\,8}{8} = 512.41KN$	
	$V_{32} = \frac{99.19\,x\,8}{2} + \frac{705.47-834.82}{8} + \frac{96.4\,x\,8}{8} = 480.07KN$	
	$V_{34} = \frac{99.91\,x\,8}{2} + \frac{96.4\,x8}{8} + \frac{705.47-595.66}{8} = 509.97KN$	
	$V_{43} = \frac{99.91\,x\,8}{2} + \frac{96.4\,x8}{8} + \frac{595.66-705.47}{8} = 482.51KN$	
	$V_{45} = \frac{99.44\,x\,8}{2} + \frac{0-595.66}{8} = 319.3KN$	
	$V_{54} = \frac{99.44\,x\,8}{2} + \frac{595.66-0}{8} = 468.22KN$	
	CHECK SHEAR	
	V = 643.19KN	
	$V = \frac{643.19 \text{ x } 10^3}{450 \text{ x } 710} = 2.013$	
	$\frac{100AS}{bd} = \frac{100\,x\,4420}{450\,x\,710} = 1.384$	
	$Vc = \frac{0.79\,(1.384)^{1/3}\,(\frac{400}{710})^{1/4}}{1.25} = 0.609$	
	$\frac{ASV}{Sv} = \frac{0.4b}{0.8fyv} = \frac{0.4\,x\,450}{0.87\,x\,250} 0.828$	
	Provide Y10 @ $200^{c}/_{c}$	

Reference	Calculation	Out put
	CHECK MAX SHEAR:	
	$Vs = 0.6f - wu \text{ x } \frac{\textit{Support width}}{2}$	
	$= 0.6 \text{ x } 643.19 - 134.96 \text{ x } \frac{0.45}{2} = 355.5KN$	
	$V = \frac{35.54 \, x \, 10^3}{450 \, x \, 710} = 1.112$	
	End Support	
	Shear at distance from support face	
	$Vd = 0.45f - wu \left(d \, x \frac{0.450}{2} \right)$	
	$0.45 \text{ x } 643.19 - 134.96 \, (0.710.0225)$	
	$Vd = 1163.25KN$	
	$V = \frac{163.25 \, x \, 10^3}{450 \, x \, 710} = 0.511$	
	$\frac{Asv}{Sv} = \frac{0.4b}{0.87 fyv} = \frac{0.4 \, x \, 450}{0.87 \, x \, 250} = 0.828$	
	Provide Y10 @ 200mm	
		21.13KN/m

Reference	Calculation	Out put
	BEAM ON GRID LINE A1 A1, B1 B1 AND C1 C1	
	Loading:	169.04KN/m
	Self weight of the beam = 0.225 x 0.75 x 24	
	= 4.05KN/m	
	Wall = 7.65 KN/m	
	Finishes = 0.25KN/m	
	Total = 11.95KN/m	
	Design load = 1.4 x 11.95 + 4.4 = 21.13KN/m	



$M = 0.125WL^2$

$= 0.125 \times 21.13 \times 8^2$

$= 169.04KN/m$

REINFORCEMENT

$$bf = 225 + \frac{0.7x\,8000}{10} = 785mm$$

$$K = \frac{169.04\ x\,10^6}{25\ x\ 785\ x\ 710^2} = 0.0171$$

$Z = 696.24mm$

$$As = \frac{169.04 \times 10^6}{0.87 \times 410 \times 696.24} = 680.66mm^2$$

Provide 4 Y 16(As prov = $804mm^2/m$ bottom)

Provide 3 Y 16 Top.

Check deflection

Reference	Calculation	Out put
	$Fs = 5/8 \times 410 \times \frac{680.66}{804} \times \frac{1}{1} = 216.93 N/mm$	
	$Mf = 0.55 + \frac{477 - 216.93}{120\left(0.9 \times \frac{169.04 \times 10^6}{785 \times 710^2}\right)} = 2.18$	
	$\frac{Limiting\ span}{depth} = 2.18 \times 20 = 43.66$	
	$\frac{Actual\ span}{depth} = \frac{800}{710} = 11.27$	
	Deflection OK	
	<u>Shear</u>	
	F = wu x span	
	= 21.13 x 8 = 169.04KN	
	At face of support	
	Shear $Vs = F/2 - wu \times \frac{Support\ Width}{2}$	
	Shear from distance d	
	Vd = Vs – Wud	
	= 86.14 – 21.13 x 0.710	
	Vd = 71.14KN	
	$Vd = \frac{71.14 \times 10^3}{225 \times 710} = 0.445$	
	$\frac{100As}{bd} = \frac{100 \times 804}{785 \times 710} = 0.144$	
	VC = 0.287	
	Shear ok $V_c < 0.8\sqrt{fcu}$	
	$\frac{ASV}{SV} = \frac{0.4b}{0.87fyv} = \frac{0.4 \times 255}{0.87 \times 250}$ 0.41	

Reference	Calculation	Out put

Provide Y10 @ 250

BEAM ON GRID LINE E – E

Loading:

Self weight of the beam = 4.05KN/m

Wall = 7.65KN/m

Finishes = 0.25KN/m

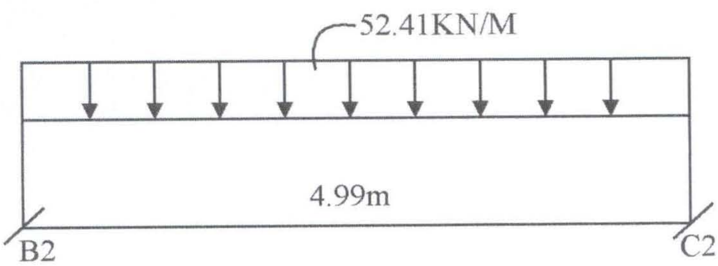
Total = 11.95KN/m

Load from stair (Flight) = 15.03KN/m

$$\tfrac{1}{2} \times 15.03 \times 3.59 \left(1 - \frac{1}{3 \times 1.7^2}\right) = 22.34 KN/m$$

Total dead load = 11.95 + 22.34 = 34.29KN/m

Design Load = 34.29 x 1.4 + 4.4 = 53.41KN/m



$M = 0.125 \times 52.41 \times 4.996^2 = 163.51KN/m$

REINFORCEMENT:

$$bf = 225 + \frac{0.7x\,4.996}{10} = 574mm$$

$$K = \frac{163.51\,x\,10^6}{25\,x\,574\,x\,710^2} = 0.0225$$

Z = 691.71mm

$$As = \frac{163.51 \times 10^6}{0.87 \times 410 \times 696.24} = 662.7mm^2$$

Provide 4 Y 16(As prov = 804mm^2/m bottom)

Reference	Calculation	Out put
	Provide 3 Y 16 Top.	
	Check deflection	
	$Fs = 5/8 \;\; x\ 410\ x \frac{662.7}{804}\ x\ \frac{1}{1} = 221.22N/mm$	
	$Mf = 0.55 + \frac{477 - 221.22}{120\left(0.9 \times \frac{662.7\ x\ 10^6}{785\ x\ 710^2}\right)} = 1.38$	
	$\frac{Limiting\ span}{depth} = 1.38\ x\ 20 = 27.56$	
	$\frac{Actual\ span}{depth} = \frac{4.996}{710} = 7.04$	
	Deflection OK	
	<u>Shear</u>	
	F = wu x span	
	= 52.41 x 4.996 = 261.84KN	
	At face of support	
	Shear $Vs = F/2 - wu\ x\ \frac{0.225}{2}$	
	$\frac{261.84}{2} - 52.41\ x\ 0.1125 = 125.02KN$	
	Shear stress $V = Vs = \frac{125.02\ x\ 10^3}{225\ x\ 710} = 0.782$	
	Shear from distance d	
	$V_d = V_s - Wud$	
	= 125.02 – 52.41 x 0.710	
	= 87.81KN	
	$Vd = \frac{87.81\ x\ 10^3}{225\ x\ 710} = 0.215$	

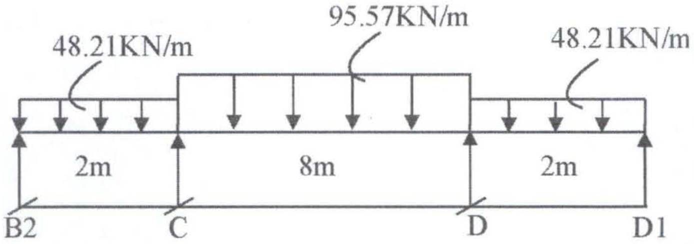
Reference	Calculation	Out put
	$\frac{100As}{bd} = \frac{100 \, x \, 804}{785 \, x \, 710} = 0.197$	
	$Vc = \frac{0.79 \, (0.197)^{1/3} \left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.320$	
	$\frac{ASV}{SV} = \frac{0.4b}{0.87 fyv} = \frac{0.4 \, x \, 255}{0.87 \, x \, 250} \; 0.41$	
	Provide Y10 @ $250^{c}/_{c}$	

Reference	Calculation	Out put
	BEAM ON GRID LINE 2B – 2B, 3 – 3 AND 3A1 – 3A1	
	Loading:	
	Beam size 225 x 750	
	Self weight of the beam = 0.225 x 0.75 x 24	
	= 4.05KN/m	
	Wall = 7.65 KN/m	
	Finishes = 0.25KN/m	
	Total = 11.95KN/m	
	Load on spans:	
	Spans B2C from panel 12	
	Trapezoidal load = $\frac{1}{3}nlx$	
	$= \frac{1}{3}\ x\ 14.5\ \ x\ 2\ sides = 19.34KN/m$	
	Load on span CD	
	Triangular Load $\frac{1}{2}nlx\left(1-\frac{1}{3k^2}\right);\ K = ly/lx$	
	$K = \frac{8}{4} = 2$	
	$= \frac{1}{2}\ x\ 14.5\ x\ 4\left(1-\frac{1}{3\ x\ 2^2}\right)$	
	= 26.58 x 2 sides = 53.16KN/m	
	Load on span DDI	
	As in span B2C = 19.34KN/m	
	Dead Load = 11.95 + 19.34 = 31.29KN/m	
	Design load = 1.4 x 31.29 + 44 = 48.21KN/m	
	Span CD	

Reference	Calculation	Out put

Dead load = 11.95 + 53.17 = 65.12KN/m

Design Load = 1.4 x 65.12 + 4.4 = 95.57KN/m



$$\text{FEMB2C} = \text{FEMDDI} = \pm\frac{wl^2}{12} = \frac{48.21\,x\,2^2}{12} = \mathbf{16.07KN/m}$$

$$\text{FEMCD} = \pm\frac{wl^2}{12} = \frac{95.57\,x\,8^2}{12} = 509.71KN/m$$

STIFNESS FACTOR:

$$\text{KB2C} = \frac{3EI}{L} = \frac{3EI}{2} = 1.5EI$$

$$\text{KCD} = \frac{4EI}{L} = \frac{4EI}{8} = 0.5EI$$

$$\text{KDC} = \frac{4EL}{L} = \frac{4EI}{8} = 0.5EI$$

$$\text{KCD} = \frac{3EI}{L} = \frac{3EI}{2} = 1.5EI$$

DISTRIBUTION FACTORS

$$\text{DFB2C} = \frac{B2C}{KB2C + KCD} = \frac{1.5EI}{1.5 + 0.5EI} = 0.75$$

DF CB2 = 1 – 0.75 = 0.25

$$\text{DFCD} = \frac{KCD}{KCD + KDC} = \frac{0.5EI}{0.5 + 0.5EI} = 0.5$$

DF CB2 = 1 – 0.5 = 0.5

Reference	Calculation	Out put
	Span CD: M = 512.317KN/m	
	$bf = 225 + \dfrac{0.7\,x\,8000}{10} = 785mm$	
	$K = \dfrac{512.31 \times 10^6}{25 \times 785 \times 710^2} = 0.05178$	
	Z = 666.51mm	
	$As = \dfrac{512.31 \times 10^6}{0.87 \times 410 \times 666.5} = 2154.92mm^2$	
	Provide 5 Y 25 (As prov = 2450mm^2/m bottom)	
	Provide 3 Y16 Top	
	Check deflection	
	$Fs = 5/8 \;\; x\; 410\; x\; \dfrac{215.92}{2450}\; x\; \dfrac{1}{1} = 225.39 N/mm^2$	
	$Mf = 0.55 + \dfrac{477 - 225.39}{120\left(0.9 \times \dfrac{512.31\,x\,10^6}{666.15\,x\,710^2}\right)} = 1.41$	
	$\dfrac{Limiting\ span}{depth} = 1.41\,x\,26 = 36.77$	
	$\dfrac{Actual\ span}{depth} = \dfrac{8000}{710} = 11.27$	
	Deflection OK	
	Shear	
	$V_{B2C} = \dfrac{WL}{2} + \dfrac{0 - M2}{L}$	
	$= \dfrac{48.21\,x\,2}{2} + \dfrac{0 - 126.74}{2} = -15.16KN$	
	$V_{CB2} = \dfrac{48.21\,x\,2}{2} + \dfrac{126.74 - 0}{8} = 111.58KN$	

Reference	Calculation	Out put
	$V_{CD} = \dfrac{95.57x\,8}{2} + \dfrac{126.74-377.76}{8} = 350.9KN$	
	$V_{DC} = \dfrac{95.57\;x\;8}{2} + \dfrac{377.76-126.74}{8} = 413.66KN$	
	$V_{DD1} = \dfrac{48.21\,x\,2}{2} + \dfrac{0-377.76}{2} = 140.67KN$	
	$V_{D1D1} = \dfrac{48.21\,x\,2}{2} + \dfrac{377.76}{2} = 237.89KN$	
	CHECK SHEAR	
	V = 416.66KN	
	$V = \dfrac{413.66 \text{ x } 10^3}{225 \text{ x } 710} = 2.59N/mm$	
	$\dfrac{100AS}{bd} = \dfrac{100\;x\;2950}{225\;x\;710} = 1.847$	
	$Vc = \dfrac{0.79\,(1.847)^{1/3}\,\left(\frac{400}{710}\right)^{1/4}}{1.25} = 0.672$	
	$\dfrac{ASV}{Sv} = \dfrac{0.4b}{0.8fyv} = \dfrac{0.4\;x\;225}{0.87\;x\;250} = 0.414$	
	Provide Y10 @ $250^c/_c$	
	CHECK MAXIMUM SHEAR:	
	$V = 0.6f - wu \text{ x } \dfrac{Support\ width}{2}$	
	$= 0.6 \text{ x } 413.66 - 95.57 \text{ x } \dfrac{0.225}{2} = 237.45KN$	
	$Vs = \dfrac{237.45\;x\;10^3}{225\;x\;710} = 1.49$	
	End Support	
	Shear at distance from support face	

Reference	Calculation	Out put
	$\text{Vd} = 0.45 - \text{wu}\left(d + \frac{Support}{2}\right)$	
	$= 0.45 \times 413.66 - 95.57\ (0.710 + 0.115)$	
	$= 107.31\text{KN}$	
	$\text{V} = \frac{107.31\ x\ 10^3}{225\ x\ 710} = 0.672$	
	$\frac{Asv}{Sv} = \frac{0.4b}{0.87\,fyv} = \frac{0.4\ x\ 255}{0.87\ x\ 250} = 0.414$	
	Provide Y10 @ $250^c/_c$	

Reference	Calculation	Out put
	CHAPTER FIVE **COLUMN DESIGN** Fcu = 25N/mm Fy = 410N/mm Cover to steel = 40mm Fire resistance 1hr	
Roof Beam	Roof beam Roof load = 1.5 x 1.5 = 2.25KN/m Slab = 0.175 x 24 = 4.2 KN/m Finishes = 1.0 Partition = 1.0 gk = 6.2KN/m qk = 2.5KN/m F = 1.4gk + 1.6qk = 1.4 x 6.2 + 1.6 x 2.5 = 12.68KN/m Say 13KN/m	

Reference	Calculation	Out put
Column C1	**COLUMN C1** Beam load Own load = 0.225 x 0.75 x 24 = 4.05 Finishes = 1.0 Wall = 3.5 (3.6) = 12.6 17.65KN/m Factored wall = 1.4 x 17.65 = 24.71KN Say 25KN 6^{th} floor – roof level Floor area = 8/2 x 8/2 = 16m^2 Roof load = 16 x 1.5 x 1.5 = 36KN Roof beam = (5.05) x 1.4 x (4 +4) = 48.27KN Column load = 0.225 x 0.4 x 24 x 1.4 x 3.6 = 12.25KN = 96.52KN 5^{th} floor – 6^{th} floor From above = 96.52 Slab load = 13 (16) = 208 Column load = 12.25KN Wall beam = 25 (4 + 4) = 200KN = 516.77KN 4^{th} floor – 5^{th} floor From above = 516.77KN Slab = 208 Column load = 12.25KN	

Reference	Calculation	Out put
	Wall/beam = 200KN	
	Total = 937.02KN	
	3rd floor – 4th floor	
	From above = 1357.27KN	
	Slab = 208KN	
	Column load = 12.25KN	
	Wall beam = 200KN	
	Total 1357.27KN	
	2nd floor – 3rd floor	
	From above = 1357.27KN	
	Slab = 208KN	
	Column load = 12.25KN	
	Wall/beam = 200KN	
	Total = 1777.52KN	
	1st floor – 2nd floor	
	From above = 1777.52	
	Slab 208KN	
	Column load = 12.25K	
	Wall beam = 200KN	
	Total 2197.77KN	

Reference	Calculation	Out put
	GOUND FLOOR – 1ST FLOOR	
	From above = 2197.77KN	
	Slab = 208KN	
	Column = 0.225 x 0.45 x 1.4 x 24 x 3.15	
	= 10.72KN	
	Total = 2616.49KN	
	Basement floor – ground floor	
	From above = 2616.49KN	
	Slab = 208KN	
	Column = 10.72KN	
	Wall beam = 200KN	
	Total = 3305.21KN	
Table 9.2 Mostley and Bengey Page 241	REINFORCEMENT	
	$\frac{Ley}{b} = \frac{0.75\,x\,3600}{225} = 13.6 < 15$	
	$\frac{Lex}{h} = \frac{0.85\,x\,3600}{450} = 6 < 15$	
	And conditions 1 Top and 2 bottom	

Reference	Calculation	Out put

Reference	Floor	N (KN)	M 0.05Nh Knm	$\frac{N}{bh}$	$\frac{M}{bh^2}$	$\frac{100As}{bh}$	Asc mm^2	
FROM CHART 9.7 MOSTLEY AND BUNGEY	6th Roof level	96.52	2.17	0.95	2.12	0.4	405	Provide 4Y16 as prove = 810mm^2
	5th floor – 6th floor	516.77	11.63	5.10	0.26	0.4	405	4 Y 16
	4th floor – 5th floor	937.02	21.08	9.25	0.46	0.4	405	4 Y 16
	3rd floor – 4th floor	1357.27	30.54	13.41	0.067	0.4	405	4 Y 16
	2nd floor – 3rd floor	1777.52	39.99	17.56	0.88	1.0	10125	6 Y 16 as prove = 1210
	1st floor – 2nd floor							8 Y 20 (As proove = 2510mm^2)
	Ground floor – 1st floor	2616.49	58.87	25.84	1.29	4.0	4050	10 Y 25 (as prove = 4910
	Basement floor – ground floor	3305.21	74.36	32.64	1.63	5.0	5062.5	10Y25 + 1Y16 As prove = 5111mm^2

Reference	Calculation	Out put
	COLUMN C2	
	6^{th} floor – roof level	
	Floor EW = 8/2 x 8/2 = 4 x 4 = 16m^2	
	Roof load = 16 x 1.5 x 1.5 = 36KN	
	Roof beam = (4.31) x 1.4 x (4 x 4) = 48.27KN	
	= 24.50KN	
	108.77KN	
	5^{th} floor – 6^{th} floor	
	From above = 108.77KN	
	Slab load = 13(4 x 4) = 208KN	
	Colum load = 24.50KN	
	Wall beam = 24 (4+4) = 192KN	
	Total = 533.27KN	
	4^{th} Floor – 5^{th} Floor	
	From above = 533.27KN	
	Slab = 208KN	
	Column load 24.50KN	
	Wall beam 192	
	Total 957.77KN	
	3^{rd} floor – 4^{th} floor	
	From above = 957.77KN	
	Slab = 208KN	
	Column = 24.50KN	
	Wall beam = 192KN	
	Total = 1382.27KN	

Reference	Calculation	Out put
	2^{nd} floor – 3^{rd} floor From above = 1382.27KN Slab = 208KN Column = 24.50KN Wall beam = 192KN Total = 1806.77KN 1^{st} floor – 2^{nd} floor From above = 1806.77KN Slab = 208KN Column = 24.5KN Wall beam = 192KN Total = 2231.27KN Ground floor – 1^{st} floor From above = 2231.27KN Slab = 208KN Column $0.45^2 \times 24 \times 1.4 \times 3.15 = 21.43$KN Wall/beam = 192KN Total 2652.7KN Basement floor – ground floor From above = 2652.7KN Slab = 208KN Column = 21.43KN Wall/beam = 192KN Total = 3074.13KN reinforcement Asc = $\dfrac{N - 0.4fcubh}{0.75fy - 0.4fcu}$	

Reference	Calculation	Out put
	$= \dfrac{108\,x\,77\,x\,10^{3} - 0.4\,x\,25\,x\,450\,x\,450}{0.75\,x\,410 - 0.4\,x\,25} = -6441.11mm^{2}$	
	As min = 0.4%bh	
	As max = 8%bh	
Section 3.12.5 of the reinforced concrete design by Victor Oyenuga 2^{nd} edition page 206	As min = 0.4%bh = 0.4 x 300 x 3000 = 360mm^2	
	Asc < As min	
	As min = 360mm^2	
	Provide 4 Y 16 (as prove = 804mm^2)	
	5^{th} floor – 6^{th} floor	
	N = 429.27KN	
	$\dfrac{533.27\,x\,10^{3} - 2025000}{297.5} = -5014.22mm^{2}$	
	As min = 0.4%bh = 0.4 x 450 x 450 = 810mm^2	
	Provide 6 Y 16 (As prove = 1210mm^2)	
	4^{th} floor – 5^{th} floor	
	N = 957.77KN	
	0.4bh = 0.4% x 450 x 450 = 810mm^2	
	Provide 6 Y 16 (As prove = 1210mm^2)	
	3^{rd} floor – 4^{th} floor	
	N = 1382.27KN	
	Asc = -2160.44mm^2	
	As = 1%bh = 1% x 450 x 450 = 2025mm^2	
	Provide 6 Y 25 (As provide = 2950mm^2)	
	2^{nd} floor – 3^{rd} floor	
	N = 747.62KN	
	Asc = 1277.38mm^2 (provide 6 Y 16)	

Reference	Calculation	Out put
	1^{st} floor – 2^{nd} floor N 920.12KN Asc = -1104.88mm^2 Provide 6 Y 16 Ground floor – 1^{st} floor N = 1089.55KN Asc = 1%bh = 1% x 450 x 450 = 2025mm^2 Provide 8 Y 20 (As Provide = 2510mm^2) Basement floor – ground floor N = 1258KN Asc = 1%bh = 1% x 450 x 450 = 2025mm^2 Provide 8 Y 20 (As Provide = 2510mm^2) LINKS ¼ of diameter = ¼ x 16 = 4mm Provide Y10 @ $200^c/_c$	

Reference	Calculation	Out put
Column C5	**COLUMN C5**	
	Beam load	
	Own load = 0.225 x 0.6 x 24 = 3.31	
	Finishes say = 1.0	
	Wall = 3.5 (4.2 – 0.6) = 12.6	
	Total = 16.91	
	Factored wall/beam load = 1.4 x 16.91 = 23.67KN/m	
	Say 24KN/m	
	6^{th} floor – roof level = $\frac{4.235}{2} x \frac{3765}{2} = 3.99mm^2$	
	Roof load = 3.99 x 1.5 x 1.5 = 8.98KN	
	Rood beam = (3.31 + 1.0) x 1.4 x (2.12 + 1.88)	
	= 24.14KN	
	Column own load and finishes = 0.45 x 0.45 x 24 x 1.4	
	(4.2 – 0.6) = 24.50KN	
	Total 57.62KN	
	5^{th} floor – 6^{th} floor	
	From above = 57.62KN	
	Slab load = 13 (2.12 x 1.88) = 52KN	
	Column load = 24.50	
	Wall/beam = 4(2.12 + 1.88) = 96KN	
	Total = 230.12KN	
	4^{th} floor – 5^{th} floor	
	From above = 230.12KN	
	Slab = 52KN	
	Column = 24.50KN	

Reference	Calculation	Out put
	Wall beam = 96KN	
	Total = 575.12KN	
	3^{rd} floor – 4^{th} floor	
	From above = 402KN	
	Slab = 52KN	
	Column = 24.50KN	
	Wall/beam = 93	
	Total = 5,75.12KN	
	2^{nd} floor = 3^{rd} floor	
	From above = 575.12KN	
	Slab = 52KN	
	Column = 24.50KN	
	Wall/beam = 96	
	Total = 747.62KN	
	1^{st} floor – 2^{nd} floor	
	From above = 747.62KN	
	Slab = 52KN	
	Column = 24.50KN	
	Wall/beam = 96KN	
	Total = 920.12KN	
	Ground floor – 1^{st} floor	
	From above = 920.12KN	
	Slab load = 52KN	
	Column = 0.45 x 0.45 x 3.15 x 24 x 1.4 = 21.43KN	
	Wall/beam = 96KN	

Reference	Calculation	Out put
	Total = 1,089.55KN	
	Basement floor – ground floor	
	From above = 1,089.55KN	
	Slab load = 52KN	
	Column load = 21.43KN	
	Wall/beam = 96KN	
	Total = 1258.98KN	
	DESIGN OF UNI-AXIAL COLUMN	
	6^{th} floor – roof floor	
	N = 101.16KN	
	Column height = 4200	
	Assume le = lo	
	Slenderness ratio lex/h = 4200/750 = 5.6 < 15	
Reinforce ment concrete design by Engr V. Oyenuga 2^{nd} edition page 206	Column design short column	
	N = 04fcubh + Asc (0.75fy – 0.4fcu)	
	$Asc = \dfrac{N - 0.4fcubh}{0.75fy - 0.4fcu}$	
	$= \dfrac{57.62 \times 10^3 - 0.4 \times 25 \times 450 \times 450}{0.75 \times 410 - 0.4 \times 25} = 6613.04mm^2$	
	Since this is negative use minimum	
	Reinforcement.	
	Area of reinforcement given by	
	Asmin = 0.4%bh	
	$= 0.4 \times 300 \times 300 = 360mm^2$	
	Provide 4Y16 (as prove = $804mm^2$)	
	5^{th} floor – 6^{th} floor	

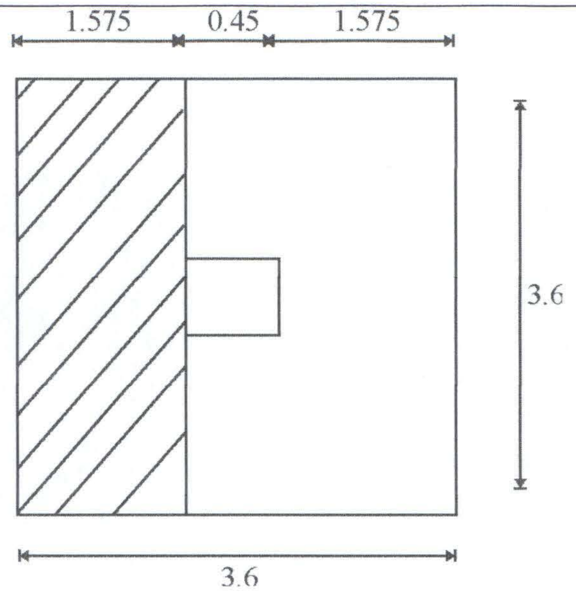
Reference	Calculation	Out put
	N = 230.12KN	
	Asc = $\dfrac{230.12\,x\,103 - 2025000}{297.5} = 1794.89mm^2$	
	Asmin = 0.4%bh = 0.4 x 450 x 450 = 810mm^2	
	Provide 6 Y 16 (As prove = 1210mm^2)	
	4th floor – 5th floor	
	N = 402.62KN	
	Asc = $\dfrac{40.62\,x\,103 - 2025000}{297.5} = 1622.38mm^2$	
	Provide 6 Y 16 (As prove = 1210mm^2)	
	3rd floor – 4th floor	
	N = 575.12KN	
	Asc = $\dfrac{575.12\,x\,103 - 2025000}{297.5} = 1449.88mm^2$	
	Provide 6 Y 16	
	2nd floor 3rd floor	
	N = 747.62KN	
	Asc = 1277.38mm^2 (provide 6 Y 16)	
	1st floor – 2nd floor	
	N = 920.12KN	
	Asc = - 1104.88mm^2	
	Prove 6 Y 16	
	Ground floor – 1st floor	
	N = 1089.55KN	
	Asc = 1%bh = 1% x 450 x 450 = 2025mm^2	
	Provide 8Y20 (As prov = 2510mm^2)	

Reference	Calculation	Out put
	Basement floor – ground floor $N = 1258.98KN$ $Asc = 1\%bh = 1\% \times 450 \times 450 = 2025mm^2$ Provide 8Y20 (As prov = $2510mm^2$) LINKS $\frac{1}{4}$ of $\Phi = \frac{1}{4} \times 16 = 4mm$ Provide Y10 @ $200^c/_c$	

Reference	Calculation	Out put
	ANALYSIS AND DESIGN OF FOUNDATION	
	Design stresses	
	Fcu = $25N/mm^2$	
	Fy = $410N/mm^2$	
	Concrete cover = 50mm	
	Thickness (h) = 600mm	
	Soil bearing capacity = $200KN/m^2$ (p)	
	Column Base Types B1	
	Axial load from C1 = 3305.21KN	
	Design load = 1.0GK + 1.0QK	
	$= \frac{3305.21}{1.47} + 10\% \, x \, 3305.21$	
	= 2578.96KN	
	Required area = $\frac{2578.96}{200} = 12.89$	
	Provide base area 3.6m square	
	Base area = $12.96m^2$	
	Column axial load = 2578.96KN	
	Earth pressure = $\frac{2578.98KN}{3.6\,x\,3.6} = 198.99KN/m^2$	
	Column size = 225 x 450	
	Assume thickness of 750mm	
	d = 750 – 50 – 10 = 690mm	
	stress on footing	
	at column face	
	shear stress, Vc = N/column perimeter xd	

Reference	Calculation	Out put
	$\frac{2578.96 \, x \, 103}{(225 \, x \, 2 + 450 \, x \, 2) \, x \, 690} = 2.77$	
	$2.77 < 0.8 \sqrt{fcu}$ shear ok	
	Punching shear	
	Critical perimeter = column perimeter + 8 x 1.5d = 1350	
	+ 8 x 1.5 x 690 = 1350 + 8280 = 9630mm	
	Area within perimeter = $(450 = 3d)^2$	
	$= (450 + 3 \text{ x } 690)^2 = 6.3 \text{ x } 10^6$	
	Pinching share force v	
	$= 198.99 \, (3.6^2 - 6.3) = 1325.27\text{KN}$	
	Pinshing shear V = $\frac{V}{Critial \ perimeter \ xd}$	
BS 81110 Part 1 1997 table 3.8	$= \frac{1325.27 \, x \, 10^3}{9630 \, x \, 690} = 0.19$	
	Vc = 0.37, hence punching shear ok	
	Bending reinforcement	
	At column face which is critical section	
	$M = (198.99 \text{ x } 3.6 \text{ x } 1.575) \, \frac{1575}{2} = 888.51\text{KNm}$	

Reference	Calculation	Out put



$$K = \frac{888.51 \times 10^6}{25 \times 3600 \times 690^2} = 0.02074$$

Z = 673.74mm

$$As = \frac{M}{0.87 fyz} = \frac{888.51 \times 10^6}{0.87 \times 410 \times 673.74} = 3697.15 mm^2$$

Provide Y 25 @ $125^c/_c$

(As prove = $3930mm^2$)

Final punching shear check

$$\frac{100As}{bd} = \frac{100\ x\ 3930}{3600\ x\ 690} = 0.16 N/mm^2$$

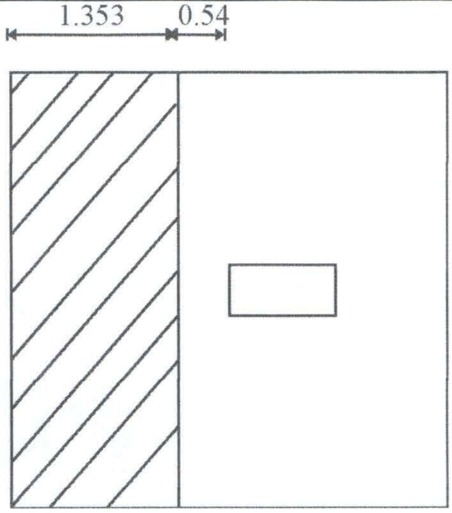
Ultimate Vc = $0.37N/mm^2$

Punching shear stress = 0.37

Hence 750mm thick pad ok

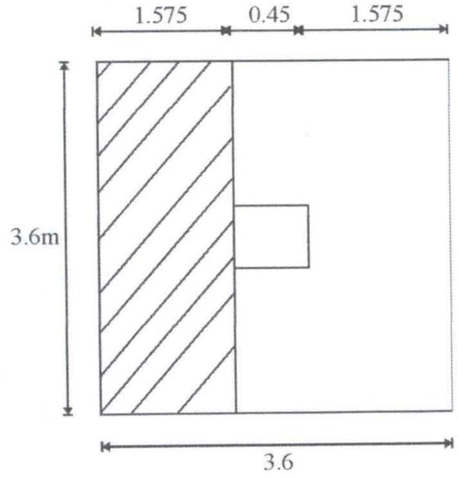
At critical section for shear 1.0d

From the column face

Reference	Calculation	Out put
		
	V = 198.99 x 3.6 x 1.035 = 741.44KN	
	$V = V = \dfrac{74.44\,x\,10^3}{3600\,x\,690} = 0.30$	
	0.30 < 0.39	
	The section ok	Area = 12.31m^2
Reinforcement concrete design by Engr V. Oyenuga 2nd edition page 249	**Base type B2**	h = 600mm d = 540mm
	Self weight of the base will be taken as 10% of the load acting on it.	
	1.47 will be used as factor to convert loads from ultiamate limit state to servicieability limit state	
	Axial load from column C2 = 3074.13KN	
	Total design axial load = 1.0GK + 1.0QK	
	$= \dfrac{3074.13}{1.47} + 10\%\ x\ 3074.13 = 2'–\&:--KN$	
	Required base area = $\dfrac{2461.66}{200} = 12.31m^2$	
	Provide base area of 3.6m.sq	
	Base area = 12.96m^2	
	Column axial load = 2461.66KN/m^2	

Reference	Calculation	Out put
	Earth pressure = 2461.66 = 189.94KN/m^2	
	Assume base thickness of 600mm	
	Depth d = h – c ½ Φ	
	= 600 – 50 – ½ x 20 = 540	
	Stress on footing	
	At column face	
	Shear stress, Vc = N/column perimeter	
	$= \frac{2461.66\ x\ 103}{4(450)\ x\ 540} = 2.53$N/mm	
	2.53N/mm $< 0.8\sqrt{fcu} = 4.0$	
	i.e. 2.53 < 4.0	
	hence share ok	
	punching shear	
	critical perimeter = column perimeter + 8 x 1.5d	
	= 4 x (4500) + 8 x 1.5 x 540	
	= 8280mm	
	Area within perimeter = $(450 + 3d)^2$	M = 848.11KNm
	= 4.28 x 10^2 mm^2	
	Punching shear force V =	
	Earth pressure (Area provided – Area within perimeter)	
	= 189.94 (3.6^2 – 4.28)	
	= 1648.68KN	
BS 8110 Part 1 1997 table 3.8	Punching shear v = $\frac{V}{Critical\ perimeter}$	
	$= \frac{1648.68\ x\ 103}{8280\ x\ 540} = 0.37$	

Reference	Calculation	Out put
	Vc = 0.39 from table hence, punching shear ok	
	Bending reinforcement	
	At column face which is the critical section	K = 0.0323 K < 0.156
	M = (Earth pressure x 3.6 x 1.575) x $\frac{1.575}{2}$	
	= 189.94 x 3.6 1.575 x 0.788	
	= 848.11KNm	Z = 519.56mm



Reference	Calculation	Out put
	$K = \frac{M}{Fcu\ bd^2} = \frac{848.11 \times 10^6}{25 \times 3600 \times 540^2} = 0.0323$	
	$Z = d\left(0.5 + \sqrt{0.25 - \frac{K}{0.9}}\right)$	
	$= 710\left(0.5 + \sqrt{0.25 - \frac{0.0323}{0.9}}\right) = 519.86mm$	
	$As = \frac{M}{0.87fyz} = \frac{848.11 \times 10^6}{0.87 \times 410 \times 519.86} = 4573.65mm^2$	
	Provide = Y25 @ 100$^c/_c$	
Mosley and Bungey page 279 Table 3.8 BS8110:	As prove = 4910mm^2	
	Asmin = 0.13%bh	
	$\frac{0.13\ x\ 3600\ x\ 600}{100} = 2808$	

Reference	Calculation	Out put
part 4 1997	Final punching shear check	

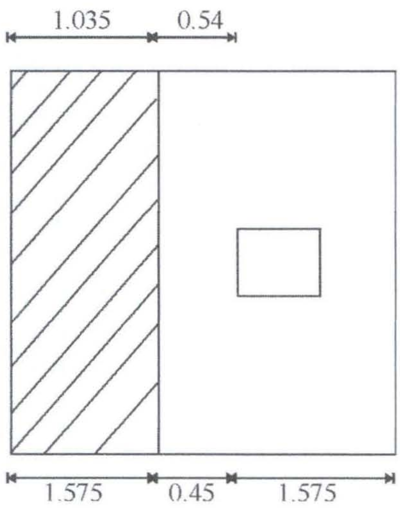
$$\frac{100As}{bd} = \frac{100\ x\ 4910}{3600\ x\ 540}\ 0.25$$

Ultimate Vc = 0.37N/mm^2 from table

Punching shear was 0.39

Hence 600mm thick pad ok

At critical section for shear, 1.0d from the face



V = 189.94 x 3.6 x 1.035 = 707.72KN

$$V = \frac{V}{bd} = \frac{707.72\ x\ 10^3}{3600\ x\ 540}\ 0.36N/mm^2$$

0.36 < 0.4

The section ok

Reference	Calculation	Out put
Reinforce ment concrete design by Engr V. Oyenuga 2nd edition page 255	ANALYSIS AND DESIGN OF COMBINED FOOTING	
	Take factor of 1.47 for ultimate limit state	
	Assume a traverse width of 2.350m	
	Load N on each column = 1258.98KN	
	Total load = (1258.98 + 1258.98) = 2517.96KN at SLS	
	At ULS = 2517.96 x 1.4 = 352514KN	

Reference	Calculation	Out put

Taking moment

From column 1, we have

$$\text{Centroid} = \overline{X} = \frac{1258.98\ x\ 2.765}{2517.96} = 1.883m$$

Centroid from column = 1.883m

And centroid from column 2 = 3.765 – 1.883 = 1.882

R.C by Engr V. Oyenuga page 249

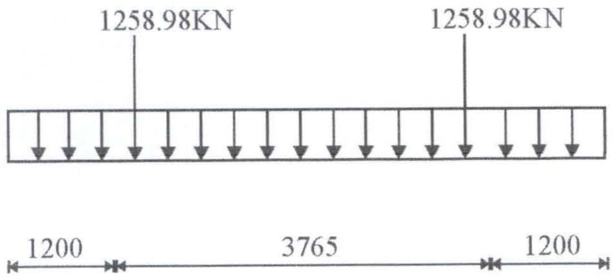
$$\text{Area} = \frac{1.1w}{Pb}$$

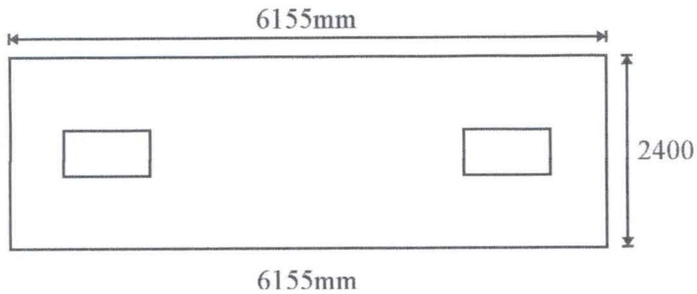
W = total loadd at serviceability state

Pb = soil bearing pressure

$$\text{Area} = \frac{1.1\ x\ 2517.96}{200} = 13.848m^2$$

Taken width as 2400m, the required lenght = 6.165m





Assuming thickness h = 750mm (base)

$$\text{Fnet}\ \frac{3525.14\ x\ 1.1}{6.165\ x\ 2.4} = 262.07KN/m^2$$

This pressure is over the entire base left over = right

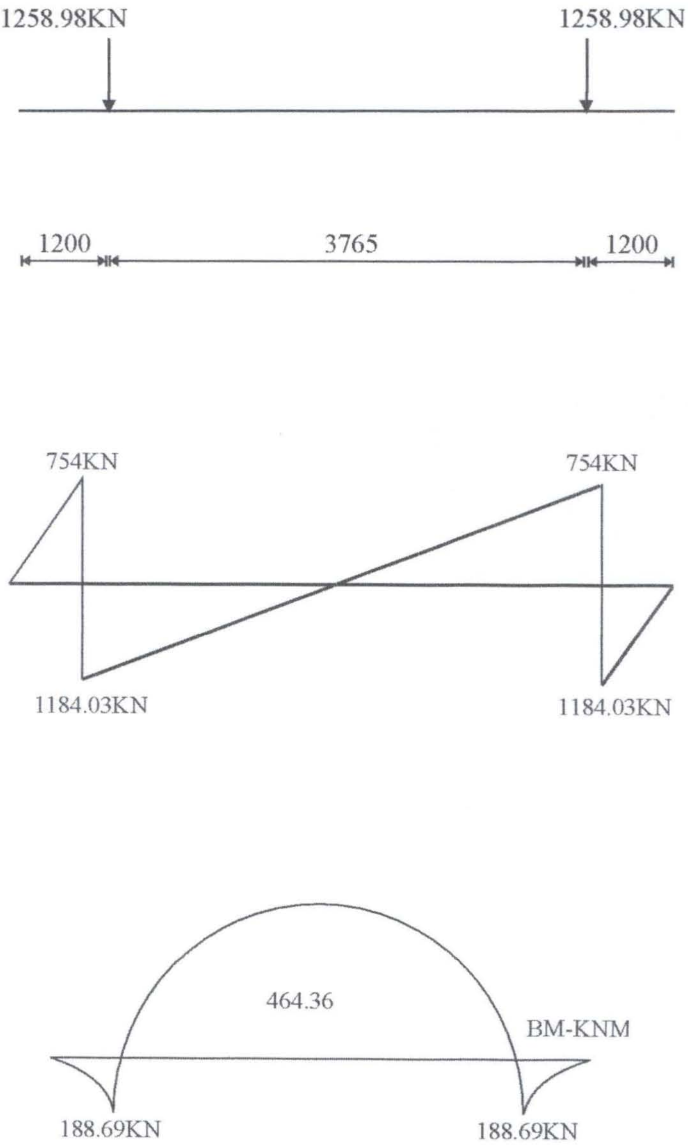
overhang

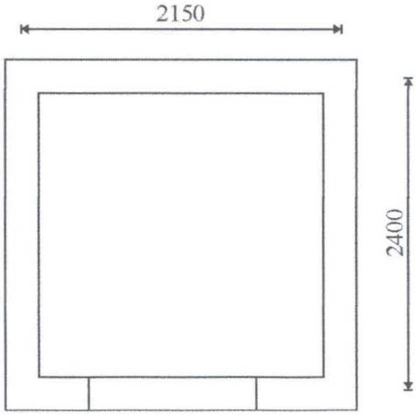
As min = $2340mm^2$

Reference	Calculation	Out put
	$= 1.2^2 \times 0.5 \times 262.07 = 188.69$KNm	
	h = 750mm	
	d = 750 – 50 – 10 = 690mm	
	$K = \dfrac{M}{Fcu\ bd^2} = \dfrac{188.69 \times 10^6}{25 \times 2400 \times 690^2} = 0.006605$	
	Z = 684.86mm	
	$As = \dfrac{188.69 \times 10^6}{0.87 \times 410 \times 684.86} = 768.89mm^2$	
	Minimum reinforcement = 0.13%bh	
	= 0.13% x 2400 x 750	
	$= \dfrac{0.13\ x\ 2400\ x\ 750}{100}$	
	$= 2340mm^2$	
	Provide Y 25 = $200^c/_c$	
	(As prove = $2450mm^2$ Bottom)	
	Span moment	
	Free moment (span) $= 0.125 \times 3.765^2 \times 262.07$	
	= 464.36KNm	
	Approximate span moment =	
	464.36 – 0.5 (188.69 + 188.69) = 275.67KNm	
	$K = \dfrac{275.65 \times 10^6}{25 \times 2400 \times 690^2} = 0.009650$	
	Z = 682.54mm	
	$As = \dfrac{275.67 \times 10^6}{0.87 \times 410 \times 682.54} = 768.89mm^2$	
	Provide Y $16175^c/_c$ top	

Reference	Calculation	Out put
	(As prove = $1150mm^2$)	
	Transverse reinforcement	
	Moment $M = 0.5 \text{ x } \frac{(2.4)2}{2} x\ 262.07 = 188.69KNm$	
	$K = \frac{188.69 \times 10^6}{25 \times 2400 \times 690^2} = 0.00661$	
	Z = 684.93mm	
	$As = \frac{188.69 \times 10^6}{0.87 \times 410 \times 684.93} = 772.32mm^2$	
	Provide Y 16 @ $250^c/_c$ as distribution bars	
	(As prove = $804mm^2$)	
	Shear	
	W = fnet x b = 262.07 x 2.4	
	= 628.97KN/m	
	For span, V_L =	
	$628.97 \text{ x } 0.5 \text{ x } 3.765 + \frac{(118.69 - 118.69)}{3765} = 1184.03KN$	
	Vr =	
	$628.97 \text{ x } 0.5 \text{ x } 3.765 + \frac{118.69 - 118.69}{3.765} = 1184.03KN$	
	Left overhang = Right overhang =	
	628.97 x 1.2 = 754.76KN	

Reference	Calculation	Out put



Reference	Calculation	Out put
	SHEAR WALL DESIGN	
		
	Assumptions	
	Wall thickness = 200mm	
	Height = 4200mm	
	Fcu = 25N/mm	
	Fy = 410mm	
	No. of floors = 7	
	Concrete cover = 40mm	
	Loadings	
BS 6399 part 1: 1997	Dead slab load = 24 x 0.15 x 3.825 = 13.77KN/m	
	Total slab load = 13.77 x 7 = 48.20KN	
	Self weight of wall = 0.20 x 24 (7 x 4.2)	
	= 141.12KN/m	
	Total characteristics dead load d	
	$= 141.12 + 48.2 + \frac{13.77}{2} = 196.21 KN/m$	
	Imposed load perm = 4.5KN/m	
BS 6399: Part 1: 1984	Imposed load from slab = 3.0 x 7 = 21KN/m	
	Total imposed load = 21 + 4.5 = 25.5KN/m	
	Ultimate design load nw = 1.4gk + 1.6qk	

Reference	Calculation	Out put
	$= 1.4 \times 196.21 + 1.6 \times 25.5$	
	$= 315.49 KN/m$	
	$nw = 0.35 \times fcu \times Ac + 0.7\ Ascfy$	
	Ac = area of concrete wall taken as wall thickness x 1m	
	run = 200 x 1000 = 2000,000mm^2	
	nw = 0.35 x 40 x 200,000 + 0.7 (410) Asc	
	$315.49 \times 10^3 = 280{,}000 + 287 Asc$	
	$Asc = \frac{315.49\ x\ 103 - 280{,}000}{287} - 8656.83 mm^2$	
	Asmin = 0.4%bh = 0.4% x 200 x 1000 = 800mm^2	
	Provide Y 12 @ $125^c/_c$ (As prove = 905mm^2)	

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATION

6.1 CONCLUSIONS

The duties of structural engineer is to ensure that the structure design meet its functional requirement. He should also strike balance between safety and economy.

Slabs were designed as one – way slab, two ways spanning slab and other structural member including beams, columns and foundation design.

Though the analysis and design was done to reduce cracks, deflations shear to minimum, ensuring serviceability of structure.

6.2 RECOMMENDATION

It is important to provide adequate cover of concrete over reinforcement for the proper protection to develop the necessary bond between steel and concrete to ensure durability. Also all specifications should be adhered to and supervised reinforcement arrangement as well as concrete mixture placement.

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