STRUCTURAL DESIGN OF SIX STOREY OFFICE COMPLEX IN ABUJA, NIGERIA

BY

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A PROJECT SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY MINNA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF POSTGRADUATE DIPLOMA OF ENGINEERING (pGD) IN CIVIL ENGINEERING DEPARTMENT OF CIVIL ENGINEERING. SCHOOL OF ENGINEERING AND ENGINEERING TECHNOLOGY.

DECLARATION

I hereby declare that this project was wholly and solely written by me under the supervision of Engr. S. F. Oritola of the department of Civil Engineering, Federal University of Technology Minna, Niger State, Nigeria

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CERTIFICATION

This thesis titled "Structur.al Design of Six Storey Office Complex in Abuja, Nigeria" by GBADEB0, Odunmorayo Lawrence (pGD/CE/08052) meets the regulations governing the award of the degree of Postgraduate Diploma of Engineering (pGD) in Civil Engineering of the Federal University of Technology Minna and is approved for its contribution to scientific knowledge and literary presentation.

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DEDICATION

This project is dedicated to Almighty God, my dear wife, Mrs Gbadebo Olukemi and my wonderful children: Aanuoluwa and Oluwaseun.

ACKNOWLEDGEMENTS

I wish to thank those people who have contributed to the success of this project. My sincere thanks goes first to Almighty God, the giver of life, who made it possible for me to run this programme and for his protection and mercy over me throughout the course of my study. My sincere appreciation goes to the Head of Department Engr. Prof S. Sadiku, who makes sure that this knowledge is well articulated and coordinated under a conducive atmosphere and for his fatherly advice at all times. My indebtedness goes to my supervisor, Engr. S.F. Oritola, despite his tight schedule who spent his time reading through the manuscript and making himself available for thorough supervision of this project. My sincere appreciation goes to all my lecturers and staff of the Department of Civil Engineering, Federal University of Technology, Minna Among them are Engr. Prof O. D. Jimoh, Engr. Dr 1 Aguwa, Engr. Dr. Ndoke P.N., Engr. Dr. A. Amadi, Engr. Kolo S.S., Engr. Dr. T. Y. Tsado, Engr. Dr. SM. Auta, Engr. Dr. M Abdullahi, Engr. M. A. Mustapha, Engr. James Olayemi, Engr. R Adesiji, Engr. Busari Afis, Engr. l. Jimoh, Engr. 1 Abdulkadir, Engr. W. T. Adejumo, and a host of others.

Similarly, for bearing with me, I convey my sincere appreciation to my wife, Mrs Gbadebo Olukemi Anthonia, and my daughters, Aanuoluwa and Oluwaseun Gbadebo for their patience, love, care, encouragement at all times and support throughout the course of my study. May the Almighty God reward you bountifully. May I also register my profound appreciation and gratitude to my boss in my place of work Lady Maria Anegbode, the managing director of Anestony Construction Company for her love and understanding through the course of my study and the entire staff of the company. I am also grateful to my numerous friends and colleagues at the Federal University of Technology, Minna, especially to Engr. Alh. N. O. Kareem and family, Engr. Busari Afis, Adesiji Richard and family, Barrister Abiola Jeleel, Efe Omoruyi, El-amin Adam, Ochigbo Clement, Monday Usman, Japhet Akor, Amin, Pius Uduhirinwa, Yusuf M. Habib and a host of others.

Finally, my love and indebtedness goes to my brothers, - Mr and Mrs Y. Adedoyin, Mr and Mrs W. Adedoyin. May the Almighty God bless you all.

ABSTRACT

This project presents structural design of 6-storey office complex at Abuja Metropolis. The area research which is the provision of structural analysis, design and detailing were emphasized. Structural members design includes roof slab, ribbed floor slab, beams, columns, raft slab and ground beam in accordance with the appropriate codes of practices were also emphasized. Limit State Methods of design is adopted for the design of all structural elements and hence appropriate checks including deflection, moment shear and serviceability were carried out.

TABLE OF CONTENT

ABBREVIATONS, GLOSSARIES AND SYMBOLS

As	Cross- sectional area of tension reinforcement
	Cross- sectional area of compression reinforcement
Asprov -	Area of reinforcement provided
Asreq -	Area of reinforcement required
	Area of shear reinforcement
b	Width of section
d	Effective depth of shear reinforcement
	Effective depth of compression reinforcement
	Characteristic concrete strength
Gk	Characteristic dead load
hr	Thickness of flange
L	Clear span of a member
	Effective height of column
les ley -	Effective height in respect of major or minor axis Respectively
	Length of shorter span of a slab panel
l_y -	Length of longer span of a slab panel
M.F	medication factor
Msx,Msy-	Maximum design ultimate moment
N	Axial load on column
N	Designed ultimate load per unit area
Qk	Characteristic imposed load
Sv	Sp[acing oflinks along the member

CHAPTER ONE

1.0INTRODUcnON

The purpose of the project is to give vivid analysis of design and detailing of a six storey office complex with five numbers of two bedrooms flat each at the sixth floor using the limit state method for designing. The understanding of respective concepts and use is made possible, by first knowing the individual properties of materials used which consist mainly of concrete and steel. Combining these two components produce a firm, durable long lasting reinforced concrete structure. Understanding the behaviour of concrete and steel has grown tremendously, over the past decades and is still growing. The application of flexible materials is used in construction so also new and improved techniques to bring out amazing structure.

Before civilization, use of building codes for design and detailing had not yet been established. It was a time of which different races did what they deem fit in building construction. Therefore one of the main purposes of design is that the entire structure will be designed using reinforced concrete in accordance with British standard (BS) 8100.

Structure advancement is paramount resulting in quick, efficient and interesting ways to building construction, with these advancements, the raising of multi-complex building was born and is built for a variety of reasons for instances, they are used for administrative purposes, as monumental structures and also help curb population explosion which is part of the main objectives of these design. Population explosion can be brought about by a variety of reasons such as rural to urban drift, commercialization, and industrializationjust to mention a few. Therefore the main purpose of this design is to carry loads, stress from roof slab tops, from floor slab, beams, partition walls, shear wall, after which these loads are transferred to foundation and distributed to the sill.

this design is to carry loads, stress from roof slab tops, from floor slab, beams, partition walls, shear wall, after which these loads are transferred to foundation and distributed to the sill.

1.1- LOCATIONOF SITE

The reinforced concrete six storey building is to be located at FCT Abuja, a proposed office complex. The building will be located in an area where population is at its minimal.

1.2 CAPACITYOF OFFICE COMPLEX

The office complex is made up of three sections labeled A, B, and C. the first part which is A, the basement floor which consist of the following: The cars parked, maintenance office, lift shaft, janitor, mechanical and electrical room. The second part, which is B, is the first floor to the fifth floor. Also section B comprises of one lift and two stair case region, glazed minimum on each floor. At the stair region, glazed windows and lights provide for both natural and artificial lighting. The building has two entrances, the main entrance and the second entrance at the left side of the building. The basement has a ramp for easy ingress and egress of vehicle. The C section which is sixth floor consist of five number of two bed room flats each.

1.3 AIMAND OBJECTIVES

The aim of this project is to design a six storey office complex with basement that will be able to stand the test of time. The specific objectives are to:

(i) To design a structurally stable six storey building in Abuja

(ii) To produce a structure which is economical to construct, maintain and service throughout its design life

(ii) To produce a structure which is economical to construct, maintain and service throughout its design life

(iii) To add to the aesthetic value of Abuja

(iv) To promote the economic development of the Federal Capital Territory

1.4 SCOPE OF WORKS

The scope of the project entails designing and detailing of the structural elements which include roof slab, beams, columns, staircase, raft slab, foundation and ground beam. These are first done by first calculating the lateral wind loads analysis, mostly for the unbraced frame. This is done by bringing out the whole building structure inform of a frames and analyzing each section to obtain the bending moment, shear forces and normal stress on each section of the frame. Various methods are used in calculating moments such as stiffness flexibility method, slope deflection, moment distribution and computer method. The method that is appropriate for vertical load analysis is the moment distribution method, because it can be applied to both plastic and semi plastic materials it can also be used to analyze complex structure and is relatively cheap. Slabs are designed mainly for bending and in few cases for shear. Next is the analysis of the roofbeam, ribbed floor slab and floor beam. After which we then design for columns. All loads from the entire structure from the roof is collected by the columns then transferred to the raft foundation. Design of columns is mainly for axial forces but in few cases for moments. Lastly, the foundation is designed; foundations relieve all loads from the columns and transferred it to the soil. It is primarily design for bending and shear.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Historical Background

Civil Engineering is perhaps the broadest of all engineering fields dealing with the creation, improvement and protection of the communal environment, providing facilities for living, industry and transportation, including large building, roads, bridges, canals, soil road lines, airports, water supply systems just to mention a few. It was stated before civilization by the construction or creation of structures, amenities to take care of man immediate wants to the satisfaction of his design. Ancient civilization engages in building of enormous projects or super structures some of which are the china wall (great wall), built to protect china's northern border in the 3rd century B.C, Egyptians pyramids at around 5000 B, C and even the coliseum of the Romans all of which are historical landmarks, or wonders pacing us into the future to the creation of amazing structure for instance the Oklahoma highways (Encarta, 2007).

2.2 Properties of Reinforced Concrete

The understanding of the individual properties of concrete and steel has lead to further improvement and use, improving on loop holes which boost efficiency. Reinforced concrete is a strong durable building material that can be made to form into various shapes and sizes ranging from simple rectangular columns to a slender curved dome or steel. Best quality of concrete and steel is attained by their combination (Oyenuga, 1999)

Table 1.1: Individual Properties of Concrete and Steel

CONCRETE	STEEL
(1) Concrete has a high compressive strength but low in tensile strength.	(1) Steel, has much compressive strength as tensile strength.
(2) Concrete's mole durable, considerable crushing strength.	(2) In the case of steel it has to be protected against corrosion.
(3) Concrete is very workable, in that it can *be* worked into different shapes and SIzes.	(3) Steel is not so workable.
(4) Concrete has a good SIze resistant nature.	(4) **It** has a poor resistance suffers rapidly in strength at high temperatures.
(5) Concrete is fair in shear.	(5) Steel is very good in shear.

Thus a combination of both properties results in both good compressive and strength, durability and a good resistance to tire and shear.

2.3 Design Codes and Standards

Basic codes are taken from Bs 8110-part 1 and part 2. 1985. Also the use of examples of the design of Reinforce concrete building to Bs 8110 fourth Edition by Charles E. Reynolds and James C Steedman, Reinforced concrete design by W.H.M and J.H. Bungey and simplified Reinforced concrete design by Victor O. Oyenuga.

2.4 Structural Analysis

Structural Analysis deals with the componetic separation and close examination of a reinforced concrete structures. A reinforced concrete structure is made up of beams, columns, slabs and walls that are rigidly connected together. Each member must be able to resist forces acting on it. So determination of these forces is of importance to the design process. First analysis is carried out by the evaluation of all loads carried by the structure, including self-weight. It is said that as a result of variability in magnitude of most loads and position, all possible critical arrangements of loads must be considered. Determination of forces in each member can be determined by the following methods:

1. Apply moment and shear co-efficient
11. Manual Calculation
111. Computer methods

Designing of a reinforced concrete member is based generally on the ultimate limit state, for the analysis ofloads. Loads on a structure are in two types namely:

~ Dead loads

~ Imposed loads

2.4.1 Dead Loads

These comprise of the weight of the structure all architectural components such as ceiling, exterior cladding and partitions. In addition, equipments and static machines, permanent fixtures are often considered. Dead load is calculated as the product of the specific weight and the volume of the structure for example concrete specific weight is 24 kNIM3 and a beam of 450 x 225 mm cross section will have the own weight as (0.450 x 0.225 x 24KNIM). A higher density should be taken for heavily reinforced or dense concretes.

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2.4.2 Imposed Load

These loads are more difficult to determine accurately, fur some, load determining is based only on conservative estimates using standard codes of practice or post experience. Example of such are, the weight of the occupants, furniture's, machinery, the pressures of wind, the weight of snow that is in cold regions, and of retained earth or water forces caused by thermal expansion or shrinkage of the concrete. Wind load is an imposed load kept in a separate category when its partial safely. Factor are specified, and the load combinatory on the structure being considered.

2.4.2 Load Combinations

Load combination for ultimate state. The loads are split and multiplied with adequate factors and summed to give the total load on the structure. The different combination of loadings are shown below

(a) Values for the ultimate limit state

I.4~ + I. 6Qt (for design dead and imposed load)

0.9~ + I.4Wk (for design dead and wind loads)

1.2~ + 2Qt + 1.2Wk (for design dead, imposed and wind loads)

(b) Values for serviceability limit state

1.0~+ 1.0~(for design dead and imposed load)

1.0~+ 1.0Wk(for design dead and imposed load)

1.0~ + 0.8 (Qt + Wk) - (for design dead, imposed and wind loads)

In assessing the effect these loads on the structures as a whole, or on any part or section of the structure, the arrangement of the loads should be such as to cause the most severe stresses.

2.5 Available Stress Conditions

(1) Bending:- The member will deflect a certain amount below the support level and stresses setup

(2) Buckling:- This is a situation were by a slender body suddenly bow or buckle in some un predetermined direction when acted upon by an increasing axial load, under excessive compression the resistance of the member to compressive force will no longer be effective in the case of columns. In general, the body experiences buckling when the plane least able to resist it in the case of columns the plane dimension is critical while in beam the breadth becomes critical.

To eliminate bow or buckling effect, a stress reduction is made in the concrete and the compressive steel above a certain slenderness ratio of the section members that are mainly subjected to buckling are columns, beams and bearing walls under compression.

(3) Stretching:- This stress condition occurs mostly in members of home work which act as ties. Stretching of reinforced concrete members is caused mainly by tension stress which can be taken care of by reinforcement base which is property protected by hook and covered by enough concrete cover.

(4) Twisting:- Twisting is a situation brought about by opposing forces or eccentric loading of the member. Resulting to twist and distortion of the member when this occurs beam will be incapable of resisting buckling and bending. This situation is common with two span slab supported at its

four edge. This stress condition can be prevented by the stiffening of members by increasing the size of the member or increasing the main and link reinforcement.

(5) Shear Stress:- This stress condition is as a result of two member part of a point lending to move in opposite to each other. It is brought about as a result of loading of one part and the resistance to the failure of the loaded part which is critical at supports and in some instance more critical over the entire span. The shear force tries to pull the reinforcement away from the support bringing about bond failure. These effect is of a higher magnitude where by a member is supported by a column. Shear force is taken on the concrete initially but in the case where force is greater than the resistance of the concrete shear reinforcement is provided in form of stress up or crank.

(6) Deftection:- This is the shifting or change in slope of a structural member subjected to loading. Deflection if not considered adequately affect adversely the slope of service and the safety and comfort of the occupants. Using the B.S code 8110, which says that the deformation of the structure or any part of it should not adversely affect its efficiency or appearance. Deflections should be compatible with degree of movement acceptable by other elements including finishes, services, partitions, glazing and cladding in some cases a degree of repair work or fixing adjustment to such elements may be acceptable where specific attention is required to limit deflections to particular values, reference should be made to 3.2 ofB.S 8119-2-1985, otherwise it will generally be satisfactory to use the span/effective depth ratios given in section 3 for reinforced concrete.

Types of Deflections

(i) Long time deflection and

(ii) Short time deflection

Under long-term deflection, creep and shrinking are the main causes. Creep is a slow deformation exhibited by concrete under sustained stress and proceeds at a decreasing rate over years. Creep increases or decreases under applied stress. The effect of stress is not often considered in reinforced concrete design but taken into account when calculating deflections. Shrinkage is also a long term whose effect is not occur at the early stage of the concrete, it occurs when concrete is hardening. Curvature is due to applied moments. Creep and Shrinkage may result to damage or collapse of the structure ifnot adequately checked.

Short-term deflection occurs after the concrete is cast. It can be easily identified when the project is still under construction and prevented to avoid its being excessive.

(7) Cracking: - This occurs when the stress induced on concrete is high or due to vibration of the structure. Cracking may also occur because of unequal settlement of column bases or poor handling of concrete in its plastic state. Excessive cracking leads to corrosion of the reinforcement's materials and deterioration of structure, which can later lead to collapse of building Control is by the limitation of reinforcement spacing and amount of concrete cover. Maximum spacing of bars can be ignored provided that it can be shown by calculation that the resulting maximum crack width does not exceed the limiting value of 0.3 millimeters (rom) permitted by B.S 8110. From above arbitrary rules described have been designed that excessive cracking is prevented in the extreme practical conditions. A suitable mathematical procedure for calculating crack which is described in exceed 0.8, FylEs design surface crack width 3acrEmil + 1 (lict-Cmin)lh-x)Where 8cr is the distance between the point on the surface at which the crack width

CHAPTER THREE

3.0 METHODOLOGY AND DESIGN

These are three basic design methods used to solve the problem of structural safety and adequate serviceability in remforced concrete which has gone through various modification these are.

3.1 THE MODULAR RATIO METHOD

Also called elastic theory loads are assessed as the actual loads but limiting permissible stresses in the concrete and its reinforcements to a fraction of their actual stresses in order to provide an adequate factor safety. Guided by cp 114, 1957.

It is assumed that both concrete and reinforcements relationships between strain is linear (i.e. materials behave perfectly elastically) the distribution of strain across a section is also assumed to be linear (allowable stress method) the ratio of the stresses in the materials depends largely on the ratio of the elastic module of steel and concrete used to design water retaining structures.

Triangular stress distribution ranges from zero at the neutral axis to a maximum at the compression or tension face.

Mathematically:

Load Factor = Ultimate Strength

Working Load

3.3 LIMIT STATE DESIGN METHOD

This is the modification which overcome the previous ones stated above, because In addition to emphasizing adequate strength, satisfactory performance under service conditions is ensured. The looking loads are multiplied by partial factors of safety and the ultimate materials strengths are divided by further partials factors of.safety. This method ensures that each individual member will be obtained. All that been said is that when designing in accordance will limit state principles as embodied in CP 100 and similar documents like Bs 8110 part I and 2 (1997) each reinforced concrete section is designed first to meet the most critical limit state and then cheeked to ensure that the remaining limit states are not reached.

Critical conditions as regards the ultimate limit state conditions of failure are usually considered for these project and so therefore the limit state method will be adopted as it is the method with little or no short comings and can also be termed an aggregate of the other two methods or section of members must satisfy two separate criteria of

~ The limit state which ensures that the probability of failure is acceptably low

~ The limit of serviceability which ensures safe factory behaviours under service working loads. Principal criteria relating to serviceability are prevention of excessive cracking and under special circumstances and certain types of structure. Other limit state standards fatigue, vibration, durability, fire resistance at all, may also have to be considered.

- By breaking a little further the characteristic loads are multiplied by a partial safety factor for loads, "At' to obtain design loads enabling calculation of the bending moments and shear forces for which the members to be designed.
- Therefore, it characteristic loads multiplied by value of AI (partial safety factor) corresponding to the ultimate limit state, moments, and forces determined afterwards win represent those occurring at facture resulting in adequate design of the sections. Also when value AI corresponding to the limit state of serviceability is used, moments, forces under service loads.

REF	CALCULATIONS		OUTPUT
4.1	ROOF SLAB DESIGN		
	Durability and fire Resistance		
BS8110	Condition of exposure = Mild		
	Required fire resistance = 1 hour		
Table 3.2	Nominal cover to meet durability = 20mm		Cover = 20mm
Table 3.4	Nominal cover meet fire resistance = 20mm		Fire resistance Ok
Table 3.5			
	Assumptions:	b= 1000mm	
	Feu = 2S N/mm2	h= lSOmm	
	Fy=410N/mm2	d = IS0-20-12/6 = 124mm	
	Cc=2Smm		
	Panel		
	4m	Ly=4	
		Lx=4	
		4m Ly/Lx = 1.0	2 way slab
	2 edges continuous		
Reynolds			
	Dead Load		
Table 2	Self weight of slab IS0mm x 24	=3.6kN/m2	
Table 4	Partition (Light)	= 1.0 kN/m2	
	Finishes (felting)	= I.S kN/m2	
	Other (services)	=0.5 kN/m2	
	Total dead Load	=6,6 kN/ml	
Table 6	Live load or impose load	= 1.5 kN/m2	
Table 1	Design load (n)	= 1.4gk + 1.6gk	
		= 1.4 x 6.6 + 1.6 xl .S	
		=9.24+2.4	
		n = 11.64 kN/m2	n = 11.64 kN/m2
	ULTIMATE MOMENTS		
Table 3.15	Use simplified load analysis		
	Msx = Bsxn Lx2		
	Msy = J3sxn Lx2		
BSBllO	SHORT SPAN		
Equation 1	At edges Mx (-ve) 0.047 x 11.64 x 42		8.7SkN/m
Equation 1	At Mid span (+ve) 0.036 x 11.64 x 42		6.71kN/m
	CONCRETE AND STEEL STRESSES		
	Characteristic strength of concrete fcu-2SNm2		
	Characteristic strength of steel = 410 Nmm2		

REF	CALCULATIONS	OUTPUPI'
	REINFORCEMENT Assume the use of minimum of 12mm Diameter bars in direction of short span d = 150 - 20 _12/2 = 124mm	
k= M	=0.023 1000 x 1242 x 25 k = 0.023 <0.156	k=0.023
	Section simply reinforced and no compression steel is required Z = d (0.51$25 - k/0.9) s, 0.95d Z = d (0.5 + 0.25 - 0.023) 0.9 Z = 0.974d> 0.95d	
	Hence use 0.959 Z = 0.95 x 124mm Z 117.Smm	Z= 117.Smm
	As req = Msx = S.75xl06 0.S7fy z 0.S7x41Ox1l7.S =20Smm2 Provide T12@1 00et, (As prov = *393mm2/m)*	As req=70Smm2 Asprov=I,130mm2
	Moment, Msx (tvc) = 6.71 kNm Size of reinforcement = 12mm Effective depth of slab = 124mm Used lever arm factor = 0.95 At *mid* span K= 6.71xl06 = 0.017 < 0.156 1000x124~25 As req = Msx = 16lmm2 0.S7x410x1l7 Provide TI0@250o/e (As porv = 314mm2)	k=0.017 Asreq= 161mm2 Aprov=314mm2
	Minimum Steel Area As min = O.13bh = 0.13 x 103x150 100 100 = 195mm2 Provide TI 0@250 ct, (As prov 314mm2)	Asreq= 195mm2 Asprov=Sl amnr'

REF	CALCULATIONS	OUTPUT
	DEFLECfION	
Bs 8110	Service stress, fs = fy xAsreq x 1	
Part 1	Aspov ß	
Table 3.11	Where ß is the ratio of mid span moment other and before of any redistribution.	
	= 5/8 x 410 x 161 x], = 181 N/mm^2	
	314 1	
	SPAN EFFECTIVE DEPfH RATIO	
	M = 6.71 x 10^6 = 0.45 N/mm^2	
	Bd^2 1000x124^2	
	M.F = 0.55 + (477.1$)	
	120 (0.9 -ml	
	bd	
	= 0.55 + (477 - 131)	
	120 (0.9+0.45)	
	M.F=2.7	
	Basic effective span ration = 26	
	Limiting span = 26x2.7 = 70.2	
	Actual span = 4000 = 32.26	
	124	
	32.26 < 70.2 Deflection OK	
	LONG SPAN	
BS8110	At edges my-Ve = 0.045 x 11.64 x 42	8.38kNm
	At mid span mys + Vc = 0.034 x 11.64 x 42	6.33kNm
	d = 150 - 20 -12 _loh = 113mm	
	= 8.38 x 10^6 = 0.026	
	10^3xl13^2x25	
	k = 0.026 < 0.156	
	la= 0.95	
	Z = 0.95 x 113 = 107.4mm	
	As req = 8.38x10^6	
	0.87x41Ox107	
	As=220mm^2	As req = 220mm^2
	Provide T10@300mmc/c(Asprov = 262mm1	Asprov=262mm^2
	As minimum = 0.13 x 10^3x150	
	100	
	= 195mm	

REF	CALCULATIONS	OUTPUT
	At Mid Span	
	Msy=6.33	
	K=Msy = 6.33 x 10^6 = 0.020	
	$bd^2 f_{cu}$ 1000x113^2x25	k= 0.020
	k = 0.020 < 0.156	
	la =0.95	
	Z = 0.95 x 113 = 107	
	As req = 6.33 x 10^6	
	08n x 416 x 107	
	As= 165mm^2	Asreq= 165mm^2
	Provide T10@300mmc/c	Asreq=262 mm^2
	(as prov = 262mm^2)	
	As minimum = 0.13 x 10^3 x150 = 195mm^2	
	100	
	Provide T10@25Ommc/c	Asreq= 195mm^2
	As prov = 314mm^2	Asprov=Sl amrrr'
	PANEL 2	
	1y=8	
	:4m lx=4	
	ly/lx=2.o	2 way slab
Reynolds	LOADING	
	Dead load	
Table 2	Self weight of slab 150mm x 24 = *3.6kN/m2*	
Table 4	Partitions. = *1.0kN/m2*	
	Finishes = *1.5kN/m2*	
	Other (service) = *0.5kN/m2*	
	6.6kN/m2	
Table 6	Live load = *1.5kN/m2*	
	= 1.4x6.6+1.6x1.5	
	n = 11.64kN/m^2	*n=11.64kN/m^2*

REF	CALCULATIONS	OUTPUT
	ULTIMATE MOMENT	
BS8110	use simplified load analysis	
Table 3.15	Msx = Bsx n lx^2	
	Msy = J3sy n }x2	
	SHORT SPAN	
	At edges Mx (-ve) = $0.063x4^2xl1.64$	11.73kNm
	At mid span Msn (+ve) = 0.048x4~11.64	12.41kNm
	k = M / bd^2fin = 11.73xl06 / 1000xI24:ZU5	k=0.031
	k = 0.031 < 0.156	
	la= 0.95	
	As req = Mx / 0.87fy 2 = 11.73xl06 / 0.87x41Ox117 = $279mm^2$	
	Provide TI0@200c/c	As req $279mm^2$
	(As prov = $393mm^2$)	As prov=$393mm^2$
	AT MID SPAN	
	MomentMsx +vc) = 12.41knm	
	Effective depth of slab = 124mm	
	Size of reinforcement = 12mm	
	Used lever arm factor = 0.95	
	K=M / Bd^2fcu = 12.41xl06 / $10^3x\ 124^2x25$ = 0.032	k= 0.032
	k = 0.032 < 0.156	
	Z = 0.95 x 124 = 117.0	
	As req =Mx / 087fy 2	
	= 12.41xl06 / 0.87 x 410 x 117.0	
	= $297mm^2$	Asreq= $229mm^2$
	Provide TI0@200c/c	Asprov=$393mm^2$
	(As prov = 393)	

REF	CALCULATIONS	OUTPUT
	LONG SPAN	
	At edges My (-ve) = 0.032 = 11.64x42	5.96knrn
	At mid span Msy +ve) = 0.024 x 11.64x42	4.47knm
	K=M =5.96xl06	
	Bd2fc 103xl132x25	
	K = 0.019 < k = 0.156	
	La= 0.95	
	Z = 0.95xl13	
	Z= 107	
	My	
	0.87fy 2	
	As req = 5.96 x 106	
	0.87x410xl06	
	=358mm2	Asreq= 358mrn2
	Provide TI0@150c/c	Aprov=523mm2
	(As prov 523mrn)	
	As minimum = 0.13xl03x150 = 195mrn2	
	100	Asreq=195 mm2
	Provide TI0@300rnrnc/c	As prov=262mm2
	As prov = 262rnm2	
	At mid span	
	k=M = 4.47x106 = 0.0140	
	ba2fcm 103xl13~5	
	k = 0.014 < k = 0.156	
	la = 0.95	
	Z = 0.95 x 113	
	Z = 107	
	As req =Msy = 4.47 x 106	
	0.87fyz 103x41Ox25	
	= 118mm2	
	Provide TI0@200c/c	As req= 118mm2
	(As prov = 314mm)	
	Asprov=374rnm2	
	DEFLECTION	
	Service stress fs = *5/8* x fy x As req x 1	
	As provide ~	
	5/8 x 410 x 295 = 240n!mm	
	314	

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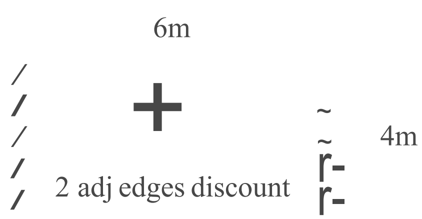
REF	CALCULATIONS	OUTPUT
	SPAN - EFFECfIVE DEPT	
	$\frac{M}{bd^2} = \frac{12.41 \times 10^6}{10^3 \times 124^2} = 0.81$	
	MF = 0.55 + $\frac{(477 - 8s)}{120\ (0.9 + m/bi)}$	
	M= 0.55 + $\frac{(477 - 240)}{120\ (0.7+0.81)}$	
	Mf= 1.70	
	Basic effective span ratio = 26	
	Limiting span = 26xl.7 = 44.6	
	Actual span = $\frac{4000}{124}$ = 32.26	
	32.26 < 44.6 detection ok	Detection Ok
	Durability and fire Resistance	
B8110	Condition of Exposure = Mild	
Table 3.2	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 20mm	Cover=20mm
-... Table 3.5		



REF	CALCULATIONS			OUTPUT
Reynolds	LOADING			
Table	Self weight of slab 150x24 = 3.6 kN/m^2			
	Partition (Light)	=	1.OkN/m^2	
	Finishes	=	1.5kN/m^2	
	Others (service)	=	0.51cN/m^2	
		=	6.6kN/m^2	
Table 6	Live load.		1.5kN/m^2	
	Design load (n)	=	1.4gk + l.6qk	
		=	6.6 x 1.4 + 1.6 x 1.5	
		=	11.64kN/m2	

REF	CALCULATIONS	.OUTPUT
	ULTIMATE MOMENT	
Bs 8110	Use simplified load analysis	
Table	Msx = I3sx n Lx2	
3.l5	Msy = I3sy n Lx2	
	SBORTSPAN	
	At edges Mx (-ve) = 0.078 x 11.64 x 42	14.53kNfm2
	At mid span Msx +ve) = 0.059 x 1l.64 x 42	11.00kN/m2
	K=M = 14.53 x 106 =0.037	k=0,037
	Bd2fcu 103x124z,u5	
	0.037 < 0.156	
	As req = Mx = 14.53x106	
	0.87fyz 0.87x410xl17	
	= 346mm2	Asreq= 346mm2
	Provide T 12 @ 200 %	
		Asprov 566mm2
	AT MID SPAN	
	Moment Msx (+ve = 11.00kN/m2	
	Size of reinforcement = 12mm	
	Effective depth of slab = 124mm	
	k - value = 0.037	
	Lever arm factor = 0.967	
	Used lever arm factor = 0.95	
	Z = 0.95 x	
	Area of steel = 11,00 x 106	
	0.87 x410 x 117	
	262mm2	Asreq =262mm2
	Provide T10 @ 250 't,	Aprov=314mm2
	(As prov = 314mm)	
	LONG SPAN	
	At edges My (-ve) = 0.045x1l.64 x 42	*8.38kN/m2*
	At mid span Msy (+ve) = 0.034 x 1l.64 x 42	*6.33kN/m2*
	K = M = ~8.~3~8x~1~0~6	
	Bd2fin 103xl 122x25	
	= 6.026 < 0.l56	
	Used lever arm factor = 0.950	
	0.95x113 = 107mm	
	As req =My = 8.38 x 106	
	0.87fyz 0.87x41Ox107	
	As = 220mm2	
	Provide T 10 @250 't,	As req =220
	(As prov = 314mm)	Asprov=Hdmnr'

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REF	CALCULATIONS	OUTPUT
	AT MID SPAN	
	$k = \frac{My}{bd^2fcn} = \frac{6.33x10^6}{103x113\sim5} = 0.02$	
	k = 0.02 < 0.156	
	Z = 0.95x113 = 107	
	As req $= \frac{Msy}{0.87fyz} = \frac{6.33x10^6}{0.87x41Ox107}$	
	= 166mm^2	
	Provide T 10 @ 250	Asreq= 165mm^2
	(As prov = 314mm^2)	Asprov=Hdmm'
	DEFLECTION	
	Service stress fs = $\frac{.5/sx\ 410\ x\ 262}{314}$ = *214n1mm*	
	SPAN EFFECTIVE DEPOSIT	
	$\frac{M=11.0x10^6}{bd^2\ \ 10sx124^2}$ =0.71	
	M.F = 0.55 + $\frac{(477 - 214)}{120\ (0.9+0.71)}$	
-_	M.F = 1.9	
	Minimum steel area	
	$\frac{0.13x10oox150}{100}$	
	Provide T10 @ 300mm c/c	
		As req=195mm^2
		Aprov=zezmrrr'

REF	CALCULATIONS	OUTPUT
	4.2 RIBBED FLOOR DESIGN (TYPICAL FLOOR)	
Bs SIlO	Condition of exposure = mild	
Table 3.2	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 20mm	cover=20mm
Table 3.4	Nominal cover to meet fire resistance	
Table 3.5		
	Panel "A" 4m Span	

Sm

4m   Iy= S
lx=4
LY/lx= 2.0

Reynolds	Loading.- per rib ofO.4Sm centre to center		
	Dead load		
	Self weight of topping SO=O.OSXO.4X24	*= 0.S064KN/m2*	
Table 2	Rib = 0.12SxO.20x24xl.4	*=0.84KN/m2*	
	Finishes = 0.4Sxl.5xl.40	= I.00S	
Table 4	Partition allowance = 0.4Sx2x1.40	= 1.344	
	Block, Say I.Ox1.40	= 1.4	
		5.39	
Table 6	Live load = 3 x 0.48 x 1.6	= 2.304	
	Ultimate load = 5.4+2.304	*= 7.704KN/m2*	

ULTIMATE MOMENT

Ultimate moment $M = \frac{nlc^2}{S} = \frac{7.704 \times 4^2}{S}$

$= 15.41 kN/m^2$

Width of rib = 125mm
Width of flange = 0.2 x 4000 + 125 = 925mm
Rib spacing = 4S0mm
Span = 4000 + 225 = 4225mm

CONCRETE AND STEEL STRESSES
Fcu = 25 N/mm^2
Fy = $410 N/mm^2$

MAIN REINFORCEMENT

Maximum lever arm factor	= 0.95
Maximum Ie - value	= 0.156
Moment msx (-ve)	= 15.41kN/m
Size of reinforcement	= 12mm
Effective depth = 250-20-6	= 224mm

REF	CALCULATIONS	OUTPUT
	k = M = 15.41xl06 / bd2fcn 1000x224~5	
	k=0.12<0.156	
	Used lever arm factor = 0.95	
	Z = 0.95 x 224 = 213mm	
	As req = 15.41xl06 / 0.87x410x213	
	As =M / 0.87fyz	
	=203mm2	As req = 203mm2
	Provide 2nos 12mm dig. Barslrib	prov=226mm2
	SHEAR FORCE	
	V = 0.5 x 15.41 x 4.225 = 32.55kn	
	V = 32.55x103 / 224x150 = *0.97N/mm*	
	100 As / bd = 100x226 / 150x224 = 0.61	
	and VC = 0.66	
	Hence, provide minimum shear links RB 150 %0	
	DEFLECfION	
	Service stress fs = *5/8* x fy x As reg / As prov = 1 / J3	
	5/8 x 410 x 203 / 226 = *230N/mm*	
	SPAN EFFECfIVE DEPTH	
	M / bd2 = 15.41 x 106 / 150x2242	
	2.04	
	M.F = 0.55 + (477 - 230) / 120 (0.9+2.04)	
	M.F= 1.25	
	Basic effective span ratio = 26	
	- 1.25x26= 32.50	
	Actual span / Effective depth = 4225 / 224mm = 18.86	
	18.86 < 32.50 Deflection ok	
	Topping: Provide BRC Mesh.	

REF	CALCULATIONS	OUTPUT
	PART OF RIBBED FLOOR DESIGN (Typical Floor)	
	Panel 'B' 2.667m span.	
	4m	
	2.667 ly=4	
	lx = 2.667	
	Ly/lx = 1.5	
		2 way slab
	Loading per rib of0.48m centre to centre	
Reynolds	DEAD LOAD	
	Self weight of topping = 0.5 x 0.48 x 24 x 1.4 =*0.8064kN/m*	
Table 2	Rib = 0.125 x 0.20 x 24 x 1.4 = 0.84	
Table 4	Finishes = 0.48 x 1.5 x 1.40 = 1.008	
	Partition allowance = 0.48 x 2 x 1.4 = 1.344	
	Block say = 1.0 x 1.4 = 1.4	
	5.4	
Table 6	Live Load = 3 x 0.48 x 1.6 = 2.304	
	Ultimate Load = 5.4 + 2.304	
	(n)> 7.704	
Bs 8110	ULTIMATE MOMENT	
	M=nlx2/8	
	$=\frac{7.7x2.667^2}{8}$	
	=6.9kNm	
	Width of rib = 125mm	
	Width of flange = 0.2 x 2662 + 125 = 658.4mm	
	Rib spacing = 480mm	
	Span = 2667 + 225 = 2892	
	CONCRETE AND STEEL STRESSES	
	Feu = *25N/mm2*	
	Fy = 41*ON/mm2*	
	MAIN REFORCEMENT	
	Maximum level arm factor = 0.95	
	Maximum k - value = 0.156	
	MomentMsx (-ve) = 6.9kNm	
	Effective depth of slab = 250 - 20 - 5 = 225mm	
	$k = \frac{M}{bd^2fcn} = \frac{6.9x10^6}{10^3x225^2x25}$	k = 0.019
	k = 0.019	

REF	CALCULATIONS	OUTPUT
	Used lever arm factor = 0.95	
	Z = 0.95 x 225 = 214mm	
	As req =M / 0.87fyz =6.9x10^6 / 0.87x410x214	As req = 9Omm2
	As = 90mm^2	
	Provide 2 nos 10mm dig. Ber/rib	As prov=l Sxmrr'
	(As prov = 158mm^2)	
	SHEAR FORCE	
	V = 0.5 x 15.41 x 2.667 =20.55kn	
	V = 20.55xl0^3 / 224 x 150 =0.61N/mm	
	100 As / bd = 100x90 / 225 x 150 =0.27	
	VC= 0.48 V > VC	
	Provide minimum sheer link B@ 225%	
	DEFLECfION	
	Service stress Fs = 5/8 x fy x As reg / As prov = 1 J3	
	5/8 x 410 x 90 / 158mm^2 = 145N/mm^2	
	SPAN EFFECfIVE DEPTH	
	M / bd^2 = 6.9x10^6 / 150x224^2 = 0.92	
	M.F = 0.55 + (477 -145) / 120 (0.9+0.92)	
	M.F=2.0	
	Basic effective span ratio = 26	
	=2x26= 52	
	Actual span = 2892 = 12.8	
	Elective depth = 225	
	12.8 < 52 deflection ok	
	Topping: provide BRC mesh.	

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REF | CALCULATIONS | OUTPUT

PART OF RIBBED FLOOR DESIGN
(fYPICAL FLOOR)

Panel 'C' 2m span
4m

ly =4
lx2
ly/lx = 2.0 — 2 way slab

Loading per rib of 0.48m centre to centre

Reynolds Table 2, Table 4

DEAD LOAD

Self weight of topping = 0.5 x 0.48 x 24 x 1.4 = 0.8064kn/m

Rib = 0.125 x 0.20 x 24 x 1.4 = *O.84kN/m*

Finishes = 0.48 x 1.5 x 1.40 = I.OO8kN/m

Partition allowance = 0.48 x 2 x 1.4 = *1.344kN/m*

Block say = 1.0 x 1.4 = *1.4kN/m*

5.4kN/m

Live load = 3.0.48 x 1.6 = 2.304

.:. Ultimate load = 5.4 + 2.304
(N= *7.74kn/m)*

ULTIMATE MOMENT

Ultimate moment M = nlx2 / 8

M=7.74x22 / 8

=3.9kNm

Width of rib = 125rnm

Width of flange = 0.2 x 2000 + 125 = 525rnm

Rib spacing = 480mrn

Span = 2000 + 225 = 2225

CONCRETE AND STEEL STRESSES

Fcu = *25N/mm2*

Fy = 410 N/mrn2

MAIN REINFORCEMENT

Maximum lever arm factor = 0.95

Maximum k - value = 0.156

Moment, msn (-ve) = 3.90knm

Size of reinforcement = 10.0m

Effective depth of slabd = 250 - 20 - 5 = 225mrn

k=M / bd2fcn =3.9x10₆ / 10₃ x 2252 x 25

k= 0.003

REF	CALCULATIONS	OUTPUT

Used lever arm factor- 0.95 x 225 = 214mm

As req = M = 3.9 x 10^6

0.87fyz 0.87 x 410 x 214

As req = 51. $1mm^2$

As= 5 1.1mm'

Provide 2nos 10mm dig bars/rib

As prov=l Sjimrn'

SHEAR FORCE

V = 0.5 x 3.9 x 2.225 = 4.3kN

V=4.3 x 10^3 0.13 N/mm

224 c 150

100 As = 100 x 158 = 0.47

bd 150 x 224

Vc = 0.57

Hence provide minimum shear links

ProvideRB@ 175 *clc*

DEFLECTION

Service stress fs = 5/8 x fy x As reg = 1

As prov J3

5/8 x410 xll = 83 *N/mm*^{2}

158

SPAN EFFECTIVE DEPTH

M =3.9xl06 =0.52

bd^2 $150x224^2$

M.F = 0.55 + (477 - 83)

120 (0.9+0.52)

M.F = 0.55 + 2.8

M.F = 3.35 use 2

= 2x Basic effective span ratio =

Limiting span = 26 x 2 = 52

Actual span = 2225 = 9.8

Effective depth = 225

9.8 < 52

Topping: provide BRC mesh

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REF	CALCULATIONS	OUTPUT
	4.3 PART OF STAIRCASE DESIGN (fYPICAL FLOOR)	
	Durability and fire resistance	
BS 8110	Condition of exposure = mild	
Table 3.2	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 20mm	
Table 3.5	Nominal cover to meet fire resistance = 20min	cover=2Omm

OPEN WELL 2M SPAN

4m

2m

REF	CALCULATIONS	
Reynolds		
Table 2	LOADING	
Table 4	Dead loading	
	Self weight of waist of 0.15x24	*=3.6kN/m2*
	Step 0.5 (0.15) x 24	*= 1.8kN/m2*
	Finishes	*= 1.5kN/m2*
	Others (services)	*=0.5kN/m2*
	Total load	*= 7.400kN/m2*

=1.118

F = (3.6 + 1.5)(1.118 +1.8) 1.40 + (3) x 1.6 =

7.50 x 1.4 + 4.8

15.3kN/per m run

REF	CALCULATIONS	OUTPUT
Table 1	**DESIGN**	
	Flight 1 & 3 no of riser = 6 no of tread = 5	
	Yz landing = 550	
	Span = 2.05mm	
	ULTIMATE MOMENT	
	Ultimate moment, $M = \frac{nlx^2}{8} = \frac{15.3 \times 2.05}{8}$	
	Main = 8.04kNm	
	d= 150-20-6= 124mm b= 1000mm	
	Concrete and steel stresses	
	fcu = $25N/mm^2$	
	fy = 410 N/mm^2	
	Main Reinforcement	
	Maximum lever arm factor =0.95	
	Maximum k - value = 0.156	
	MomentMsx (-ve) =8.04kNm	
	Size of reinforcement =12mm	
	Effective depth of slab = 124mm	
	$k=\frac{M}{bd^2fcn} = \frac{8.04 \times 10^6}{10^3 x124^2 x25} = 0.021$	
	Used lever arm factor =0.95	
	As req $= \frac{8.04 \times 10^6}{0.87x410x117} = 191mm^2$	
	Provide T12 200c/c	As req = $191mm^2$
	(As prov $566mm^2$)	As prove=$655mm^2$
	SHEAR FORCE	
	V = 0.5 x 15.3 x 2.05 = 15.7kN	
	$V = \frac{15.7 \times 10^3}{1000 \times 124} = O.13N/mm$	
	$\frac{100 As}{bd} = \frac{100 \times 566}{1000 \times 125} = 0.5N/mm$	
	VC - 0.59	
	VC > V no shear reinforcement required	
	DEFLECfION	
	$5/8 \times 410 \times \frac{191}{566} = 86.5$	
	$\frac{M}{bd^2} = \frac{8.04x10^6}{1000x124^2} = 0.52$	

REF	CALCULATIONS	OUTPUT
	0.55 + (477 - 86.5)	
	120 (0.9 + 0.52)	
	M.F=2.84	
	Basic effective span ratio = 26	
	Limiting span = 26 x 2 = 52	
	Action span = 2050 =16.5	
	Effective depth = 124	
	52> 16.5 deflection OK	
	Flight 2 No of riser = 10 no of tread =9	
	Y2 landing = 1100	
	Span=4.1m	
	ULTIMATE MOMENT	
	Ultimate moment M = nlx2 / 8 = 15.3 x4.l2 / 8	
	= 32.15kNm	
	Main = 32.l5kNm	
	Effect depth d = 124mm b = 1000mm	
	CONCRETE AND STEEL STRESSES	
	Fcu = 25 *N/mm2*	
	fy = 410 *N/mm2*	
	BENDING REINFORCEMENT	
	Maximum lever arm factor = 0.95	
	Maximum k - valve = 0.156	
	MomentMsx(-ve) = 32.15knm	
	Size of reinforcement = 12mm	
	k=M / bd2t'cn = 32.15 x 106 / 1OOOx124~25 = 0.084	
	Used lever arm = 0.95	
	As req = 32.15 x 106 / 0.87x41Ox117 = 770mm	As req = 7701mm
	Provide T 12 125 c/o	As prov 905mm2
	(As prov 905mm2)	
	CONTINUITY BARS	
	50% of the main reinforcement should be provided at the top and boxthom of the span	

REF	CALCULATIONS	OUTPUT
	4.4 DESIGN OF ROOF GUITER	
	Slab self weight = 0.15 x 24 = *36kN/m2*	
	Roofing felt and screed = *l.5kN/m2*	
	Total dead load Cik = *5.6kN/m2*	
	Imposed load = *0.95kN/m2*	
	Weight of water (pendin~ = 0.3 x 9.81 = *2.94kN/m2*	
	Total hreel ok = 3.9kn1m	
	Ultimate design load (n) = 1.4gk + 1.6 qk	
	= 1.4 (5.6) + 1.6 (3.9)	
	4.03 = 14.08	n = 14.08
	14.08	
	::j/YYYYYYX)	
	~ 0.6	
	Point load (parapet self weight)	
	= O.IS x 24x 1.4	
	= *5.04kN/in2*	
	$M = \frac{wf}{2} + PL = \frac{14.08 \times 0.6^2}{2} + SO.04 \times 0.6$	
	= 2.S3 + 3.024	
	= 4.9 say 5kNm	
	SUPPORT	
Bs 8110	M=S.6knm	
Part 1	Reinforcement	
Section	$k = \frac{5.6 \times l0^6}{25 \times l000 \times 124^2} = 0.015$	
3.4.4.4	Z=0.9Sx 124 = 117	
	$As\ req = \frac{5.6 \times 10^6}{0.87 \times 41O \times 117} = 134mm^2$	
		As req=134mm^2
	Provide Y 12 @ 300 c/c	As prov=377mm^2
	(As prov 377mm^2)	
	$Mlbd^2 = \frac{5.6 \times 10^6}{1000 \times 124^2} = 0.36$	
	$Fs = S/8 \times 410 \times 289 \times \frac{1}{377} = 196N/mm$	
	$M.F = 0.55 + \frac{(477 - 196)}{120\,(0.9+0.36)}$	
	M.F=2.4	
	Limiting span = 2.0 x 26 = 52	
	Actual span = 600 = 4.84	
	Effective depth 124	
	Since 4.84 < 52.0 defection ok	detect ok

REF	CALCULATION		OUTPUT
	4.5 DESIGN OF ROOF BEAM Beam designed as continuous over supports are capable of free rotation about them. Lateral wind loads taken by columns and end shear wave.		
BS8	Durability & Fire Resistance		
Table 3:2	Conditions of exposure	= Moderate	
Table 3:4	Required fire resistance	= 1 hour	
Table 3:4	Nominal cover to meet durability	=20mm	
Table 3:5	Nominal cover to meet resistance	=20mm	Cover = 20 fire resistance
	ASSUMPTIONS		
	Fy = 410 N/mm^2	Feu = $25N/mm^2$	
	Ce=25mm	b=230mm	
	Q=16mm	h=450mm	
	fyr = $250N/mm^2$	d = 450-25-16=407	
BS 8110	ROOF BEAM 1		
Rert 1:	Short span Wx = 113 n lx		
1997	Long span Wy = y,_ n lx (I _I hk2)		
	Where n = l.4gk + 1.6qk = $11.64kN/m^2$		
		$30.l0kN/m^2$	

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(1) (2) (3) (4) (5)

LOADING		
(i) Weight rib = 0.230 x 0.3 x 24	= 1.656kn/m	
(ii) From plane 1 = 1/3 x 11.64 x 4000	= 15.52kn/m	
(iii) Fishing = 20	=2.0knlm	
	1.9.176	

'V 'V

<)

230

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REF	CALCULATION	OUTPUT

BENDING MOMENT

At Ist interior support MB = Mc = -OIIFL

Wberef=qL=30.1 x4 = *120.4kN/m*

MB = 0.11 x 120.4x4 = *53.98kN/m*

Mc = -0.11 x 120.4 x 4 = *53.98kN/m*

AT MID SPANS

MA - B = 0.098L =43.3kNm

MB - c = 0.078L =33.71kNm

MD=E=0.9FL = 43.3kNm

SHEAR FORCE

VA = 0.45F = 0.45 x 120.4 = 54.l8kN

VB = 0.6F = 0.6 x 120.4 = 72.24kN

VC = 0.6F ;::0.6 x 120.4 = 72.24kN

VD = 0.6F = 0.6 x 120.4 = 72.24kN

VE = 0.45F = 0.45 x 120.4 = 54.l8kN

MAIN REINFORCEMENT

LBeam

Over all depth = 450mm

Web width (bw) = 225mm

Flange breath bf= bw + 1(0.72)
10

= 230 + 1x 0.7 x 4000 = 510
10

Effective depth d = 450 - 25 - t - *8/2*
=407mm

Concrete cover = 25mm

Feu =*25N/mm*

Fy = 41 *ON/mm2*

Durability and fire resistance = 1 hour

Ar support B M = *53.98kN/m*

Mu = 0.156 bd2fcu

Mu 0.156 x 230 x 4072 x 25

Mu = 148.59kNm

Mu > m no compression reinforcement required

K = M = 53.98 x 106
Bd2fcu 510 x 4072 x 25
=0.025
0.02 < 0.156

Z = 0.95 x 407 = 387mm — K=0.025

As req = 53.98xl06 — As req = *391mm21*
0.87 x 410 x 387mm

As = $391 mm^2/m$ As prov=402mm

Provide 2 Y 16 (As prov 402)

Check for shear

V=72.24

V = V = 72.24xI03 =0. $77N/mm^2$

Bd 230x407

100 AS = 100 x 402 =*0.43N/mm*

bd 230x407

VC = 0.53

ASV = b(v-vc)

Sv 0.87fyv

230 (0.55-0.53)

0.87x250

= 0.021

Provide RIO @ 300 (As prov = 0.528)

Bnl2 (Grid *BI 1-5)*

Loading

Span 1-2

(1) Beam self weight = 0.3 x 0.23 x 24 = *1.656kN/m*

(2) From slab PI =*113*nlx 113x 11.64 x 4 = 15.*52kN/m*

(3)·From slab P4 = 113nix 113x 11.64 x 4 = *13.52kN/m*

Totalload = *32.70kN/m*

Span 2-3

(1) Ream self weight = 0.3 x 0.23 x 24 = *1.656kN/m*

(2) From P4 = 113n be = 1/3 x 11.64 x 4 = *15.52kN/m*

(3) From P5 = Y2n lxl - 1I3k2) where k = ly/Lx

= Y2x 11.64 x 4 (1 - *1I3(2i)* = 21.34

Totalload = *38.52kN/m*

Span 3-4

(1) Ream self weight = 0.3 x 0.23 x 24 = 1.656

(2) From P4 = *1/3* nix = 1/3 x 11.64 x 4 = 15.52

(3) From P5 = Y2n be (1 - 1I3k2)

= Y2x 11.64 x 4 *(1-113(2i)* = 21.34

Total load = 38. *52kN/m*

Span 4 - 5 =Span 1-2 = *32.70kN/m*

~

REF	CALCULATION	OUTPUT

~ 'f>~ ~

r-.

	(l)4m		(2)4m		(3)4m		(4)4m	
K	1/4 x3/4=O.l88		Y. =0.25		Y. =0.25		Y.x3/4 = 0.188	
DF	0	=0.43	057	0.5	05	057	0.43	0
FEM	-42.67	42.67	-51.4	51.4	-51.4	51.4	-42.67	42.67
DM	42.67	3.8	4.97	0	0	4.97	3.8	
COM			-	2.5	-25	-	-	
EM	0	46.47	-46.43	53.9	-53.9	46.43	- 46.47	
Elastic shear	64	64	77.04	77.04	77.04	77.04	64	64
Static Shear	-11.68	11.68	-1.87	1.87	-1.87	1.87	11.68	-11.68
Total Shear	52.32	75.68	75.2	78.91	78.91	75.17	75.7	52.32
Reaction		15	0.88			150	87	
SM	43		27		27		43	

-

Bending Moment Diagram

A- 539foA r;

43kn/m 27kn1m 27kn/m 43kn/m

Shear Diagram

5~ '~~ 751~

78.91 75.71 52.32

SHEARING FORCES

V_{AB} = WIl2 + (MA- MB/c) = 32 x 4 + (0.46.47) = 75.6kN
2 4

VBA = WIl2 + (MB - MAIL) = 32x4+(46.47-O) = 75.62kN
2 4

~

REF	CALCULATION	OUTPUT

VBC= WlI2 + (Ma - Mc/L) = 38.52x4 + (46.43-53.9)
2 4
= 77.04 = 1.87
= 75.2kN

YCB=WI + (Mc-MB/c) = 38.52x4 +(53.9-46.43)
2 2 4
= 78.91kN

YCD= WI + (MC - *MDIL)* = 38.52x4 + (53.9-46.43)
2 2 4
= 78.91kN

Voc = WI + (MD - McIL) = 38.52x4 + (46.43 - 53.9)
2 4
= 75.2kN

VDE= WI + (MD - MEiL) = 32x4 + (-46.74-O)
2 4
= 75.7kN

YED= WI + (ME- MoIL) = 32x4 + (O-46.74)
2 4

YCB= WI + (Mc - Malc) = 38.52 x4 + (53.9 -46.43)
2 4
= 78.91kN

VCD= WI + (M, - *MDIL)* = 38.52x4 + (53.9 - 46.43)
2 4
= 78.91kN

Voc = WI + (MD - Mc/L) = 38.52 x 4 + (46.43-53.9)
2 4
= 75.2kN

YDE= WI + (MD - MEIL) = 32x4 + (-46.74-0)
2 4
= 75.7kN

YED = WI + (ME - MD/L) = 32x4 + (0-46.74)
2 4
= 52.32kN

MAXIMUM MOMENTS

MAB = *v'* AB hw */MN* = 52.322 *-/MN* = *43kN/m*
2x32

MBC = V2 BC/2w *-MBl* = 75.22 - 46.43 = 27kNm
2x38.52

MCD = V2 CDhw *-Mcl* = 78.91 - 53.9 = *27kN/m*
2x38.52

MDE = y2 DE *12w/MDI* = 52.322 - 0 = 43kNm

~

REF	CALCULATION	OUTPUT
	SPAN 1-2	
	M=43kN/m	
	Section	
	BF = Bw + 0.2 (0.7L) - 230 + 0.2 (0.7 x 4(00) = 790mm	
	bf	
	15 ,....---------' ~ hf	
	.. 230·	
B58110		
Rant 1		
Section	REINFORCEMENT	
3.4,4.4	k=M = 43xl06	
	fcubd^2 25x79Ox407^2	
	K = 0.013 < 0.156	k= 0.013
	Z=0.94x407	
	Z=383mm^2	z=383mm
	As req = M = 43xl06	As req=Sl Smm"
	0.87fyz 0.87x410x383	As prov = 402mm
	= 315mm^2	
	Provide 2 Y 16 (As prov = 402mm^2)	2Y16
	Support 2/3	
	M=53.9	
	k=M = 53.9Oxl06	
	bd^{2}fcn 23Ox407^{2}x25	
	= 0.057 < 0.156	K=0.057
	Z = 0.95d = 0.95 x 407 = 387mm	
	As req = M___ = 53.9x106	
	0.87fyz 0.87x400x387	
	= 391mm^2	
	Provide 2 Y 16 (As prov = 402mm^2)	As req=391mm^2
		As prov=402mm^2
	CHECK FOR SHEAR	
BS 8110	Vmax=78.91	
Rant 1	Shear stress V = vlbd < *4N/mm2*	
Section		

REF	CALCULATION	OUTPUT
3.4.5.2	= 78.91x10 3	
1997	230x407	
	=0.84N/mm2	
	100 As prov = 10Ox402 = 0.43 *N/mm2*	
	bd 23Ox407	
	Vc=0.71	
	Since V > vc shear reinforcement is required	
	ASU = b(Y-vc) = 230 (0.89-0.71)	
	SV 0.87x250 0.87x250	
	0.1375	
	Provide Y 10 (As prov = 0.523)	
	CHECK FOR DEFLECTION	
B58110	M =53.9xl0 6	
Rant 1	bd 2 23Ox407 2	
Table 3.11	= *1.42N/mm2*	
	Fs=5/8x410x315 =201N/mm2	
	402	
	M.F = 0.55 + 477-201	
	120(0.9+1.42)	
	= 1.54	
	Limiting span = 1.54 x 26 = 40.1	
	Actual span =4000 =9.83	
	Effective dearth 407	
	Since 9.83 < 47.59 deflection 15 ok	deflection ok
	Beams 3 (grid (1-5»	
	Span 1-2 total load = *32.70kN/m*	
	Span 2-4 total load = 21.34x2 = *42.68kN/m*	
	Span 4-5 total load = *38.52kN/m*	

L --~(\,- _L_

	(1) 4m		(2) 8m	(4)	4m	(5)
K	Y4x% = 0.188		118= 0.125		Y4X% =0.188	
DF	0	0.6	0.4	0.4	0.6	0
FEM	-43.6	43.6	-228	228	-43.6	43.6
DM	43.6	111	73.8	73.8	111	43.6
COM		----	36.9	36.9	----	
DM		22.14	14.76	14.76	22.14	
COM			7.4	7.4	----	
DM		4.44	2.96	2.96	4.44	
COM		--	1.48	1.48	---	
DM		0.89	0.59	0.59	0.89	
EM		18207	181.67	181.67	182.07	
Elastic shear	65.4	65.4	170.72	170.72	65.4	65.4
Static Shear	-45.5	45.5	0	0	45.5	-45.5
Total	19.9	110.9	170.72	170.72	110.9	19.9
Reaction		28	2	28	2	
SM	6		160		6	

BENDING MOMENT DIAGRAM

1 4

6kN/m *160kN/m* *6kN/m*

SHEAR FORCE DIAGRAM

170.72kN

19.9kN

REF	CALCULATION	OUTPUT
	MAXIMUM MOMENT	
	MAB=y2AB/2w=(19.9i -1MN=6kN/m	
	2(32.70)	
	MBC= y2 BC/2w-IMB/ = (170.72i - 1MB = 160kN/m	
	2(42.68)	
	Mco = y2 CD/2w/MC/ = 110.92- /MC = 6kN/m	
	2(32.70)	
	SPAN 1-2	
BS 8110	M=6kN/m	
Part 1	SECTION=T	
Section	BF = bw + 0.2 (0.7L) = 230 + 0.2 (0.7x4000)	
3.4.4.4'	= 790mm	
1997	k=M =6x10^6 =0.01	
	Fcubd2 25x790x4072	k = 0.01
	0.01 < 0.156	
	Z = 0.95x407 = 387mm	
	Asreq=M. = 6x10^6	
	0.87fyz 0.87x410x387	
	=44mm2	
	Provide 2 Y16	As prov=402mm2
	As prov=402mm2	
	SUPPORT 112	
	M= 182.07	
	K=M = 182.07x10^6	
	bd'fcu 230x407~5	
	= 0.191 > 0.156	
	Therefore compression reinforcement is required assume 20mm has compression reinforcement cover = 35mm	
	∴ d' = 35+3% = 50mm	
	As1 = k - k1 fcubd2	
	0.87fy (d-d1)	
	= (0.2-0.156) 25x230x4072	As req=329.11mrr
	0.87x410 (407-50)	
	= 329.11mm2	As prov = 402mm
	Provide 2 - Y16 (As ~rov = 402mm2)	
	As = kl fcubd + A1S	
	0.87fyz	

REF	CALCULATION	OUTPUT

$= \frac{0.156x25x230x407^2}{0.87x410 \times 315} + 329.11$

$= \frac{365,079}{112,360.5} + 329.11$

I,651.5mm2

Provide 2 - Y25 + 3 - Y20T (As prov = 1.925)

CHECK FOR SHEAR

Vmax= I70.72kN

Shear stress v = v/bd ≤ *4N/mm2*

$= \frac{I70.72xl0^3}{230 \times 407}$ = *1.8N/mm*

$\frac{100As}{bd} = \frac{100 \times 1.925}{230 \times 407}$ = 2.05

Vc = *0.583N/mm2*

Since V > Vc shear reinforcement is required.

$As\,v/Sv = \frac{b\,(v-vc)}{0.87fy} = \frac{230\,(1.6-6.583)}{0.87 \times 250}$

1.08

Provide YIO @ 125 (Asv/Su = 1.256)

CHECK FOR DEFLECI10N

$Mlbd^2 = \frac{182.07 \times 10^6}{23Ox407^2}$ = 4*.78N/mm2*

$Fs = \frac{5/8 \times 410 \times 1,651.5}{1,925}$ = 219.8 kn/mnr'

$M.F = 0.55 + \frac{477 - 219.8}{120\,(0.9+4.19)}$ = 0.97

Limiting span = 0.97 x 26 = 25.25

Actual span $= \frac{4000}{407}$ = 9.83

Since 25.25 < 9.83 deflection is ok

Span 2 - *4/4.5*

M= 160 *kn/m*

4Y20

As prov=1260mm

REF	CALCULATION	OUTPUT

REINFORCEMENT

K=M = 16Ox10^6 =0.049

bd'fcu 25x407~790

0.049 < 0.156

Z = 0.95 x 407 = 387mm

As req = 160 x 10^6 = 1,159.1mm^2

0.87x410x387

Provide 4 Y 20 (As prov 1260)

CHECK FOR DEFLECTION

M = 160x10^6 = 4.2 *N/mm^2*

bd^2 230x407^2

Fs = *5/8* x 410 x 1.1591 = *236N/mm^2*

1260

M.F 0.55 + 477-236

120(0.9+4.2)

M.F=0.94<2

Limiting span = 0.94x26 = 24.44

Actual depth = 8000 = 19.66

407

Since 19.66 < 24.44

Deflect ok

Beam 4 (larid D 1-4)

Span 1-2 total load = *32.70kN/m*

Span 2 _3$_1$total load = *36.65kN/m*

Span 3$_1$_ 4 total load = *14.51kN/m*

\

REF	CALCULATION	OUTPUT

L ~,- L

	(1) 4m		(2) 5m	(3)	3m	(4)
K	~x%=0.188		1/5=0.2		Ih x%=0.25	
DF	0	0.48	0.52	0.44	0.56	0
FEM	-43.6	43.6	-76.3	76.35	-10.9	10.9
DM	43.6	15.72	17.03	28.80	36.70	12.5
COM		----	-14.40	8.52	----	
DM		6.91	7.49	-3.75	-4.77	
COM		----	1.88	3.75	---	
DM		0.90	0.98	-1.65	2.1	
COM		--	0.83	0.49	---	
DM		0.39	0.43	-0.22	-0.27	
EM	0	67.52	-67.53	54.69	-54.74	0
Elastic Shear	65.4	65.4	91.63	91.63	21.77	21.77
static shear	-16.88	16.88	2.6	-2.6	18.3	18.3
Total Shear	48.52	82.3	94.23	89.02	40.07	3.47
Reaction		176	.53		138.09	
SM	36			53.5		55

BENDING MOMENT DIAGRAM

67.52KN

54.74kN

36KN 53.5kN 55kN

SHEAR FORCE DIAGRAM

94.23kN

82.3kN 889.02kN

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REF	CALCULATION	OUTPUT
	MAXIMUM MOMENT	
	MAB= V2 AB/2w -/MN = 48.522 -/MN =36kN/m	
	2(32.70)	
	MBc = V2 BC/2w-1MB1 = (94.23i -IMBI = 53.5kN/m	
	2(36.65)	
	McD = V2 CD/2w -/MCl = (3.47i -/MCl =SSkN/m	
	2(14.51)	
B 58110	SPAN 1-2	
Pert 1	M=36.0knlm	
Section	BF = bw + 0.2 (0.7L) = 230 + 0.2 (0.7 x 4(00) = 790mm	
3.4.4.4		
1997	k =M = 36x106 =0.01	
	fcubd2 25x79Ox4072	k=0.01
	0.01 < 0.156	
	Z=0.94x407	
	= 387mm	
	Asreq =M =36x106	As req=387mm2
	0.87fy 0.87x410x387	
	387mm2	As prov=402mm2
	Provide = 2 Y 16 (As prov 402mm2)	
	SUPPORT 112	
	M=67.52	
	K=M K = 67. 52x106	
	fcubd2 25x79Ox4072	
	k = 0.02 < 0.156	As req = 489mm2
	as req =M = 67.52x106	
	0.87fyz 0.87x41 Ox387	As prov=603mm2
	= 489mm2	
	Provide 3 Y 16 (As prov 603mm2)	
	CHECK FOR SHEAR	
	Vmax= 94.23	
	Shear stress v = v/bd = < 4N/mm2	
	94.23x103	
	230x407 = 1.01 N/mm2	
	VC = 0.75N/mm2	
	Since V > VC sheer reinforcement	
	ASV = b (v - vc) = 230 (1.01 - 0.75)	
	Sv 0.87x250 0.87x250	
	=0.27	
	Provide Y10 @ 300 (ASV/SV = 0.523)	

REF	CALCULATION	OUTPUT

CHECK FOR DEFLECTION

M = 67.52x106 = *1.8N/mm2*
bd2 230x4072

FS = *5/8* x 410 x 489 = *208N/mm2*
603

M.F = 0.55 + 477 - 208
120(0.9+1.8)

M.F= 1.9 <2

Limiting span = 1.90 x 26 = 49.4

Actual span = 4000 = 9.8
407

Since 9.8 < 49.4 deflection is ok

Span 2 _3113 - 41

M=53.5 *kN/m*

SECTIONT

.:. BF=bw+O.2 (0.7L)=790mm

K = M = 53.5x106
fcbd2 25x790x4072

k = 0.016 < 0.156

Z = 0.95 x 407 = 387mm

As =M = 53.5 x 106
0.87fyz 0.87x410x387

= 388mm2

Provide 2Y16 (As prov = 402mm2)

CHECK FOR SHEAR

VMax=89.02

Shear stress = V = vlbd =< 4 *N/mm2*

= 89.02x103 = *0.95N/mm*
230x407

Vc=0.77N/mm2

Since V > Vc shear reinforcement

ASV = b(v-vc) =230 (0.95-0.77)
SV 0.87x250 0.87x250

=0.19

Provide Y10 @300 0-5 (ASv = 0.523)
sv

~

REF CALCULATION OUTPUT

_ .L_ ~

3.140m 3.S60m

	3	4		5
K	Ih14 x 314 = 0.0.24		IhS6 x ',4 =0.19	
DF	0	0.55	0.45	0
FEM	-11.67	11.67	-40.24	40.24
DM	11.67	15.7	12.9	40.24
EM	0	27.34	-27.34	0
Elastic shear	22.3	22.3	62.6	62.6
Static shear	-S.71	S.71	7.1	-71
Total shear	13.59	31.01	69.7	55.5
Reaction		100	71	
SM	6.5		47.6	

BENDING MOMENT DIAGRAM 27.34kN

-404. / """"L I :vA"CIY

47.6kN

6.5kN

SHEAR FORCE DIAGRAM

31.01kN

69.7kN 55.5kN

~

REF	CALCULATION	OUTPUT

Span3 -4

M=6.5 *kN/m*

BF = bw + 0.2 (O.7L)= 230 + 0.2 (0.7 x 3140)

= 670mm

K=M =6.5 x 10^6 =0.02

fcubd" 25x67Ox4072

Z=0.95 x 407

Z=387mm

As = M = 6.5 x 10^6

0.87fyz 0.87x410x387

= 47.09mm2

Provide 2Y16

As prove = 402mm2

Support 3 *1/4*

M=27.34

K=M = 27.34X10^6 = 0.09

fcubd' 25x670x4072

0.9 < 0.156

No compression reinforcement is required

As=M

0.87fyz

27.34x10^6 = 200mm2

0.87x410x387

Provide 2Y16 (As prov = 402mm)

CHECK FOR SHEER

V max = 69.7kN

Shear stress V = v/bd = < *4N/mm2*

V = 69.7 x 10^3 = 0.74 *N/mm*

230x407

Vc=0.6O

Since V > vc shear reinforcement

ASU = b r.y.vc)

Sv 0.87x250

230 (0.74 - 0.60)

0.87x250

= 0.148

Provide Y10 @ 300 (As prov = 0.523)

SPAN 4-5

M=47.6kN/m

REF	CALCULATION	OUTPUT
	SECfIONT	
	K=M =47.6x106 =0.0145	
	fcubd2 25x79Ox4072	
	Z = 0.95 x 407	
	Z=387mm	
	As=M = 47.6x106	
	0.87fyz O.87x41 Ox387	
		As req = 348mm2
	Provide 2 Y 12 (As prov = 402mm2)	
		As prov=402mm2
	CHECK FOR DEFLECTION	
	M = 47.6xl06 = *1.25N/mm*	
	bd2 230x4072	
	FS *5/8* x 410 x 348 = *222N/mm2*	
	402	
	M.F = 0.55 + 477 - 222	
	120 (0.9 + 1.25)	
	M.F= 1.56	
	Limiting span = 1.56 x 26 = 40.55	
	Action spen = 3860 = 9.48	
	Effective depth 407	
	9.48 < 40.55 deflect is ok	
	Beam 6 (Grid E 1- 31)	
	Span 1-2 total load = 32.70 kn/m	

REF	CALCULATION	OUTPUT

	(1) 4000		(2) 5000	(3)
K	*1/4* × % = 0.188		*I/sx%=0.15*	
DF	0	0.55	0.45	
FEM	-43.6	43.6	-76.25	76.25
DM	43.6	18.0	15.0	76.25
EM		61.6	-61.25	
Elasn shear	65.4	65.4	91.5	91.5
Static shear	-15.4	15.4	12.25	-12.25
Total shear	50	80.8	103.8	79.25
Reaction		185		
SM	38.2		86	

BENDING MOMENT DIAGRAM

SHEAR FORCE DIAGRAM

l03.8kN

RO.RkN 79.25kN

SPAN 1-2

M=38.2 *kn/m*

Bf= bw+ 0.2 (0.7L) = 230 + 0.2 (0.7x4000)

= 790mm

$= \frac{38.2 \times 10^2}{25x790x407^2} = 0.012$ k= 0.012

0.012 < 0.156

REF CALCULATION OUTPUT

CHECK FOR DEFLE ION

M=53.5Knfm

M = 53.*sOxl* 0[6] = 1.40 *N/mm2*

bd[2] 230x407[2]

fs = *5/8* x 410 x 388 = 396.4 *N/mm2*

402

M.F = 0.55 + 477 - 396.4

120(0.9+1.4)

Vc = 0.77 *N/mm2*

Since V > Vc shear reinforcement

ASV = b(v-vc) =230 (0.95-0.77)

SV 0.87x250 0.87x250

= 0.19

Provide YI0 @ 300 0-5 (ASv = 0.523)

sv

CHECK FOR DEFLECfION

M= 53.5 *Knfm*

M = 53.*sOxl* 0[6] = 1.40 *N/mm2*

bd[2] 230x407[2]

fs = *5/8* x 410 x 388 = 396.4 *N/mm2*

402

M.F = 0.55 + 477 - 396.4

120(0.9+1.4)

= 0.84 <2

Limiting span = 0.84 x 26 = 21.89

Actual span = 5000 = 12.3

407

Since 12.3 < 48.1 deflection is ok

Beam5 (Grid DI 3[1] - 5)

Span = 3 - 4 = *14.20kN/m*

Span 4 -5 = 32.41kNm

REF | CALCULATION | OUTPUf

No compression reinforcement is required

As = M = 38.2 x 10⁶ → $As = \frac{M}{0.87fyz} = \frac{38.2 \times 10^6}{0.87x410x387}$

Provide 2Y16 (As prov = 402mm2)

SUPPORTYJ

$K = \frac{M}{Fcubd2} = \frac{61.6xl0^6}{25x79Ox4072} = 0.019$

K = 0.019 < 0.156 — k=0.019

$As = \frac{M}{0.87fyz}$

$= \frac{61.6 c 10^6}{0.87x410x387}$

= 446.2mm2

Provide 3 Y 16 (As prov 603mm2)

CHECK FOR SHEAR FORCE

V max = 103.8kN

Shear stress V = v/bd = < *kN/mm2*

$V = \frac{103.8x10^3}{230x407} = 1.1$ *N/mm*

VC=0.67

Since V > Vc shear reinforcement required

$\frac{ASV}{SV} = \frac{b (v - vc)}{0.87x250} = \frac{230 (1.1 - 0.67)}{0.87x250}$

$\frac{ASV}{Sv} = \frac{b(v-vc)}{0.87x250} = 0.35$

Provide YIO @ 300 (As prov 0.523)

CHECK FOR DEFLECTION

$\frac{M}{Bd_2} = \frac{61.6 x 10^6}{230 x 407_2} = 1.62N/mm_2$

$Fs = 5/8 x 410 x \frac{446.2}{603} = 190 N/mm_2$

$M.F = 0.55 + \frac{477-190}{120(0.9+1.62)}$

MF= 1.49

Limiting span = 1.49 x 26 = 38.98

Actual span = 4000 = 9.83

REF CALCULATION OUTPUT

BEAMS

Span 2 - 3₁ total load = 24.6 knIm

Span 3₁-4 total load = 19.83kn/m

Span 4 5 total load = *39.7knlm*

~1.CIcMn

(2) 4000 : 1000 (3) 3140 (4) 3860 (5)

K	1/5=0.2x %	= 0.15	lh.14 = 0.32		lh.8=O.26xo/..+04	
DF	0	0.28	0.72	0.38	0.62	0
FEM	-84.05	84.05	-16.3	16.3	-49.3	49.3
DM	84.05	18.97	-48.78	12.54	20.46	49.3
COM		----	6.27	24.39	----	
DM		1.78	4.51	9.27	15.12	
COM		----	3.91	-2.26	---	
DM		-1.00	-2.82	0.86	1.40	
COM		--	0.43	1.41	---	
DM		0.12	0.31	0.45	0.87	
EM		62.18	-62.11	11.36	-11.45	
Elasn Shear	69.7	69.7	31.1	31.1	12.61	12.61
Static shear	-15.55	15.55	16.2	16.2	2.86	2.86
Total	54.15	85.25	14.93	47.3	15.47	9.75
Reaction		100	18	62	.77	
SM	59.59		56.41		8.4	

BENDING MOMENT DIAGRAM

62.18kN 11.36kN

59.59kN 56.41kN

SHEAR FORCE DIAGRAM

14.53kN 15.47kN

54.15kN

85.25kN 59 47.3kN 9.75

REF CALCULATION OUTPUT

Effective depth 407

Since 9.83 < 38.46 deflection is ok

SPAN 2 -3

M=86kN/m

K=M = 86xl0^{6}

Fcubd^{2} 25x279Ox407^{2}

Z = 0.9 x 407

Z=387mm

As=M = 86xl0^{6}

0.87fyz 0.87x410x387

622mm2

Prvide 4Yl6 (As prov 804mm2)

CHECK FOR SHEAR

V = vlbd = < *4N/mm^{2}*

V = 80.8x10^{3} = *0.86N/mm*

230x407

Vc = 0.72

Since V > Vc shear reinforcement is required

ASV = b(v-vc) = 230 (0.86-0.72)

Sv 0.87x250 0.87x250

= 0.148

Provide YIO @300 (As prov 523mm2)

BEAM 10

Span 1-2 total load = *16.35kN/m*

Total load = 19.83

~

"II1 IIJY

M=W12 = 36.2x42 = 72.36

8 8

M= 72.36

K=M = 72.36xl0^{6} = 0.022

fcubd" 25x790x407^{2}

k= 0.022

Z = 0.95 x 407 = 387mm

As = M = 72.36x10^{6}

0.87fyz 0.87x410x387

= 524.2mm2

Provide 3Y16 As prov 603mm^{2}

REF | CALCULATION | OUTPUT

Shear = V= 0.5 x (4) 36.7 = 73.4kN

$\frac{73.4x10^3}{13Ox407}$. = 0.78 *N/mm*^{2}

100 As prov = 100 x 524
bd 230x603
=0.38
Vc=0.51

BEAM 14
(Grid 3E1 - 4)
Span El - 4 total load *Yz* ben (I - 1I3k2)
Yz x 11.64 (I - 113*(1.83i)* x 2 = 41 *kN/m*

L'''-

~
We = 41.7.33^2 = *275.4kN/m*
8 8
K=M = 275.4x10^6 =0.08
Fcubd^2 25x79Ox407^2
K = 0.08 < 0.156
Z=0.95 x 407
Z=387mm
As=M = 275.4xl06

0.87fyz 0.87x410x387
2,016mm^2
Provide 5 Y 25mril2 (As prov 2450mm2)

CHECK FOR SHEAR
V = 0.5 x 7.3 x 41 = 149.65kN
V = 149.65kN
V = y_ = 149.65x10^3
bd 230x407
= 1.59 *N/nun*2
100 As = 100 x 2. 450
bd 230 x 407 =2.6%
Vc = *0.77N/mm2*

REF CALCULATION OUTPUT

SPAN 2 - 3₁

M = 59.59 *kn/m*

BF = bw + 0.2 (0.7L) = 230 + 02 (0.7 x 4(00) = 790mm

K = M = 59.59x10⁶

fcnbd2 25x790x4072 = 0.018

Z = 0.95 x 407

Z=387mm

As=M = 59.59x10⁶

0.87fyz 0.87x410x387

= 432mm2

Provide 3 Y16 (As prov 603mm2)

SUPPORT 2 _3₁

M = 62.1Skn/m

K=M =K62.18x10⁶

fcnbd2 25x790x4072

= 0.019

Z = 0.95 x407

= 387mm

As=M = 62.l8 x 10⁶

0.S7fyz 0.87x410x387

= 450mm2

Provide 3 Y 16 (As prov = 603mm2)

CHECK FOR SHEAR

Shear stress V = v/bd = *<4N/mm2*

= 85 x 10³ = 0.91 *N/mm*

230x407

Vc = 0.8 *N/mm2*

Since V > Vc shear reinforcement

ASV = b(v - vc) = 230 (0.91-08)

Sv 0.87x250 0.87x250

= 0.l16

Provide YI0 @ 300 (ADV = 523)

Sv

Span 3₁ - 4 *14- 51*

M = 56.41kn/m

REF CALCULATION OUTPUT

E I

K=M = 56.41xl06 =0.017

Fcnbd2 25x790x4072

Z = 0.95 x 407 = 387mm

As=M = 56.41xl06

087fyz 0.87x410x387

=408mm2

Provide 3 Y16 (As prov = 603mm2)

CHECK FOR SHEAR

Shear stress = V = v/bd = <4N/mm

= 47.7kn

=47.7x103 =0.50 *N/mm*

23 Ox407

Vc =0.59 *N/mm2*

Since VC > V shear reinforcement is not required

CHECK FOR DEFLECTION

M = 56.41 kn/m

M = 56.41xl06 = 1.48 *N/mm2*

bd2 230x4072

8/5 = 5/8 x 410 x 408 = 173 *N/mm2*

603

M.F = 0.55 +477 -173

12(0.9+1.48)

= 1.6 < 2

Limiting span = 1.626 =41.6

Actual span = 3140 =7.7

407

Since 41.6 < 7.7 deflection is ok

Beam 22, (Grid 31nEI)

SpanD-E

Span E - E total load = *12.46kn1*

11...

2667/ 1knfm

(? т .LD

K "h.7x%=0.27 1"~x3;'=0.188

DF 0 0.58 0.42 0

FEM ~.6 7.6 -21.47 21.47

DM '.6 8.04 5.8

15.6-115.67

Elastic 6.82 16.82 32.2 32.2

Shear

Static - 5.8 5.8 8.8 3.8

Total Shear 11.02 22.62 38 26.4

Reaction 60.12

~M 14.9 29.4

BENDING MOMENT DIAGRAM

15.64kN

A

29.2kN

4.9k:N

SHEAR FORCE DIAGRAM

22.62kN

Il.02kN

'~

~

38kN 26.4kN

REF	CALCULATION	OUTPUT

SPANE-E1

K=M =4.9x 10⁶ k=O.017 — k = 0.017

fcubd2 25x69Ox4072

Z =0.95 x 407

= 387mm

~=M = 4.9xl06

0.8fyz 0.87x410x287

Provide 2 Y16 (As prov 402mm2)

SUPPORT E - El

M = 15.64 *kN/m*

K=M = 15.64xl06

fcubd2 25x690x4072

= 0.05 < 0.156

Z = 0.9S x 407

Z=387mm

As=M =IS.6Sxl06

0.87fyz 0.87x410x387

= 11Smm2

Provide 2 Y16 (As proved 402mm2)

CHECK FOR SHEAR

Vmax=38kn

Shear stress V = vlhd = < *4N/mm2*

V = 38 x 10³ = *0.42N/mm2*

230x407

Vc=0.49

Vc > V no shear reinforcement required.

SPANDE

M=29.2

K=M = 29.2xl06 =0.011

fcnbd" 2Sx670x4072

0.011 < 0.1S6

As=M = 29.2 x 106

0.87fyz 0.87x410x387

= 214mm2

. Provide 2 Y 16 (As prov 402mm2)

1.c. c> ..... c. c> ~:~~c.oooooooo~9.85kN/m
400C ~ 4000..t.· 4000 lt- 4000 lt- 4000 lt- 600c lt-
C D E F H
,..------,
¢ 150
t 300

BENDING MOMENT
At ist interior support MB = Me = MD = ME = O.IIFL
Where F = qL = 14.5x = 58.0kN/m
MB = - 0.11 x 58 x 4 = 25.52 kN/m
MC = -0.11 x 58 x 4 = 25.52kN/m
MD = -0.llx58x4 = 25.52kN/m
ME = 0.11 x 58x4 = 25.52 kN/m
'MF = -011 x 114.1x6 = 78.61kN/m

AS MID SPANS
MA-B=0.9FL =20.88kNm
MB - C = 0.07FL =16.24kN/m
MC-D= 0.07FL = 16.24 kN/m
MD - E = 0.07FL =16.24kNm
ME - F = 0.07FL = 16.24kN/m
MF - H= 0.09 FL = 64.314 kN/m

SHEAR FORCE
VA = 0.45F = 0.45x58 = 26.lkN
VB = 0.6F = 0.6 x 58 = 34.8kN
VC = 0.6F = 0.6 x 58 = 34.8kN
VD = 0.6F = 0.6 x 58 = 34.8kN
VE = 0.6F = 0.6 x 58 = 34.8kN
VF = 0.6F = 0.6 x 58 = 34.8kN
VH = 0.45F = 0.45 x 119.1 = 53.60kN

MAIN REINFORCEMENT
b -Bean
Over all depth = 450mm
Web width (bw) = 230
Flange breath, bf= bw + 1110(0.7L)
= 230 + 1/10 x 0.7 x 400 = 510
Effective depth d = 450 - 25 - t.9/z
= 450 - 25 - 10 - 16/z

REF CALCULATION OUTPUT

=407mm
Concrete over = 25mm
Feu = *25N/mm2*
Fy = 41 *ON/mm2*
Durability and fire resistance = 1 hr
Condition of exposure = mild

At support BM 25. 52kNm
Mu = 0.156 bd^2 Fcu
Mu = 0.156 x 225 x 407^2 x 25
Mu = 145,36 kNm

Mu > M no compression reinforcement require

$$K = \frac{M}{bd^2 fcn} = \frac{25x52xl0^6}{230x510^2x25} = 0.017$$

0.017 < 0.156

Z = 0.95 x 407 = 387mm

$$As = \frac{25.52xl0^6}{0.87x410x387}$$

= $185mm^2$

Provide 2 T 16 As prov 402mm

CHECK FOR SHEAR
V= 53.60

$$V = v = \frac{53.60xl0^3}{230x407} = 0.57N/mm^2$$

$$\frac{100AS}{bd} = \frac{100 \times 402}{230x407} = 0.43N/mm2$$

VC=0.5

$$\frac{ASV}{Sv} = \frac{b\ (v-vc)}{0.87dyv}$$

$$\frac{230\ (0.57-0.51)}{0.87\ x250}$$

= 0.063

Provide YIO link at 300mm

BEAM 23-19
Yz SPAN E - E1 total load = 21kN/m
Span E1_ G total load = 41kN/m
SPAN G - H total load = *24.08kN/m*

REF	CALCULATION	OUTPUT

--v-v- _;;_~v--?y-y-,~

	(E) 2000		(E1) 5333	(G)	2667	(5)
K	1/'lX % = 0.373		1/5.3 = 0.188		Ih.7x%=0.27	
DF	0	0.66	0.34	0.41	0.59	0
FEM	-7	7	-98.03	98.03	-14	14
DM	7	60.1	30.95	34.5	49.6	
COM		----	17.25	15.4	----	
DM		11.39	5.87	6.35	9.09	
COM		----	3.18	2.94	---	
DM		2.10	1.08	1.21	1.74	
COM		--	0.61	0.54	----	
DM		0.40	0.21	0.22	0.32	
EM	0	80.99	-80.96	74.68	-74.75	0
Elasn Shear	21	21	112.0	112.0	32.2	32.2
Static shear	-40.5	40.5	1.18	1.18	27.9	27.9
Total	-19.51	61.5	113.2	110.8	601	4.3
SM	9.1		71.8		0.38	

BENDING MOMENT DIAGRAM

SO.96kN 74.6SkN

O.3SkN

SHEAR FORCE DIAGRAM

113.2kN 6() 1kN

61 5leN 110 SleN

REF	CALCULATION	OUTPUT

SPANE-E_1
M=9.1 *kN/m*
B.F = bw + 0.2 (0.7L) = 230 + 0.2 (0.7x4000)
= 790mm

$K = \frac{M}{fcubd2} = \frac{9.1x10^6}{25x790x4072} = 0.02 < 0.156$

= -0.95 x 407
= 387mm

$As = \frac{M}{0.87fyz} = \frac{9.10 \times 10^6}{0.87x410x387}$

$= 66mm^2$
Provide 2 Y16 (As prov 402mm2)

Support E - El
M=80.%

$K = \frac{M}{fcnbd''} = \frac{80.96xl0^6}{25x79Ox407^2}$
K= 0.024
Z = 0.95 x 407
= 387mm

$As\ req = \frac{M}{0.87fyz} = \frac{80.96xl0^6}{0.87x410x387}$
= 587.5mm2

Provide 3 Y16 (As prov = 603mm2)

CHECK FOR SHEAR
V max:= 113.2 lm
Shear stress V = vlbd = < *4N/mm2*

$\frac{113.2 \times 10^3}{230 \times 407} = 1.21$ *N/mm*

REF	CALCULATION	OUTPUT

y

Vc=0.79N/mm2
V> vc shear rainforcement
ASV = b(v-vc) = 230 (1.21 - 0.29)
Sv 0.87x250 0.87x250

=0.444
Provide Y 10 @ 300 - (As v/sv = 0.523mm)

Span El - G *IG-H*
M= 71.8 *kN/m*

SECfIONT
BF=790mm

K = M = 71.8x10^6 = 0.22
fcnbd" 25x790x407^2
K = 0.022<0.56
Z = 0.95 x 407 = 387mm

Asreq=M._ = 71.8 x 10^6
0.87fyz 0.87x410x387
= 520mm^2

Provide 3 Y 16 (As prov = 603mm^2)

Check for shear
Vmax= 110.8
Shear stress = v = vlbd = < *4N/mm2*
110.8 x 10^3 = 1.20 *N/mm*
230x407

Vc =0.79
Since V > vc shear reinforcement
ASv = b(v-vc) = 230 (1.20 - 0.79)
Sv 0.87x250 0.87x250
=0.43

Provide YI0 @ 300 (ASv = 523)
Sv

CHECK FOR DEFLECfION
M=71.8 knlm
M = 71.8 x 10^6 = 1.9 *N/mm2*
bd^2 230x407^2

REF	CALCULATION	OUTPUT

Fs=ix410x520 = *221N/rrun2*
8 603

y

M.F = 0.55 + 477 - 221
120 (0.9 + 1.9)

M.F = 1.3 < 2
Limiting span = 1.3 x 26 = 33.8
Actual span = 5333
Effective span = 407
= 13.10
Since 13.10 < 33.8 deflection is ok

BEAM 9 = BEAM 11
Total load from slab P7 = 13.15 *kN/m*
Total load from slab P8 = *18.77kN/m*
31.92kN/m

/,31.9211Mn

	(5) 4m		(6) 4m		(l)4m (8)		(9)4m		(10) 4m	
K	~ x 3.4=0.188		~=0.25		~=0.25		~=0.25		~~.4=O.l88	
DF	0	0.43	0.57	0.5	0.5	0.5	0.5	0.57	0.43	0
FEM	-42.56	42.56	-42.56	42.56	-42.56	42.56	-42.56	42.56	-42.56	42.56
DM	42.56	--	---	--	--	--	---	---	--	42.56
COM		21.28	---	--	–	---	–	--	21.28	
DM		-9.2	12.13	–	--	--	---	12.13	9.2	
COM			----	6.07	--	---	6.07	---	---	---
DM		---	----	3.04	3.04	3.04	3.04	--	----	
COM			1.52	----	1.52	1.52	--	1.52	---	
DM		0.65	0.87	0.76	0.76	0.76	0.76	0.87	0.65	
EM		54.00	54.04	44.88	-44.84	40.28	4O.29	31.1	-31.13	
Elatic	63.8	63.8	63.8	63.8	63.8	63.8	63.8	63.8	63.8	63.8
static sh	-13.5	13.5	2.3	2.3	1.14	1.14	2.3	2.3	7.78	7.28
Total	50.3	77.3	66.1	61.5	64.04	62.7	66.1	61.5	71.6	56.02
Reaction		143	.4	125	.54	128	.8	133	.1	
SM	39.6		14.4		21		28.2		49.2	

BENDING MOMENT DIAGRAM

54.0kN 44.83kN 40.2kN 31.3kN

dA j ~ /|~ ~ |~

"li11 'l.LIV \!j__LJ) \lJY \jj__y

39.6kN 14.4kN 28.2kN 49.2kN

SHEAR FORCE DIAGRAM

66.1kN 64.04kN 66.1kN 71.6kN

50.3kN

~ ~ ~ ~ ~

"\ ~

~ <, ~ ~ 56.0kN

77.3 kN 61.5 kN 627kN 61.5kN

REF	CALCULATION	OUTPUT
	SPAN 5-6 SECTIONT	
	REINFORCEMENT	
BS 8110	k = M / Fcnbd2 = 39.6x106 / 25x79Ox4072 = 0.012	
Section 3.4,44	K = 0.012 < 0.156	
	Z = O.95x407 = 387mm2	z=387mm
	As = M / 0.87fyz = 39.6x106 / 0.87x410x387 = 287mm2	area = 287mm2
	Provide 2 Y 16 (As prov = 402mm2)	2 Y 16
	Support *5/6* *M=54.0kn/m*	
	K =M / bd2fcn = 54 x 106 / 230x407~5 =0.057	
	Z = 0.951 = 0.95 x 407 = 387	As req = 391mm2
	Asreq=~ / 0.87fyz = 54xl06 / 0.87x41Ox387 = 391mm2	As prov=402mm
	Provide 2 Y 16 (As prov = 402mm2)	2Y16
	CHECK FOR SHEAR Vmax=77.3	
	Shear stress V = v/bd *< 4N/mm2* = 77.3 x 103 / 23Ox407 = *mO.83N/mm2*	
	100 As prov / bd = 10Ox402 / 23Ox407 = 0.43	
	Vc = 0.71	
B88110 Rent I Table 3.9	Since V > vc shear reinforcement is required ASv / Sv = b(v-vc) / 0.87x250 = 230 (0.83-0.71) / 0.87x250 0.13	
	Provide YI0 @300 (As prov 0.523)	
	Span 6 *-71 7-818.9* M=49.2kn1m	

J

J

REF	CALCULATION	OUTPUT

Reinforcement

$K = \frac{M}{f_{cn}bd^2} = \frac{49.2x10^6}{25x79Ox407^2} = 0.015$

Z = 0.95x407 = 387mm

$As\ req = \frac{M}{0.87f_yz} = \frac{49.2x10^6}{0.87x410x387}$

$= 356mm^2$

Provide 2Y16 (As prov = $402mm^2$)

CHECK FOR DEFLECTION

$Mlbd^2 = \frac{49.2x10^6}{230x407^2}$

$= 1.29N/mm^2$

$FS = \frac{5/8 \times 410 \times 356}{402} = 227N/mm2$

$M.F = 0.55 + \frac{477 - 227}{120(0.9+1.29)}$

M.F= 1.6

Limiting span = 1.6x26 =41.6

Actual span = $\frac{4000}{407}$ = 9.83

Since 9.83 < 41.6 deflection is ok

Beam 7

Span 5 - 6 -7 = 7 - 8 = 8 - 9 = 10.67kn/m

Span 9 10 total load = 19.83 *kn/m*

L

REF | CALCULATION | OUTPUT

K = 0.04 < 0.156 k=O.04

Z=0.95 x 407

= 387mm

As = M = 13.1×10^6

0.87fyz 0.87x410x387

$95mm^2$

Provide 2 Y 16 (As prov $402mm^2$)

SUPPORT *5/6*

M= 18.51 *kn/m*

K=M k= 18.51×10^6 =0.05

fcnbd" $25 \times 790 \times 407^2$

k = 0.05 < 0.156

Z = 0.95 x 407

Z=387mm

As=M = 18.51×10^6

0.87fyz 0.87x410x387

= $134mm^2$

Provide 2 Y16 (As prov = $402mm^2$)

CHECK FOR SHEAR

V max = 20.95kn

Shear stress v = vlhd < *4N/mm²*

20.95×10^3 = *0.22N/mm*

230x407

VC=0.42

Since Vc > V shear is not required

Span 6 *-7/7-8/8.919-101*

M=34.9kN/m

K=M = 34.9×10^6 =0.010

$Fcubd^2$ $25 \times 790 \times 407^2$

K = 0.010 < 0.156

Z= 0.95 x 407

= 387mm

As=M = 34.9×10^6

0.87fyz 0.87x410x387

Provide 2 Y16 (As prov = 402mm1

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

V max = 37.l6kn

Shear stress = V v/bd = < *4N/mm2*

= 37.16xl03 = *O.39N/mm*

23Ox407

VC =54

Since VC > 0.39 shear reinforcement is nor required

REAM 12

SpanA-B total load = *16kN/m*

SpanB-C total load = *16kN/m*

SpanC-D total load = *16kN/m*

SpanD-E total load = *16kN/m*

SpanE-F total load = *16kN/m*

SpanF-H total load = *19.83kN/m*

16kn1m /' 19.8:tMn

	(A) 4m		(B)4m	(C)	4m		(D)4m	(E)	(F)4m		6	(II)
K	';4 x% =0.188		1~ =0.25		';4 = 0.25		';4 = 0.25		';4 = 0.25		1/6 x %=0.125	
DF	0	0.43	0.57	0.5	0.5	0.5	0.5	0.57	0.5	0.34	0.66	0
FEM	21.33	21.33	21.33	21.33	21.33	21.33	~1.33	21.33	21.33	21.33	-59.49	59.49
DM	21.33	0	0	---	---	---	---	---	----	12.97	25.19	25.19
COM		10.67	---	---	---	---	---	---	6.49	---	29.75	12.60
DM		4.59	6.08	---	---	---	---	3.25	3.25	10.12	19.64	
COM			-----	3.04	---	---	1.63	----	5.6	1.63	---	
DM		-----	-------	1.52	1.52	0.82	0.82	2.8	2.8	0.55	1.08	
COM			0.76	---	0.41	0.76	1.4	0.41	0.29	1.4	_._---	
DM		0.33	0.43	0.91	0.21	0.32	0.32	0.06	0.06	0.48	0.92	
EM		27.74	-27.74	19.6	19.6 .	23.23	20.42	21.23	20.66	24.02	24.03	
Elatic	32	32	32	32	32	32	32	32	32	32	59.49	59.49
Static sh	-6.94	6.94	-6.94	6.94	-6.94	6.94	-6.94	6.94	-6.94	6.94	4.01	4.01
Total	25.06	38.94	25.06	38.94	25.06	38.94	25.06	38.9	25.06	38.9	63.5	55.48
Reaction		64		64		64		64				
SM	19.63		27		24		26.1		23.27		77.6	

BENDING MOMENT DIAGRAM

27.74kN 19.16kN 23.23kN 21.23kN 24.03kN

19.63kN 24kN 26kN 23.29kN 77.6kN

SHEAR FORCE DIAGRAM

25.06kN 25.06kN 25.06kN 63.5kN

25.04kN

55.48kN

38.94kN 38.94kN 38.94kN 38.98kN 36.99kN

~ 77 '1 .r

REF	CALCULATION	OUTPUT
	SPANA-B	
BS SIlO	REINFORCEMENT	
Part	K=M = 19.64xl06 =0.06	
Section	fcnbd' 25x79Ox4072	
3.4,4.4	Z = 0.95 x 407	k=O.06
	K = 0.06 < 0.156	
	As=M = 19.63xl 06	
	0.S7xfyz 0.S7x410x3S7	z=387mm
	Provide 2 Y 16 (As prov = 402mm2)	2 Y16
	SUPPORTA-B	
	M = 27.74 *kn/m*	
	K=M = 27.74xl06 = O.OS	
	fcnbd2 25x79Ox4072	
	As = M = 27.74x106	
	0.87fyz 0.87x410x381	
	= 210mm2	
	Provide 2 Y 16 (As prov = 402rnm2)	
	CHECK FOR SHEAR	
BS SIlO	V max = 3S.94kn	
Part 1	Shear stress v = v/bd < *4N/mm2*	
Section	= 3S.94x103 = 0.4INmm2	
3.4.52	23Ox407	
1997	VC = 0.55 *N/mm2*	
	Since VC > V shear reinforcement is not required.	

~

Beam 16 span A-B Totalload=27.7kN/m Span B-C = Span C-D Totalload=40.l7kN/m Span D-E =37.18 Span E-F=37.1 *SQanF-H=41.4IkN/m*

27.7kN/m *40.l7kN/m* 37.18 *kN/m* *37.70kN/m* 41*Al kN/m*

	A		B		C		D		E		F	H
K	\4 x 3/4 =0.188		1/4 = 0.25		\4 =0.25		\4 =0.25		1/4 = 0.25		1/6x~=0.125	
DF	0	0.43	0.57	0.5	0.5	0.5	0.5 0.57		0.5	0.34	0.7	
FEM	-50.27	50.27	53.56	-53.56	53.56	-53.56	~9.53	-49.53	50.27	-50.27	124.4	-124.4
DM	50.27	1.41	1.88	......-	----	2.02	2.02	0.37	0.37	22.24	51.9	124.4
COM			-··_-	0.94	1.07	------	0.19	1.01	11.12	0.9	----	
DM		......._-	----_-	0.04	0.04	0.095	0.095	5.06	5.06	0.06	0.133	
COM		-_--	0.02	....	0.05	0.02	2.53	0.05	0.03	2.53	---	
DM		0.08	0.011	0.03	0.03	1.26	1.26	0.01	0.01	0.76	1.8	
EM	0	51.6	51.6	54.57	54.55	52.73	52.0	53.89	-53.76	74.41	74.2	
Elatic	75.4	75.4	80.34	80.34	80.34	80.34	74.2	74.2	75.4	75.4	124.4	124.4
Statich	-12.9	12.9	0.74	0.74	0.46	0.46	0.47	0.47	-5.2	5.2	18.55	18.55
Total	62.5	88.3	79.6	81.8	80.8	79.88	73.73	74.67	70.2	80.6	143	105.9
Reaction		167.9		161.9		153.61		144.87		223.6		
SM	51.81		27.27		26.7		21.16		11.7		172	

BENDING MOMENT DIAGRAM

REF	CALCULATION	OUTPUT
	SPANA-B	
	M = *51.86kN/m*	
	REINFORCEMENT	
	K = M / fcnbd2 = 51.8x10 6 / 25x790x407 2 = 0.016.	
	k = 0.016 < 0.156	
	z = 0.95 x 407 = 387mm	
	As=M / 0.87xfyz = 51.8xl0 6 / 0.87x410x387 = 375.3mm2	
	Provide 2 Y16 (As prov = 402mm2)	
BS 8110	SUPPORTB/C	
Part I	M = *54.57kn1m*	
Section	K=M / bd2fcn = 54.57x10 6 / 23 Ox407 2 x25 =0.057	
3.4.5.2		
1997	Z = 0.95d = 0.95 x 407 = 387mm	
	As req =M / 0.87fyz	
	= 54.57 x 10 6 / O.87x41 0x387	
	= 395mm 2	
	Provide 2 Y 16 (As prov = 408mm2)	
	CHECK FOR SHEAR	
BS 8110	V max = 88.3 kn	
Part 1	Shear stress v = vlbd < *4kN/mm2*	
Table 3.9	= 88.3xl0 3 / 230x407 = *0.94kN/mm 2*	
	VC = 0.67 k/mnr'	
	Since V > vc shear reinforcement is required	
	ASV / SV = b (v - vc) / 0.87x250 = 230 (0.96 - 0.67) / 0.87x250	
	= 0.2855	
	Provide YIO @300 As prov 0.23	
	SPANF-H	
	M=74.41 *kN/m*	
	REINFORCEMENT	
	K = M / fcubd2 = 74.l4xl 0 6 / 25x790x4072 = 0.023 < 0.56	

REF	CALCULATION	OUTPUT

Z = 0.95 x 407 = 387mm

As req = M / 0.87fyz = 74.41x10^6 / 0.87x410x387

= 539mm^2

Provide 3 Y 16 (As prov = 603mm)

M= 172 *kN/m*

REINFORCEMENT

K = M / fcubd^2 = 172x10^6 / 25x790x407^2 = 0.051

k = 0.051 < 0.156

z=0.95 x 407 =387mm

as req =M / 0.87fyz = 172x10^6 / 0.87x410x387

1.246mm^2

Provide 4 Y 20 (As prov 1200mm^2)

CHECK FOR SHIER

V max = 14 3kn

Shear stress v = v/bd < *4N/mm^2*

- 143 x 10^3 / 23Ox407 = *1.53N/mm*

VC=0.87

Since V > vc shear reinforcement is required

ASV / Sv = b(v-vc) / 0.87x250 x 230(1.53 - 0.87 / 0.87x250

=0.69

Provide YI0 @ 225 As prov 0.698

REF | CALCULATION | OUTPUT

4.6 DESIGN OF FLOOR BEAM

(1) Beam self weight = 0.3xO.23x24=1.656kNm
(2) From slab (PI) = lh n be = lh xll.04x4 = *13.65kN/m*
(3) Block wall = 3x3.47xl.4 = 14.57
(4) Remedy = 0.45 x 0.3 x 1.4 = 0.189
30.065

LOADING
Span 1 -2 *31.0kN/m*

BEAMl
(1) Beam selfweight- 0.3xO.23x24 = *1.656kN/m*
(2) Block slab (PI) = lh n be = lh x 11.64x4 = *15.65kN/m*
(3) Block wall = 3 x.47x1.4 =14.57
(4) Rendering = 0.45 x 0.3 x 1.4 = 0.189
= *32.065kN/m*

Span 2-3
(1) BM self weight = 1.656
(2) From slab (P2) = 15.65
(3) Block wall = 14.57
(4) Rendering = 0.189
32.065kN/m

Span 3-4
(1) BM self weight = 1.656
(2) From slab (P2) = 15.65
(3) Block wall =14.57
(4) Rendering = 0.189
32.065kN/m
Span 4 - 5 = span 1 -2 = *32.065kN/m*

. / *32.1kN/m*

BENDING MOMENT
At 1st intoner support = MB = MC = O.lIFL
Where F = qL = 32.1x4 = *1284kN/m*
MB = 0.11 x 128.4x4 '= *56.50kN/m*
MC = -0.11 x 28.4x4 = 56.50kNm

REF	CALCULATION	OUTPUT

AT MID SPAN

MA - B = O.09FL= 128.4x4xO.9= 46.22

MB - C = O.07FL= 35.95

MC - D = 0.07FL = 35.95

MD - E = 0.09FL = 46.22

SHEAR FORCE

VA = 0.45F = 0.45xI28.4 = 57.78kN

VB = 0.6F = 0.6 x 128.4 = 77.04kN

VC = 0.6F = 0.6 x 128.4 = 77.04kN

VD =0.6F = 0.6 x 128.4 = 77.04kN

VE = 0.45F = 0.45xI28.4 = 57.78kN

MAIN REINFORCEMENT

LBeam

Over all depth = 450mm

Web with (bw) = 230mm

Flange breath, bf= bw+ 1110(0.7L)

= 230 + *1/10* x 0.7 x 4000 = 510

Effective depth d = 450 - 25 - 6.0/2

= *450-25-10-16/2*

=407mm

Concrete cover = 25mm

Fcu = *25N/mm2*

Fy = 410 *N/mm2*

Durability and fire resistance = 1 hr.

At support B M = 56.50 *kn/m*

Mu = 0.l56 bd'fcu

Mu = 0.156 x 230 x 4072x 25

Mu = 148.6 knm

Mu > M no compression reinforcement required

K = M = 56.50xl06

Bd2fcu 230x4072x25

K = 0.059 < 0.l56

Z = 0.95 x 407 = 387mm

As req = 56.50xl 06

0.87x410x387

As = 409.1mm2

Provide 3 Y 16 As prov 603mm

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

V=77.04kn

V=V = 77.4x10[3] =*0.823N/mm2*

bd 23Ox407

100AS = 100x603 = *0.64N/mm2*

bd 23Ox407

V=0.65

ASV = b (v-OO)

Sv 0.87fyv

= 280 (0.823 - 0.65)

0.87x250

= 0.183

Provide Y 10 @ 300 (As prov 0.523)

Beam 2 (Grid BIl-5)

Loading

BEAM 2

Span 1-2

(1) Beam self weight = *1.656kN/m*

(2) From slab (PI) = Ih n be = Ih x 11.64x4 = *15.52kN/m*

(3) From slab P4 = Ihnbx = 1/s xll.64x4 = *15.52kN/m*

(4) Block wall = 3 x 3.47x1.4 = *14.57kN/m*

(5) Rendering = 0.45xO.3x1.4 = 0.189

47.46 kN/m

47.5kN/m

Span 2-3

(1) BM self weight = 1.656

(2) From slab 4 = 15.52

(3) from slab 5 = Y2nbx (1-1I3k2) = 21.34

(4) Block wall = 3 x 3.47x1.4 = 14.57

(5) Rendering = 0.45x0.3x1.4 = 0.189

53.3kN/m

Span3-4

(1) BM self weight = 1.656

(2) From slab 4 = 15.52

(3) from slab 5 = 21.34

(4) Block wall = 14.57

(5) Rendering = 0.189

53.3kN/m

Span 4 - 5 = span 1 -2 = *47.5kN/m*

51.6kN 54.57kN 52.0kN 53.89kN 74.41kN

51.81kN 26.7kN 21.16kN 11.7kN 17.2kN

SHEAR FORCE DIAGRAM

79.6kN 80.8kN 70.2kN 14.3kN

67.8kN

88.3kN 81.1kN 79.88kN 74.67kN 80.6kN 195.9kN

REF CALCULATION OUTPUT

47.5kN 53.3lcN 47.5lcN

_L-y--~ L_

	(1) 4m		(2) 4m		(3)4m	(4)	(5) 4m	
K	\4 x 0/..-0.188		\4 =0.25		\4 =0.25		\4~..4=().I88	
DF	0	0.43	0.57	0.5	0.5	0.5	0.43	0
FEM	62.7	62.7	-71.06	71.06	-71.06	71.06	62.7	62.7
DM	62.7	3.6	4.77	-	--	4.8	3.6	62.7
COM		--	---	2.39	2.4	--	---	
DM		--	---	0.05	0.05	---	---	
COM		--	0.025	----	---	0.025	---	---
DM		O.otl	0.0143	--	---	0.0143	0.01	
EM	0	66.31	-66.3	73.5	73.41	66.30	66.30	
Elastic shear	95	95	107	107	107	107	95	95
Static shear	-16.6	16.6	-1.8	-1.8	1.8	1.8	16.6	16.6
Total	78.4	112	105.2	108.8	108.8	1052	112	78.4
Reaction		217		217.6		2172		
SM	65		37.5		37.5		65	

BENDING MOMENT DIAGRAM

66.31kN 73.5kN 66.30kN

~

65kN 37.5kN 37.5kN 65kN

SHEAR FORCE DIAGRAM

105.2kN 108.8kN 112kN

7g.4kN~

~ ~ ~

"\J, "4J

~ ~ 78.4kN

112kN 108.8kN 108.8kN

REF	CALCULATION	OUTPUf

"

YAB= wI + (MA-MBIL) = 47.5x4 + (0.-66.31) = 78.4kn
2 2 4

YAB= wI + (MB -MAIL) = 47.5x4 + (66.31-0) = 112kn
2 2 4

YBC= wI + (MB - MCIL) = 53.3x4 + (66.3-73.5) = 107-1.8 =105.2
2 2 4

YCB=wI +(MC-MBIL)=53.3x4 +(73.5-55.3) =108.8
2 2

Yco=wl +(MC-MDIL) = 108.8kN
2

Voc=wl +(MC-MDIL) =105.2kN
2

YOE = wi + (Mc - MDIL) = 112.4kN
2

MAXIMUM MOMENT

MAB= y2 AB/2w /MAl = (78.4)2 - 0 = 65kN/m
2(47.5)

MBc = y2 AC/2w IMB/= (105.2i - 66.3 = 37.5kN/m
2(53.3)

Mev = y2 CD/2w fMC/= (108.8)2 -73.41 = 37.5kN/m
2(53.3)

MOE = y2 *DE/2w !MDI* = (78.4)2 - 0 = 65kN/m
2(47.5)

Span 1-2

M=65kN/m

Section L

hf

,---------, ~ hf

~hw+

BF = bw + 0.1 (0.7L) = 230 + 0.1 (0.7 x 4000) = 510mm

REF	CALCULATION	OUTPUT
	REINFORCEMENT	
BS 8110	*M=65kN/m*	
Parts		
Section	K=M = 65xl0^{6} =0.031	
Parts	fcubd" 25x510x4072	
Section	k = 0.031 < 0.156	k=0.031ok
3.4,4.4	Z = 0.75 x 407 = 387mm	
	As req =M = 65xl0^{6}	
	0.S7fyz 0.87x410x3S7	As req =471mm2
	= 471mm2	As prov=eoamm"
	Provide 3 Y 16B (As prov 603mm2)	
	Support 112	
	M=66kN/m	
	K=M = 66.31xl0^{6} =0.031	
	bd2fcn 25x51 Ox4072	
	Z = 0.95x407 = 387mm	
	As req =M.__ = 66.31xl0^{6}	
	0.S7fyz 0.S7x410x387	
	=4S0mm2	
	Provide 3 Y T (As prov = 603mm2)	
	CHECK FOR SHEAR	
	Vmax= 112kn	
	Shear stress V = v/bd < *4N/mm2*	
BS SUO		
Part 1	= 112 x 10^{3}	
Section	23 Ox407	
3.4.5.2	= 1.19N/mm2	
1997		
	100 As nrov = 100x603	
	bd 23 Ox407	
	=0.64N/mm2	
	Vc = *0.65N/mm2*	
BS81l0	Since V > vc shear reinforcement is required	
Rent 1	ASY = b(v-vc) = 230 (1.19-0.65)	
Table 3.9	Sv 0.S7x250 0.87x250	
	=0.57	

REF | CALCULATION | OUTPUT

CHECK FOR DEFLECTION

$M/bd^2 = \frac{65 \times 10^6}{23O \times 407^2}$

$= 1.71\ N/mm^2$

FS $= \frac{5/8 \times 410 \times 471}{603} = 200\ N/mm^2$

M.F $= 0.55 + \frac{477 - 200}{120(0.9+1.71)}$

M.F= 1.43

Limiting span = 1.43x26 =37.29

Actual span $= \frac{4000}{407}$ =9.83

Since 9.83 < 37.29 deflection is ok

Beam 3 (Grid 1-5)

Span 1-2

(1) Beam self weigh = *1.656kN/m*

(2) From slab P4 = 15.52 x 2 = *31.04kN/m*

(3) Block wall = *14.57kN/m*

(4) Rendering = 0.1*89kN/m*

47.45kN/m

,..

Span 2-4

(1) Beam self weigh = *1.656kN/m*

(2) From slab P5 = 21.34x2 = *42.68kN/m*

(3) Block wall = *14.57kN/m*

(4) Rendering = *0.189kN/m*

59. 1*OkN/m*

Span4-5

(1) Beam self weigh = *1.656kN/m*

(2) From slab P2 = *15.52kN/m*

(3) Block wall = *14.57kN/m*

(4) Rendering = 0.1*89kN/m*

50.50kN/m

REF CALCULATION OUTPUT

r>« L~''' ~1GNn ~ ▮▮▮
Y "Y Y -y -y-V Y"Y Y Y Y "Y ~

(1) 4m (2) 4m (4)4m (5)

K	\4x~-O.l88			. \4 =0.25		\4xl,4=O.188	
OF	0		0.6	0.4	0.4	0.6	0
FEM	62.3		62.3	-315.2	315.2	-67.3	67.3
OM	63.3		1511	100.8	- 99.2	14.9	67.3
COM			---	-49.6	50.4	--	
OM			29.76	19.84	-202	30.4	
COM			----	-10.1	9.93	---	--
OM			6.06	4.04	-4.0	-5.%	
COM				-2.0	2.0		
OM			1.2	0.8	-0.81	-121	
COM	0.41		0.4				
OM			0.25	0.164	0.16	0.24	
EM			251.7	-251.0	25.4	-254	
Elastic shea	94.9		94.9	236.4	236.4	1.01	1.01
Static shear	-63	63		0.75	0.75	63.5	-635
Total	31.9	156.9		236	2372	164.3	375
Reaction		39		2.9	4	01.7	
SM		11			222	13.9	

BENDING MOMENT DIAGRAM

25.1kN 2541N

;(l~

'1J IJY ~l~_ "II LY

llkN 222kN 13.9kN

SHEAR FORCE DIAGRAM

236kN 164.5kN

131.9kN lr~ ~lh",

~, ~ ~ 37.5kN

156.9kN 237.2kN

REF	CALCULATION	UTPUT

Span 2-4

M=222kN/m

Section =T

Bf= bw+ 0.2 (0.7L) = 230+0.2 (0.7x8000)
= 5,830.2

$K = \frac{M}{Bd^2fcu} = \frac{222x10^6}{25x5,830x407^2}$

K = 0.09 < 0.156

Z = 0.95 x 407

Z=387mm

$As. = \frac{M}{} = \frac{222x10^6}{0.87x410x387}$

= 1,608mm2

Provide 4 Y 25mmB (As prov 1960mm2)

Support 214

M=254kN/m

Section =T

$K=\frac{M}{Bd^2fcu} = \frac{254x10^6}{25x5,830x407^2}$

K = 0.010 < 0.156

$As\ req = \frac{254x10^6}{0.87x410x387}$

= 1840mm2

Provide 4 Y 25mmT (As prov 1960mm2)

CHECK FOR SHEAR

V max = 237.2kn

Shear stress v = vlbd = $\frac{237.2x10^3}{230.0x407}$

=2.5N/mm2

$\frac{100As\ prov}{bd} = \frac{10Ox1608}{230x1960}$ = *0.36N/mm2*

VC = 0.53

Since v > vc shear reinforcement is required

$\frac{ASV}{fy} = \frac{b\ (v-vc)}{0.87fy} = \frac{230\ (2.5-0-53)}{0.87x250}$

=2.1

Provide Y 10 @ 90 (As prov 2.1)

REF CALCULATION UTPUT

CHECK FOR DEFLECTION

$M/bd^2 = \frac{222x10^6}{230x407^2}$

$= 5.8\ N/mm^2$

FS = $\frac{5/8 \times 410 \times 1.608}{1960}$ = 21 N/mm^2

M.F = 0.55 + $\frac{477 - 200}{120(0.9+1.71)}$

M.F = 0.55 + $\frac{477 - 210}{120(0.9+3.7)}$

M.F= 1.03 < 2

Limiting span = 1.03x26 =26.78

Actual span = $\frac{8000}{407}$ = 19.66

Since 19.66 < 26.78 ok deflection is ok

Span4-

M = *13.9kn/m*

Section =L

BF=bw+O.1 (0.7L) = 230 +0.1 (0.7x4000)

= 510

= $\frac{13.94x10^6}{25x510x407^2}$

K = 0.06 < 0.156

= 0.95x407 = 387

As req = $\frac{M}{0.87fyz}$ = $\frac{13.9x10^6}{0.87x410x387}$

= $101mm^2$

Provide 2 Y 16 B (As prov $402mm^2$)

Beam 14 (Grid 3 El - H)

SpanE[1]-H

(1) Beam self weight = *1.656kN/m*

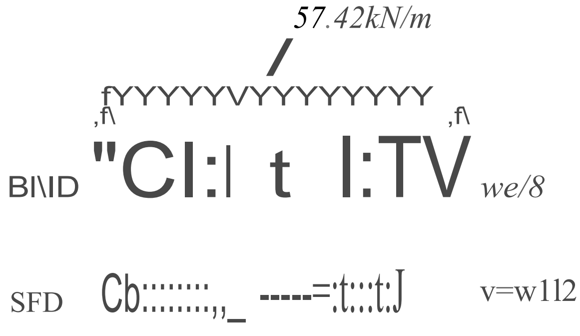
(2) From slab = *41kN/m*

(3) Block wall = *14.57kN/m*

(4) Rendering = *0.189kN/m*

57.42 kN/m

REF CALCULATION UTPUT



Since v > vc shear reinforcement is required

ASV = b (v-vc) =230 (2..24-0.98)
fv 0.87x250 0.87x250

= 1.3324

Provide Y 10 @ 100 (As prov 1.57)

Beam 8

Span 2 - 3 total load

(1) Beam self weigh = *1.656kN/m*
(2) From slab = *24.6kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = 0.1*89kN/m*
41.02kN/m

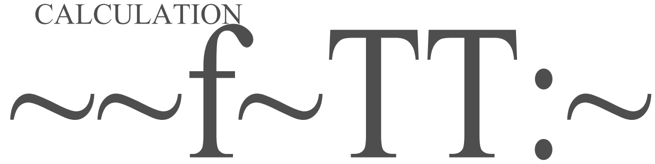
Span 3' - 4 total load

(1) Beam selfweigh = *1.656kN/m*
(2) From slab = *19.83kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = 0.189kN/m
36.25kN/m

Span 4 - 5 total load

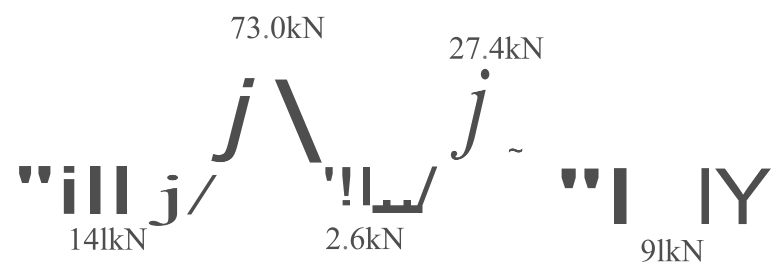
(1) Beam self weigh = *1.656kN/m*
(2) From slab = *39.70kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = 0.189kN/m
56.12kN/m

REF	CALCULATION	OUTPUT

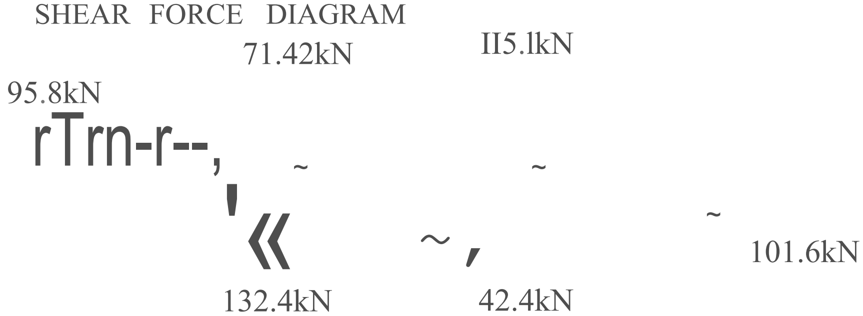


	(2) 4000	1000	(3) 3.140	(G)	3860	(5)
K	lJzX% = 0.373		l/s.3 = 0.188		l/2.7x % =0.27	
DF	0	0.28	0.72	0.38	0.62	0
FEM	-94.7	94.7	-29.78	29.78	-69.7	69.7
DM	94.7	18.2	47	15.2	25	
COM		---	7.6	24	-----	
DM		2.13	5.5	9.12	15	
COM		---	4.47	2.75	---	
DM		1.25	3.22	1.05	1.71	
COM		---	0.51	1.61		
DM		0.14	0.37	0.61	0.99	
EM	0	73.0	-73.0	27.4	-27	
Elastic Shear	114.1	114.1	56.9	56.9	108.3	108.3
Static shear	-18.3	18.3	14.52	14.52	6.75	-6.75
Total	95.8	132.4	71.42	42.4	115.1	101.6
Reaction		20	4	15	8	
SM	141		2.6		91	

BENDING MOMENT DIAGRAM



SHEAR FORCE DIAGRAM



REF	CALCULATION	UTPUT

i

Section T

Reinforcement

M= *141kN/m*

k = M = 141xl06

fcubd2 25x790x4072 = 0.043

K = 0.043 < 0.156

Z=0.95x407

Z=383mm2

As =M._ = 141xl06

0.87fyz 0.87x410x383

= 1.021mm2

Provide 4 Y 20 B (As prov = 1260mm2)

Support *213*

M=73kN/rn

k=M = 73xl06 = 0.02

bd'fcn 25x790x4072

k = 0.02 < 0.156

As = M_ = 73.xl06

0.87fyz 0.87x400x387

= 529mm2

Provide 3 Y 16 (As prov = 603mm2)

CHECK FOR SHEAR

Vrnax:= 115kn

Sheer stress V = v/bd *< 4N/mm2*

V = 115x103 *= 1.2N/mm*

23Ox407

100 As = 10Ox603 = 0.64

bd 23Ox407

V *= 0.65N/mm2*

Since V > vc shear reinforcement is required

ASV = b(Y-vc) = 230 (1.2-0.65)

SV 0.87x250 0.87x250

=0.58

Provide Y 10 @ 250 (As prov = 0.628)

BEAM 15

Span A - B total load

(1) Beam self weigh = $1.656kN/m$

(2) From slab 1 = $15.65kN/m$

(3) Block wall = $14.57kN/m$

(4) Rendering = $0.189kN/m$

32. $1kN/m$

Span B - C = CD = D - F = E - F = 32.1kN/mm1

Span F - H total load

(1) Beam self weigh = $1.656kN/m$

(2) From slab = $19.83kN/m$

(3) Block wall = $14.57kN/m$

(4) Rendering = $0.189kN/m$

36.26kN/m

REF | CALCULATION | UTPUT

57.42kN/m

/

O O O O O O O O O O O O O O

t 7333 It

We = 57.42x7.3^2 = 383kN/m

8 8

K=M = 383xl0^6 = 0.117

Fcubd^2 25x790x407^2

K = 0.117 < 0.l56

Z = 0.9Sx407 = 387mm

As=M = 383xl0^6

0.87fyz 0.87x410x387

= 2,775mm^2

Provide 6 Y 25mm (As prov 2980mm^2)

Shear V = O.Sx 7.3 x 57.42

= 210kN

V = y =21 Ox10^3

bd 230x407

v = *2.24N/mm^2*

lOOAs = 100x29S0 = 3.1S

bd 23 Ox407

VC=0.98

~32.111Mn

»> 38.3kn

y--.., (t y y y .'\.

4 4 4 4 4 6

K	t,4x~=0.188		t,4= 0.25		t,4=0.25		t,4=0.25		t,4=0.25		1/6 x 34=0.125	
DF	0	0.43	0.57	0.5	0.5	0.5	0.5	0.5	0.5	0.34	0.66	0
FEM	-42.8	42.8	-42.8	42.8	-42.8	42.8	-42.8	42.8	-42.8	42.8	-108.9	108.9
DM	42.8	......-	------	----	....._...	----	------	.._--	------	22.5	43.6	108.9
COM		21.4	------	----	------	----	------	.._--	11.25	--_..	54.45	21.S
DM		9.20	12.20	----	....	----	--_.._-	5.63	-5.63	1S.51	36	
COM		-_.....	._--_ ..	6.10	------	----	2.S2	...._--	9.3	2.S2		
DM		----	------	3.05	3.0	1.41	1.41	4.7	4.7	1.0	1.9	
COM			1.53	----	0,71	1.53	2.35	0.71	0.5	0.71	.._..-	
DM		0.66	0.87	0.36	0.36	0.41	0.41	0.11	0.11	0.24	0.47	
EM		54.44	-54.44	40.11	-40.1	43.43	-42.0	31.97	-33.07	44.5	-44.46	
Elastic shear	64.2	64.2	64.2	64.2	64.2	64.2	64,2	64.2	64.2	64.2	108.9	108.9
Static shear	-27.22	27.22	3.6	3.6	0.83	0.S3	2.S1	2.51	2.8	2.S	22.24	-22.24
Total	36.98	91.42	67.8	60.6	63.37	65.03	66.71	61.69	61.4	67.0	131.14	86.66
Reaction				12.4								
SM	21.3		17		22		27		25		103	

BENDING MOMENT DIAGRAM

544.4kN 40.1kN 43.43kN 33kN 45kN

21.3kN 22kN 27kN 25kN 103kN

SHEAR FORCE DIAGRAM

67.8kN 63.37kN 66.71kN 61.4kN 131.14kN

36.98kN

91.42kN 60.6kN 65.03kN 61.69kN 67.0kN 86.66kN

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

Vmax = 91.42kN

Sheer stress V = v/bd < *4N/mm2*

V = 91.42x10^3 / 23Ox407 = *0.97N/mm*

100 As / bd = 10Ox402 / 23Ox407 = 0.43

Ve51

ASV / SV = b(V-vc) / 0.87x250 = 250 (0.97 - 0.51) / 0.87x250

=0.53

Provide Y 10 @ 275 (As prov = 0.571)

Beam 10 = beam 11 (Grid G)

Span 5-6

(1) Beam self weigh = *1.656kN/m*

(2) From slab 7 = *13.15kN/m*

(3) Form slab 8 = *18.77kN/m*

(4) Block wall = *14.57kN/m*

(5) Rendering = *0.189kN/m*

48.34kN/m

Span 6 - 7 = 7 - 8 = 8 - 9 = *48.3kN/mm*

REF	CALCULATION	OUTPUT

~/-'V~-'v'-'·v~-V·'7<:'"v-"""'~r--...~

	(5) 4m		(6) 4m		(7)4m	(8)	(9) 4m	
K	~ x 3;'= 0.188		~=0.25		~=0.25		~ x 3;'=0.188	
DF	0	0.43	0.57	0.5	0.5	0.57	0.43	0
FEM	-64.45	64.45	-64.4564.45		-64.4564.45		-64.45	64.45
DM	64.45	--	---	---	---	---	----	64.45
COM		32.23		--	--.--		32.23	
DM		-13.9	18.4	--	----	18.4	13.9	
COM		--		-9.2	-9.2	---	---	
DM		---	----	---	---	---	---	
EM	0	82.80	-82.80	55.25	-552546.05		-46.12	
Elatic	96.69	96.68	96.69	96.68	96.69	96.68	96.69	96.68
Static sh	-41.42	41.42	6.89	6.89	2.3	2.3	11.53	-11.53
Total	55.26	138.1	103.6	89.8	98.9	94.4	108.2	85.2
Reaction		241	.7	18	8.7	20	2.6	
SM	31.6		28		46		75	

BENDING MOMENT DIAGRAM

;,·

82.82kN 55.25kN 46.1kN

~

31.6kN 28kN 46kN 75kN

SHEAR FORCE DIAGRAM

1O5.2kN 1O8.8kN 112kN

55.26kN~ ~ ~ ~

~ ~ ~ ~kN

138.1kN 89.8kN 94.9kN

REF	CALCULATION	OUTPUT

Reinforcement

k = M = 31.62x106
fcubd2 25x790x4072 =kO.09

K=0.09<0.156
Z = 0.95x407 = 378mm2

As =M_ =31.62x106
0.87fyz 0.87x410x378
= 236mrn2

Provide 2 Y 16B (As prov = 402mm2)

SupportS/6
M = *82.82kn1m*
Section T

k=M = 82.82x106 =0.03
bd2fcn 25x79Ox4072
k=O.03 <0.156
Z=0.95 x 407
Z=387mrn

r:

As req = M_ = 82.82.x106
0.87fyz O87x41Ox387
= 560mm2

Provide 3 Y 16T (As prov = 603mm1

SPANS-9
M=75kn/m
Section T

k=M = 75x106 =0.023
bd'fcn 25x79Ox4072
k = 0.023 < 0.156

Z=0.95 x 407
Z=387mrn

As req = M_ = 75x106
0.87fyz 0.87x410x387
= 543mm2

Provide 3 Y 16T (As prov = 603rom2)

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

Vmax = 138.1kn

Sheer stress V = vlbd < *4N/mm2*

$$V = \frac{138.1x10^3}{230x407} = 1.48N/mm^2$$

$$\frac{100 As}{bd} = \frac{100x560}{23Ox407} = 0.59N/mm2$$

VC = *0.63N/mm2*

$$\frac{ASV}{SV} = \frac{b(Y-vc)}{0.87x250} = \frac{250 (1.48-0.63)}{0.87x250}$$

=0.98

Provide Y 10 @ 150 (As prov = 1.047)

Beam 7 (Grid H)

Span 5-6

(1) Beam self weigh = 1.656kN/m

(2) From slab 7 = *10.67kN/m*

(3) Block waH = *14.57kN/m*

(4) Rendering =*0.189kN/m*

27.10kN/m

Span 6 - 7 = 7.8 =*29.93kN/mm*

Span 9-10

(1) Beam self weigh = 1.656kN/m

(2) From slab 7 = *19.83kN/m*

(3) Block wall = *14.57kN/m*

(4) Rendering = *O.189kN/m*

36.25kN/m

REF	CALCULATION	OUTPUT

Reinforcement

M=36.8kN/m

Section L

BF=510mm

k = M = 36.8x106

fcubd2 25x510x4072 = 0.07

K = 0.017 < 0.156

Z = 0.95x407 = 387mm2

As req = M = 36.8x106

0.87fyz 0.87x410x387

= 267mm2

Provide 2 Y 16B (As prov = 402mm2)

Support *5/6*

M= *51.9kN/m*

k=M = 51.9x106 = 0.015

bd2fcu 25x790x4072

k=0.015 <0.156

Z = 0.95 x 407

Z=387mm

As req =M = 51.9x106

0.87fyz 0.87x410x387

= 376mm2

Provide 2 Y 16T (As prov = 402mm2)

SPAN9-10

M=57kN/m

REINFORCEMENf

k = M = 57x106 = 0.027

bd2fcu 25x510x4072

k = 0.026 < 0.156

Z = 0.95 x 407 = 387mm

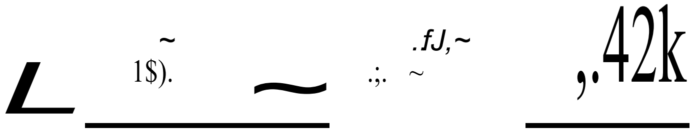
As req = M = 75x106

0.87fyz 0.87x410x387

= 543mm2

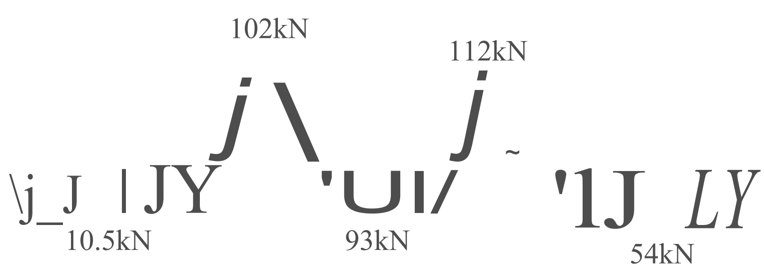
Provide 3 Y 16T (As prov = 603mm~

REF	CALCULATION	OUTPUT

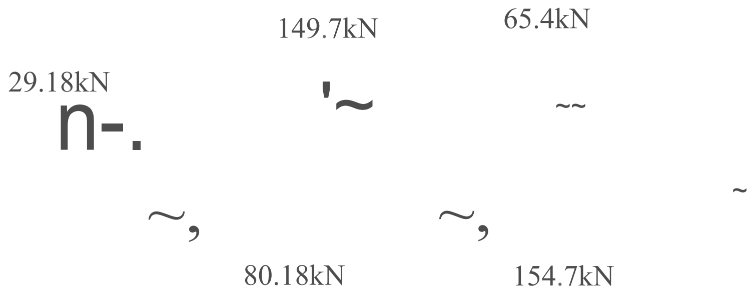


	(E) 2667		(EI) 5333	(G)	2000 (H)	
K	1/27x % = 0.27		l/s.3 =0.188		1/1)< % =0.38	
DF	0	0.5	0.41	0.33	0.67	
FEM	-24.6	24.6	134.4	134.4	12.5	12.5
DM	24.6	65	45	40.2	81.7	12.5
COM		--	20.1	22.5	----	
DM		11.86	8.24	7.43	15.1	
COM		–	3.72	4.12	----	
DM	2.2		1.5	1.4	2.8	
COM			0.7	0.75		
DM		0.41	2.9	0.25	0.5	
EM		102.1	-102	112.9	-112	
Elastic Shear	54.68	54.68	152.2	152.2	37.4	37.4
Static shear	-25.5	25.5	-2.5	2.5	28	-28
Total	29.18	80.18	149.7	154.7	65.4	9.4
Reaction		22	9.9	22	0	
SM	10.5		93		54	

BENDING MOMENT DIAGRAM



SID.:AK J4'UKl:E DIAGRAM



REF	CALCULATION	OUTPUT

Reinforcement

M=93kN/m

Section T

BF=790mm

k=M = 93xl06
fcubd" 25x79Ox4072 = 0.028

K = 0.028 < 0.156

Z = 0.95x407 = 387mm2

As req =M_ = 93xl06
0.87fyz 0.87x410x378
= 690mm2

Provide 3 Y 20B2 (As prov = 943mm2)

Support El - H

M= 112kn!m

Section

k=M = 112xl06 = 0.034
bd2fcn 25x79Ox4072

k = 0.034 < 0.156

Z = 0.95 x 407

As req =M_ = 112xl06
0.87fyz 0.87x410x378
= 830mm2

Provide 3 Y 20T (As prov = 943mm2)

SPANG-HI

M=54kn/m

REINFORCEMENT

k = M = 54xl06 = 0.026
bd2fcn 25x51 Ox4072

k = 0.026 < 0.156

As req = M_ = 54xl06
0.87fyz 0.87x410x378
=400mm2

Provide 2 Y 16B (As prov = 402mm1

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

Vmax = 154.7kn

Sheer stress V = vlbd < $4N/mm^2$

$V = \frac{154.7x10^3}{230x407}$ = *1.7N/mm2*

$\frac{100 As}{bd} = \frac{100x943}{23Ox407}$ = $1.0N/mm^2$

VC = *0.75N/mm2*

$\frac{ASV}{SV} = \frac{25O(Y-vc)}{0.87fyv} = \frac{250 (1.0-0.75)}{0.87x250}$

=0.287

Y 10 @ 300 (As prov = 0.523)

Beam 16 .

SpanA-B

(1) Beam self weigh = *1.656kN/m*
(2) From slab = *37.7kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = 0.1*89kN/m*
54.*12kN/m*

Span B- C

(1) Beam self weigh = *1.656kN/m*
(2) From slab =*40.l7kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = *0.189kN/m*
56.59kN/m

SpanC-D

(1) Beam self weigh = *1.656kN/m*
(2) From slab =*40.08kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = *0.189kN/m*
57.54kN/m

Span D - E = Span E - F

(1) Beam selfweigh = *1.656kN/m*
(2) From slab = *37.l5kN/m*
(3) Block wall = *14.57kN/m*
(4) Rendering = *O.189kN/m*
54.12kN/m

REF	CALCULATION	OUTPUT

CHECK FOR SHEAR

Vmax = SO.3kN

Sheer stress V = vlbd < 4N/mm2

V = SO.3x10 3 / 23 Ox407 = 0.S6N/mm2

100 As / bd = IOOx603 / 23 Ox407 = 0.64N/mm2

VC = 0.65N/mm2

ASV / SV = 25O(Y-vc) / 0.S7fyv = 250 (0.S6 - 0.65) / 0.S7x250

= 0.345

Provide Y 10 @ 300 (As prov = 1.523)

Beam 19

Span E_El

(1) Beam self weigh = 1.656kN/m
(2) From slab 7 = 24.08kN/m
(3) Block wail = 14.57kN/m
(4) Rendering = 0.189kN/m
40. 5OkN/m

r

Span E1_ G

(1) Beam self weigh = 1.656kN/m
(2) From slab 7 =41.0kN/m
(3) Block wail = 14.57kN/m
(4) Rendering = 0.1 89kN/m
57.42kN/m

BEAM 16

SpanG-H

(I) Beam self weigh = 1.656kN/m
(2) From slab 7 =21.0kN/m
(3) Block wall = 14.57kN/m
(4) Rendering =0.IS9kN/m
37.42kN/m

REF	CALCULATION	OUTPUT
	SpanF-H	
	(1) Beam self weigh = *1.656kN/m*	
	(2) From slab = *47.41kN/m*	
	(3) Block wall = *14.57kN/m*	
	(4) Rendering =*O.189kN/m*	
	58.54kN/m	

254.12k56.59 57.54 54.12 54.12 58.54

	4	B	4	C	4	D	4	E	4	F	6	H
K	\4 x 0/4 =0.188		\4 = 0.25		\4 =0.25		\4 =0.25		\4 =0.25		1/6 x 0/4=0.125	
DF	0	0.43	0.57	0.5	0.5	0.5	0.5	0.5	0.5	0.3	0.7	
FEM	-72.11	72.16	-75.5	75.5	-76.72	76.72	72.16	72.16	-72.16	72.16	-17.6	17.6
DM	72.16	1.44	1.9	0.61	0.61	2.28	2.28	--..	.._--	31.15	72.7	176
COM		36.08	0.31	0.95	-l.14	0.31	_-·.	1.14	15.58	---	88	
DM		15.4	20.4	0.1	0.1	0.16	0.16	7.22	7.22	26.4	62	
COM		-.._--	0.05	10.2	0.08	0.05	3.61	0.08	13.2	3.61	---	
DM		0.022	0.029	5.06	5.06	1.78	1.78	6.56	6.56	1.08	2.53	
COM			2.53	O.ot5	0.89	2.53	3.28	0.89	0.54	3.28	----	
DM		1.09	1.44	0.44	0.44	0.38	0.38	0.18	0.18	0.98	2.3	
EM		95.40	-95.0	82.75	-81.96	74.27	-75.74	85.59	-87.0	82.0	-82.13	
Elastic shear	108.2	108.2	119.2	119.2	115.1	115.1	108.2	108.2	108.2	108.2	175.6	175.6
Static shear	-23.85	23.35	3.1	3.1	1.9	1.9	2.47	2.47	1.25	1.25	13.7	-13.7
Total	84.35	132.1	122.3	116.1	117	113.2	111	105.7	109.1	107.5	189	162.1
Reaction		25	4.4	23	3.1	22	4.2	2	14.8	296		
SM	65.8		37		37		27		23		223	

BENDING MOMENT DIAGRAM

595kN 382.75kN 75.74kN 87.0kN 82kN

65.8kN 37kN 27kN 23kN 33.3kN

SHEAR FORCE DIAGRAM

122.3kN 117kN lllkN 109.1kN 189kN

84.35kN

132.1kN 116.1kN 113.2kN 105.7kN 107kN

REF	CALCULATION	OUTPUT

Reinforcement
M=65.8kN/m
Section T
BF=790mm

k=M = 65.8x106
fcubd" 25x790x4072 = 0.02

K = 0.02 < 0.156
Z=387mm2

As req = M__ = 65.8x106
0.87fyz 0.87x410x378
= 477mm2
Provide 3 Y 16B (As prov = 603mm2)

SupportA-B
M=95kN/m

k=M = 95x106
bd'fcu 25x790x4072
k = 0.03 < 0.156
Z=0.95 x 407
Z=387mm

As req = M_ = 95x106
0.87fyz 0.87x410x387
= 688mm2
Provide 4 Y 16mm2 (As prov = 804mm~

SPANF-H1
M=223kN/m

k=M = 223xl06 =0.068
bd2fcu 25x79Ox4072
k = 0.068 < 0.156
z = 0.95x407 = 387

As req = M_ = 223x106
0.87fyz 0.87x410x387
= 1 615mm2
Provide 4 Y 25~ (As prov = 1960mm1

REF CALCULATION OUTPUT

CHECK FOR SHEAR

Vmax= 189kN

Sheer stress V = vlbd < *4N/mm2*

$$V = \frac{189.7x10^3}{23Ox407} = 2.02N/mm2$$

$$\frac{100\,As}{bd} = \frac{100x1960}{23Ox407} = 2.09N/mm^2$$

VC = $1.06N/mm^2$

$$\frac{ASV}{SV} = \frac{250(Y-vc)}{0.87fyv} = \frac{250\,(2.02 - 1.06)}{0.87x250}$$

= 1.103

Provide Y 10 @ 125 (As prov = 1.256)

CHECK FOR DEFLECfION

$$M/bd^2 = \frac{68.5x10^6}{230x407^2}$$

= $1.8\,N/mm^2$

$$FS = 5/8 \times 410 \times \frac{477}{603} = 203\,N/mm2$$

$$M.F = 0.55 + \frac{477 - 203}{120(0.9+1.8)}$$

M.F= 1.4

Limiting span = 1.4x26 =37.1

$$\text{Actual span} = \frac{4000}{407} = 9.8$$

Since 9.8 < 37.1 deflection is ok

4.7 DESIGN OF COLUMN

Fy=410N/mm^2 b=230mm

Cc = 30mm h = 450mm

Feu = *25N/mm^2*

Col 1 on grid *41B* Designed as axial Loading (230mm x 450mm)

ROOF LEVEL - SIXTH FLOOR

Self weight = 0.4xO.230x24x1.4

= lO.4kN

REF	CALCULATION	OUTPUT
	reaction from beam & slab = 150.87 + 2236	
	= 374.47KN	
	Total = 384.87kN	Ni = 384.87kN
	SIXTH FLOOR LEVEL - FIFI'H FLOOR LEVEL	
	Self weight = 10.4kN	
	Beam slab = 217.2 + 296 = 513.2kN	
	From above = 384.87	
	Total = 908.47kN	N2 = 908.47kN
	FIFTH FLOOR - FOURTH FLOOR LEVEL	
	= 384.87 + (523.6x2)	
	= 1,432.07kN	
	FOURTH FLOOR - THRID FLOOR LEVEL	
	= 384,87+(523.6x3)	
	= 1,955.67kN	N4 = 1,955.67kN
	TEmrrDFLOOR-SECONDFLOOR	
	= 384,87+(523.6x4)	
	= 2,479.27	Ns = 2,479.27kN
	SECOND FLOOR LEVEL - FIRST FLOOR LEVEL	
	= 384,87+(523.6x5)	
	= 3,OO02.87kN	N6 = 3,OOO2.87kN
	FIRST FLOOR LEVEL - GROUND FLOOR LEVEL	
	= 384,87+(523.6*6)	
	= 3,526.47KN	Nr=3,526.47kN
BS 8110	GROUND FLOOR LEVEL -BASEMENT	
Part 1	= 384,87+(523.6x7)	
Section	= 4,050.07kN	Ns=4,050.07kN
3.3.13,		
3.8.1.6		
3.8.8.4		
1997	DESIGN USING SIMPLIFIED METHOD FOR AXIALLY LOADED COLUMN	
	Roof level - fifth floor	
	Ni = 384.87kN	
	The column is assumed braced	
	Check for slenderness	
	Le=Blo	
	Lex1 = O.75x 3000 < 15	
	3 = 5 < 15(ok)	

REF	CALCULATION	OUTPUT

leylb = 0.75x3000 < 15
230
= 9.85 < 15 (ok)

Therefore the column is short

N = 0.35 feu Ac + 0.67 Asc fy to take account of the area of connect displaced by the reinforcement the above equation may be written for rectangular section as

N = 0.35fcu Act (0.17fy - 0.35fcu)
Asc

ASe = n - 0.35 Feu Ac/cor67fy-0.35 feu

As = M-0.35 fcubh
0.7fy-0.35fcu

Fy = *410N/mm2* feu = *25N/mm2,* b = 230

H = 450 0.35fcu = 8.75, 0.7fy = 287

Bh = 103,500

ROOF TO SIXTH FLOOR

N= 384,87kN

= 384.87xl0₃ (8.75xl03.500)
278.25

As = -1,871.5mm2

Provide nominal reinforcement

Minimum reinforcement = O.4%bh
= 0.4% x 230x450
= 414mm2

Provide 4 Y16, area prov = 804mm2 — provide 4Y16

= y., x 16 =4mm

Maximum spacing = 12 x(size of the smallest se)
= 12x16 = 196mm

Therefore, provide RI0@200 — RI0@200

SIXTH FLOOR LEVEL - FIFTH FLOOR LEVEL

N=908.47kN

= 908.47x10₃(8.75xl03.500)
278.25
= 10225mm2

Provide 6 Y16 area prov = 1210mm2 — Prov6Y16

Minimum area links = y., x (size of the largest bar) = y., x 16
=6.25mm

Maximum spacing = 12 x (size of the smallest bar) = 12.x 25
=300mm

Therefore, provide RI0@200 — provide RI0@200

REF	CALCULATION	OUTPUT
	FDTHFLOORLEVEL-FOURTHFLOORLEVEL	
	N = 1,432.07kN	
	= 1.432.07xl0^3 - (8.75xl03.500)	
	278.25	
	= 1891.6mm^2	
	Provide 6 Y20, Area prov = 1890mm^2	Prov6Y20mm
	Minumum area oflink = ~ x (size of the largest bar) = ~ x 16	
	=4mm	
	Maximum spacing = 12x(size of the smallest bar) = 12x16	
	= 192mm	provide RI0@200
	Therefore, provide RI0@200	
	FOUTH FLOOR - THIRD FLOOR LEVEL	
	N = 1,955.67kN	
	= 1,955.67xl0^3 - (8.75 x 103,500)	
	278.25	
	= 2773mm^2	
	Provide 6Y25, Area prov = 295Omm^2	prov6Y25mm
	Minimum area of links = ~ x (size of the large bar)	
	=~x20=5mm	
	Maximum spacing = 12 (size of the smallest bar)	
	= 12x16 = 240	
	Therefore, provide RI0@200	provide Rl 0@200
	THIRD FLOOR - SECOND - FLOOR	
	N=2,479.27kN	
	= 2.479.27xl0^{3}7xl0^3 - (8.75 x 103,500)	
	278.25	
	= 3654mm^2	
	Provide 8 Y25, Area prov = 393Omm^2	prov8Y25mm
	Minimum area oflinks = ~ x (size of the large bar)	
	= ~x25 =6.25mm	
	Maximum spacing = 12 (size of the smallest bar)	
	= 12x25 = 300mm	
	Therefore, provide Rl 0@200	provide RI0@200

REF	CALCULATION	OUTPUT

SECOND FLOOR LEVEL - FIRST FLOOR LEVEL

N = 3,002.S7kN

= 3,002.S7xl03 - (S.75xI03.500)

278.25

= 3535mm2 — provide 8Y25mm

Provide S Y 25, Area prov 3930mm2

Minimum area of link V4 x (size of the largest bar)

= V4 x 25 = 6.25mm

Maximum spacing = 12 x (size of the smallest bar)

= 12x25 = 300mm

Therefore provide RIO@200

FIRST FLOOR LEVEL - GROUND FLOOR LEVEL

N = 3,526.47kN

= 3,526.47x103 - (8.75xI03.500)

278.25

= 4200mm2

GROUND FLOOR LEVEL - BASEMENT

N = 4,050.07KN

= 4,050.07*103 - (8.75*103500)

=4200mm2

Provide 10 Y 25, Area prov 4910mm2 — prov 10Y25mm

Minimum area oflink If.. x (size of the largest bar)

= If.. x 25 = 6.25mm

Maximum spacing = 12x(size of the smallest bar)

M x 25 = 300mm2

Therefore, provide RIO@200mm

,

REF	CALCULATION	OUTPUf
	4.8 DESIGN OF RAFT SLAB	
BS 8110	durability and fire resistance	
Table 3.2	condition of exposure = Mild	
Table 3.4	Required fire resistance = 1 hour	
Table 3.5	Nominal cover to meet fire resist = 75mm	cover=75fire resist ok.
	PANELl	
	4	
	ly=4	
	2 edges Ix=4	
	continuous Ly/bx= 1.0	2-way slab
	LOADING	
	Depth of raft slab = 500 = 500mm	
	Allowable bearing pressure = *150kN/m2*	
	Total load = 66,376.2kN	
	Factored load = *66,376.2/1.45* = 45,776.69kN	
	Req raft area Ar = factored axial load / Allowable bearing pressure	
	= 45,776.69 / 150 = 305.1Sm2	
	Actual floor area, Aa = 425.913m2	
	Used area of floor = 425.913m2	
	Total design pressure = Axial load / Used area	
	= 45,776.69 / 425.913	
	Design earth pressure = 107.4Sknm2 r= 107.48kN/m2	
Table 3.15	ULTIMATE MOMENT	
	use simplified load analysis	
	Msx = f3syn bx2	
	Msy = Bsy n l~	
BS S110	SHORT SPAN	
	At edges Mx (-ve) 0.047xl 07.48x42	SO.S2kNm
	At mid span (+vc) 0.036xl07.4Sx42	61.91kNm
	CONCRETE AND STEEL STRESSES	
	Characteristic strength of concrete. Feu = 25Nmm2	
	Characteristic strength of steel = 41 ONmm2	

REF	CALCULATION	OUTPUT
	REINFORCEMENT	
	Assume the use of minimum of 16mm diameter bars in direction of short span	
	d = 500-75 _16/2 = 417mm	
	k=M / bd2fcn = 80.S2xl06 / 1000x4172u5	
	k= 0.019 <0.156	
	Hence used 0.95d	
	Z=0.95x417	
	Z=396.15mm	z= 1178mm
	As req = M_ / 0.S7fyz = 80.82xl06 / 0.87x410x396	
	= 572mm2	As req=572mm2
		As prov=754mm2
	Moment msn (+re) = 61.91kN/m	
	Size of reinforcement = 16mm	
	Effective depth of slab = 417mm	
	Used lever arm factor = 0.95	
	At mid span	
	k=M / bd2fcn = 61.9lxl06 / 1000x4172x25 = 0.014	
	k= 0.014	k=0.014
	Z=0.95x417 = 396.15mm	
	As req = M_ / 0.S7fyz = 61.91xl06 / 0.S7x41 Ox396	
	= 438.3mm2	
	Provide Y *12@200c/o*	As req=438.3mm
	As prov = 666mm Top	As prov=Seemm'
	LONG-SPAN	
BS S110	At edges my-ve = 0.045xl07.48x42	*77.39kN/m*
	At mid span mys+ve = 0.034xl07.48x42	*58.47kN/m*
	Effective depth = 417mm	

REF	CALCULATION	OUTPUT
	k=M = 77.39xl06 = 0.017	
	$bd_{2}fcn$ 103x4172x25	
	k=0.017<0.156	k=0.017
	la= 0.95	
	Z=0.95x417 = 396.15mm	
	As req = 77.39xl06 = 547.9mm2	
	0.87x410x396	*As* req=547.9mm2
	Provide Y 12@150c/c	
	As prov = 754mm2/m	*As* prov=754mm2
	= 58.47xl06 =0.013	
	103x4172x25	
	k = 0.03 < 0.156	
	la = 0.95	k= 0.013
	Z = 0.95 x 417 = 396mm	
	As req = 58.47xl06 = 414mm2	
	0.87x410x396	
	Provide Y 12mm~50%	*As* req=tl-tmm'
	As prov = 452mm 1mTop	*As* prov=452mm2
BS 8110	Durability and fire resistance	
Table 3.2	Condition of exposure = mild	
TabJe3.4	Required fire resistance = 1 hour	
Table 3.4	Norma] cover to meet durability = 50mm	cover = 75 ok
Table 3.5	Normal cover to meet fire resist = 75mm	
	ly=4	
	Ix=4	
	ly/bc = 10	2 way slab
	LOADING	
	Depth of raft slab = 500mm	
	As low able bearing presture, qu = 150kNm2	
	Total load = 66,376.2kN	
	Factored load = 66,376.2/1.45 = 45,776.69kN	
	Required raft area, Dr = factored Axiaload	
	Allowable bearing pressure	

REF	CALCULATION	OUfPUT
	= 45,776.69 = 305.18m^2	
	150	
	Actual floor area Da = 425.913m^2	
	Used area offloor = 425.913m^2	
	Total design pressure = Axial load	
	Used area	
	=45,776.69	
	425.913	
	Design earth pressure = *107.48kN/m_2*	n = *107.48kN/m_2*
Table	ULTIMATE MOMENTS	
3.15	use simplified load analysis	
	Msx = psx n l~	
	Msy = Bsy n Ix2	
	SHORT SPAN	
BS8110	At edges Mx(-ve) = 0.032xl07.48x42	*55.03kN/m*
Equation 14	at mid span (+ve) = 0.0240 x 107.48x42	*41.27kN/m*
Equation 14		
	REINFORCEMENT	
	Moment, Msx (-ve) = *55-03kN/m*	
	Size of reinforcement = 12mm9	
	Used lever arm factor = 0.95	
	Effective depth of slab = 417mm	
	k=M = 55x$l0_6$ = 0.013 < 0.156	
	bd2fcn 10_3x4192,a5	
	la= 0.95	
	Z=0.95 x417 =396mm	
	Dsreq=MSx	
	0.87fyz	
	As req = 55x$l0_6$ = 389.4mm_2	
	0.87x410x396	
	Provide Y 12mm~50c/cB + M	As req= 389.4mm_2
	As prov = 452mm *1m*	As prov=452mm_2

REF	CALCULATION			OUTPUT
	Moment, Msx (+ve) = *41.27kN/m*			
	Size of reinforcement = 12.mme			
	Effective depth of slab = 419mm			
	Used lever arm factor = 0.95			
	At mid span			
	k=M / bd'fcn	= 41.27x106 / 103x419~5	=0.09	k=O.09
		k = 0.09 < 0.156		
	As req = 41.27x106 / O.87x41 Ox396		= 292.2mm2	As req= 292mm2
	Provide Y 12mm~0(nc Top			As prov= 377mm2
	As prov = 377mm *1m*			
	LONG SPAN			
	At edge mu - ve = 0.032x107.48x42			55.03knm
	At mid span mus + ve = 0.024xI07.48x42			*42.96kn/m*
	REINFORCEMENT			
	Moment, Msy (-ve) = 55-03kn/m			
,-	Size of reinforcement = 12mm			
	Used lever arm factor = 0.95			
	Effective depth of slab = 417mm			
	k=M / bd2fcn	= 55.03x106 / 25x1000x4192	= 0.013 < 0.156	
	k = 0.013 < 0.156			
	Z=396mm			
	As req = M / 0.87fyz	= 55x106 / 0.87x410x396	= 389.4mm2	As req= 389.4mm2
	Provide Y 12@250c/c B + M			
	As prov = *452mm2/m*			As prov = 452mm2
	Moment, Msy (-ve) = 41-27kn1m			
	Size of reinforcement = 12mm			
	Effective depth of slab = 417mm			
',	Used lever arm factor = 0.95			
	At mid span			

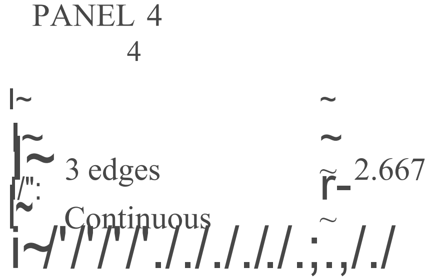
REF	CALCULATION	OUTPUT
	$k = \frac{M}{bd^2 f_{cn}} = \frac{41.27x10^6}{l000x4192x25} = 0.09$	k=O.09
	k = 0.09 < 0.156	
	As req $= \frac{41.27xlO^6}{0.87x410x396} = 292.24mm^2$	As req= 282.2mm2 As prov = 377mm2
	Provide Y 12mm~0(nc Top As prov = 377mm 1m	
BS 8110	Durability and fire resistance	
Table 3.2	condition of exposure = mild	
Table 3.4	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 50mm	cove = 75
Table 3.5	Nominal cover to meet fire resist = 75mm	fire resist ok
	ly=5.333 4 bi=4 Ly/lx= 1.3	2 way slab
,		
	LOADING Depth of raft slab = 500mm Allowable bearing pressive qu = $150kN/m^2$ Totalload = 66,376.2kn Factored load 66,376 2/1.45 = 45,776.69kN Required raft area AT = $\frac{\text{factored Axiallxad}}{\text{Allowable bearing pressure}}$	
	$= \frac{45,776.69}{150}$ 305.ISm2	
	Actual floor area Da = 425.913mm2 Used area of floor = 425.913mm2	
	Total design pressure = $\frac{\text{Axial load}}{\text{Used area}}$ $= \frac{45,776.69}{425.913}$ $n = 107.48kN/m^2$	n=107.48kN/m2

REF	CALCULATION	OUTPUT
	ULTIMATE MOMENT	
	Use simplified load analysis	
	Msx = J3sxn l~	
	Msy = J3sxn Ix^2	
BS 8100	SHOR SPAN	
Equation 14	At edges mx (-ve) = 0.046 x I07.48x4^2	*79.11kN/m*
Equation 14	At mid span (+ve) = 0.0350 x I07.48x4^2	*60.19kN/m*
	REINFORCEMENT	
	Moment, Msy (-ve) = 79.11 *kN/m*	
	Size of reinforcement = 16mm	
	Used lever arm factor = 0.95	
	Effective depth of slab = 417mm	
	k=M / bd'fcn = 79~llxI06 / 25xl000x417^2 =0.018	
	k=0.OI8<0.I56	k=0.018
	Z=3%mm	
	As req = M / 0.87fyz = 79.1 IxI0^6 / 0.87x4I0x3%	As req= 56Omm^2
	Provide Y 16mm~300c/cB + M	As prov = 670mm^2
	As prov = 670mm *1m*	
	Moment, Msy (+ve)	
	Size of reinforcement = 12mm	
	Used lever arm factor = 0.95	
	Effective depth of slab = 417mm	
	k=M / bd'fcn = 6O.19xl 0^6 / 25xIOOOx417^2 = 0.014	
	k = 0.014 < 0.1"56	k = 0.014
	Z=396mm	
	As req = M / 0.87fyz = 6O.19xI 0^6 / 0.87x4I0x3%	As req= 426mm^2
	Provide Yl2mm@250c/c Top	As prov = 452mm^2
	As prov = *452mm^2/m*	

REF	CALCULATION	OUTPUT
	LONG SPAN	
	At edges Mx (-ve) = 0.032Oxl 07.48x42	55.03kNm
	At mid span (+ve) = 0.024 x 107.48 x 42	41.27kNm
	REINFORCEMENT	
	Moment, Msy (-ve) = *55.03kN/m*	
	Size of reinforcement = 12mm	
	Used lever arm factor = 0.95	
	Effective depth of slab = 417mm	
	k=M / bd'fcn = 55.03x106 / 25xlOOOx4192 = 0.013 < 0.156	k=O.013
	Z=3%mm	
	Asreq =M_ / 0.87fyz = 55.03xl06 / 0.87x410x3% = 390mm2	As req= 39Omm2
	= *39Omm2/m*	As prov = 452mm2
	Provide Y 12mm~50c/c	
	As prov = 452mm 1m B + M	
	Moment, Msy (+ve) = 41.27	
	Size of reinforcement = 12mm	
	Used lever arm factor = 0.95	
	Effective depth of slab = 419mm	
	k=M / bdfcn = 41.27x106 / 25xlO00x4192 =0.09	
	k = 0.09 < 0.156	k=O.09
	Z = 0.95x419 = 398mm	
	As req =M_ / 0.87fyz = 41.27x106 / 0.87x410x398 = 291mm2	
	Provide Y 12mm~OO%	
	As prov = 377mm Top2	
BS 8110	Durability and fire resistance	
Table 3.2	condition of exposure = mild	
Table 3.4	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 50mm	cove = 75
Table 3.5	Nominal cover to meet fire resist = 75mm	fire resist ok

REF | CALCULATION | OUTPUT

PANEL 4



ly=4
Ix=2.667
Lyllx = 1.5 — 2 way slab

LOADING

Depth of raft slab = 500mm

Allowable bearing pressive qu = *150kN/m*^{2}

Totalload = 66,376.2kN

Factored load 66,376 *2/1.45* = 45,776.69kN

Required raft area AT= factored Axial load / Allowable bearing pressure

= 45,776.69 / 150 = 305.1Sm^2

Actual floor area Da = 425.913mm^2

Used area of floor = 425.913mm^2

r

Total design pressure = Axial load / Used area

=45,776.69 / 425.913

n = *107.48kN/m*^{2} — OUTPUT: n = *107.48kN/m*^{2}

ULTIMATE MOMENT

Use simplified load analysis

MSx = psx n Ix^2

Msy = psy n Ix^2

BS8110 — SHORT SPAN

Equation 14 — At edges mx (-ve) = 0.073 X 107.48x2.667^2 — 55.77kNm

Equation 14 — At mid span (+ve) = 0.055Oxl07.48x2.667^2 — 42.05kNm

REINFORCEMENT

Moment, Msx (-ve) = *55.77kN/m*

Size of reinforcement = 12mm

Used lever arm factor = 0.95

Effective depth of slab = 419mm

REF	CALCULATION	OUTPUT
	$k = \frac{M}{bd^2 f_{cn}} = \frac{55.77x10^6}{25x1OOOx419} = 0.013 < 0.156$	k 0.013
	Z = 0.95 x 419 = 398mm	
	$As_{req} = \frac{M}{0.87 f_y z} = \frac{55x10^6}{0.87x410x396} = 393.4mm^2$	As req= $393mm^2$
	Provide Y 12rnm~50c/c B + M As prov = 452rnm 1m	
	Moment, Msx (+ve) = 42.05kNm Size of reinforcement = 12mm Used lever arm factor = 0.95 Effective depth of slab = 419mm Z = 0.95 x 419 = 389mm	
	$k = \frac{M}{bd^2 f_{cn}} = \frac{42.05x10^6}{25x1000x419^2}$ $= 0.09 < 0.156$	kO.09
)or	$As_{req} = \frac{M}{0.87 f_y z} = \frac{42.05x10^6}{0.87x410x389} = 303rnm^2$ Provide YI2mm@300mm% As prov = *377mm²/m*	
	LONG SPAN At edges Mx (-ve) = $0.037x107.48x2.667^2$ At mid span (+ve) = $0.028 x 107.48 x 2.667^2$	28.29kNm 21.41kNm
	REINFORCEMENT Moment, Msy (-ve) = *28.29kN/m* Size of reinforcement = 12mm Used lever arm factor = 0.95 Effective depth of slab = 419mm Z = 0.95x419 = 389rnm	
	$k = \frac{M}{bd^2 f_{cn}} = \frac{28.29x10^6}{25xIOOOx419^2}$ k=O.06	k=O.06
	$As_{req} = \frac{M}{0.87 f_y z} = \frac{28.29x10^6}{0.87x41 Ox3389} = 204rnm^2$ Provide Y 12mm~300% As prov = 377mm *1m*	As req = $204mm^2$ As prov = $377mm^2$

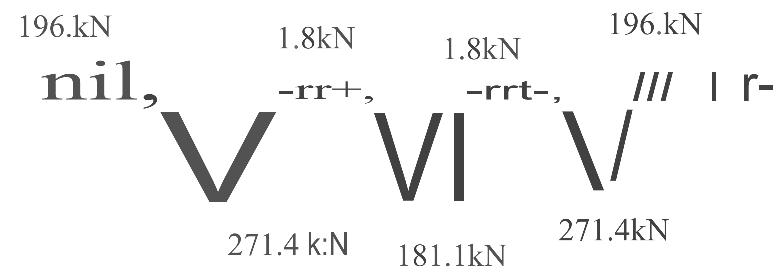
REF	CALCULATION	OUTPUT
	Moment, Msx (+ve) = 21.41kNm	
	Size of reinforcement = 12mm	
	Used lever arm factor = 0.95	
	Effective depth of slab = 419mm	
	Z = 0.95 x 419 = 389mm	
	k=M / bd'fcn = 21.41x10^6 / 0.87x410x389 =0.04	As req = 154.3mm^2
	As req = M / 0.87fyz = 21.41xl0^6 / 0.87x41 Ox3389 = 154.3mm^2	
	Provide y 12mm~OOc/c	
	As prov = 377mm 1m	As req = 377mm^2
	4.9 DESIGN OF GROUND BEAM	
	Beam designed as continuous over supports and capable of free rotation about them latent wind loads taken by columns and end shear walls,	
BS 8110	Durability and fire resistance	
Table 3.2	condition of exposure = moderate	
Table 3.4	Required fire resistance = 1 hour	
Table 3.4	Nominal cover to meet durability = 20mm	
Table 3.5	Nominal cover to meet fire resist = 20mm	
	BMI (Grid *B/l-5)*	
	Loading	
	(1) Beam self weight = 0.45xlx24x1.4 = *15.12kN/m*	
	(2) From slab = *'t,* x 107.48x4 = *143.3kN/m*	
	158.42kN/m	
	Span 2 -3 = span 3 - 4 span = span 4 - 5 = *158.41kN/m*	

REF	CALCULATION	OUTPUT

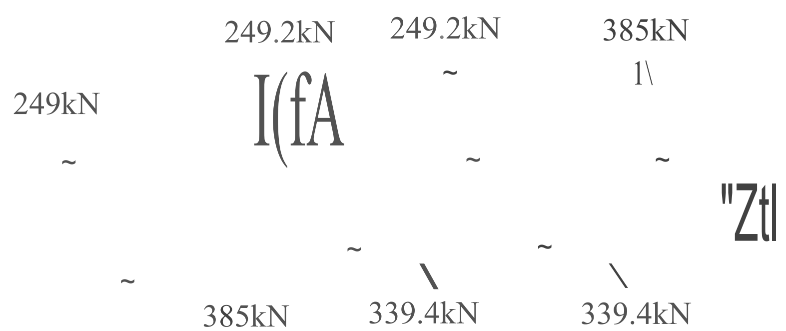
/1514IN

	(1) 4m		(2) 4m		(3)4m	(4)	(5) 4m	
K	';.4x₃,1..-0.188		';.4=0.25		';.4=0.25		';.4x%=O.l88	
DF	0	0.43	0.57	0.5	0.5	0.5	0.43	0
FEM	-211.2	211.2	-211.2	211.2	-211.2	211.2	-211.2	211.2
DM	211.2	---	----	--	-----	---	----	
COM		105.6	----	--	-----	--	105.6	
DM		45.4	60.20	---	-----	60.20	45.4	
COM		--	----	30.10	30.10	--	------	
DM		---	-----	---	-----	--	----	
EM		271.4	-271.4	181.1	-181.1	271.4	-271.4	
Elastic shear	316.8	316.8	316.8	316.8	316.8	316.8	316.8	316.8
Static shear	-67.85	67.85	22.6	22.6	-22.6	226	67.85	-67.85
Total	249	385	294.2	339.4	294.2	339.4	385	249
Reaction		63	9.2	63	3.6	72	4.4	
SM	196		1.8		1.8		196.0	

Bending Moment Diagram



Shear Force Diagram



REF	CALCULATION	OUTPUT

Design

Span 1-4 M= 196kNm

B = 450mm, h = 1500mm

d= 1500-50-10-20-10= 1,410mm

Assume 2 layer of20mm diameter with 10mm diameter stirrup
10mm diameter stirrup as links

k=M / bd'fcn = 196xl0^{6} / 25x45Oxl,410^{2} k=0.08

k=0.08<0.156

Z = 0.95x1,410 = 1340mm — **Output:** kO.08

Asreq =M_ / 0.87fyz = 196xl0^{6} / 0.87x41Ox3340

= *410mm^{2}/m*

Minimum steel = 0.15%bd

= 0.15xI0-2x 450xl,410 — **Output:** As prov=1260mm2

= 951.2mm2

Provide 4 - Y 120mm bars Top

As prov = *1269mm^{2}/m*

Support 2

M=271.4

k=M / bd'fcn = 271.4x10^{6} / 25x450xl,1410^{2} =0.012 — **Output:** k 0.012

Asreq =M_ / 0.87fyz = 271.4xl0^{6} / 0.87x410x1340

= *568mm^{2}/m*

Minimum steel = 0.15%bd

= O.l5xlO-2x 450xl,410

= 951.2mm2

Provide 4 - Y20mm Bar Top — **Output:** As prov=1260mm2

As prov = 1260mm2

CHECK FOR SHEAR

V max = 385kN

Shear stress v = *vlbd*

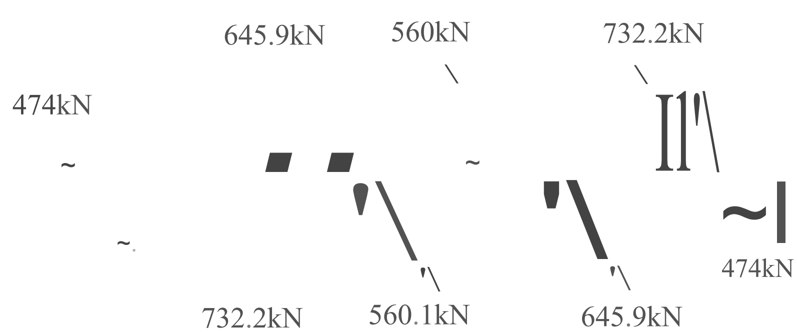
v = 385x10^{3} / 450x1,340 = 0.68 *N/mm2*

REF | CALCULATION | OUTPUT

	(1) 4m		(2) 4m		(3)4m	(4)	(5) 4m	
K	Y4k%-0.188		Y4-0.25		Y40.25		Y4~I4=O.l88	
DF	0	0.43	0.57	0.5	0.5	0.57	0.43	0
FEM	-402	402	402	402	402	402	402	402
DM	402	---	-----	-----	--.--	-----	-----	402
COM		201		---	---.-		201	
DM		86.43	114.6			114.6	86.43	
COM		---	-----	57.29	57.29	---	----	
DM		---	-----	---	——	---	----	
EM	0	516.6	-516.6	345	-345	516.6	-516.6	0
Elastic shear	603	603	603	603	603	603	603	603
Static shear	-129.2	129.2	42.9	-42.9	-42.9	42.9	129.2	-1292
Total	474	732	645.9	560.9	560.1	645.9	732.2	474
Reaction		1.3	78.1	11	20.2	1,37	8.1	
SM	372		175		175		372	

Bending Moment Diagram

Shear Force Diagram



REF	CALCULATION	OUTPUT
	Span 1 - 4 M = *372kN/m*	
	k=M = 372xl0[6] =0.016	
	bd'fcn 25x450xl,4102	
	= 0.016 < 0.156	
		k 0.016
	As req =.M_ = 372xl0[6]	
	0.87fyz 0.87x41 Oxl,340	
	= *778.3mm2/m*	
	Minimum steel = 0.15%bd	
	= 0.15xlO-2 x 450xl,410	
	= 951.2mm2	
	Provide 4 - Y20mm Bars Top	
	As prov = 1260mm2	
	Support 2	
	M=516.6knm	
	k=M = 516.6xl0[6] =0.023	
	bd2fcn 25x450xl,14102	
	k = 0.023 < 0.156	k 0.023
	Z = 0.95xl,410 = 1,340mm	
	As req =.M_ = 516.6x10[6]	
	0.87fyz 0.87x41Oxl,340	
	= 1,081mm2/m	
	Provide 9 - Y20mm bars B+M	
	As prov = 1260mm2	
	CHECK FOR SHEAR	
	V max:=732.2kN	
	Shear stress v = *v/bd*	
	v = 732.2xl03 = *1.2N/mm2*	
	450xl,340	
	100AS = 100xl081 = *0.17N/mm2*	
	bd 450xl,340	
	VC = *0.37N/mm2*	
	v=vc	

REF	CALCULATION	OUTPUT

100AS = 100x1260 = *0.2N/mm2*
bd 450xl,340

VC = *0.36N/mm2*
ASV = b ~ = 250 (0.68 - 0.36)
Sv 0.87fyv 0.87x250
=0.37

YI0@300(As prov 0.523)0

Beam 2 (Grid *B/I-5)*

LOADING SPAN 1-2
Beam self weight = 15.12kn1m
From slab 't« n be = lh x l07.48x4 = 143.3x2=286.6kn/m
Total load = 301.72kn1m
SPAN 2-3, SPAN 3-4, SPAN 4 - 5 = SPAN 1 - 2

r:

REF	CALCULATION	OUTPUT

ASV = b !Y...:.9 = 250 (l.2 - 0.37)
Sv 0.87fyv 0.87x250
=0.95

YI2@225(As prov 1.004)

BEAN 20

SpanA-B = 5panB-C

(1) Beam self weight = 0.45 xl x 24 x l.4 = 15.12

(2) From slab raft slab = 113n be = 113x 107.48 x 4 = *143.3kNl*

Total load 158.42 m

Span n' - El = C - D

(1) Beam self weight = *15.12kN/m*

(2) From raft slab = Y2n be (1 - *1/3* k2) = Y2x 107.48
(1 - *1/3* (5.3)2) x 2 = 198.2x2
= *348kN/m*

SpanE1-H

(1) Beam self weight = *15.12kN/m*

(2) From raft slab = Y2n be (1 - Ih k2)
= Y2x 107.48 (1 - Ih *6/4)*

= *396.4kN/m*

'1

185A2KNIM ~ 398.4KNIM

	A	4	B	4	C	5.334	0	5.334	El 6	H
K	'ih:y.. = 0.188		\4 =0.25		'ls.3 = 0.18		'/5.3=0.18		l6 =0.1167 x 3;'	
DF	0	0.43	0.57	0.58	0.42	0.5	0.5	0.52	0.48	0
FEM	-211	211	-211	211	8150	8150	~15	8150	1,189	1,189
DM	211	---	...-	350	254	---	---.	194.5	180	-1,189
COM	--_...	105.5	175	-.--	..--	127	97.3	._.....	595	
DM		30	40	......	-----	14.9	14.9	309.2	286	
COM			----	20	7.45	--.-	155	7.45	......---	
DM		-..--	--.-	7.28	5.3	-77	-77	3.9	3.6	
COM		-....-	3.64	----	39	2.7	1.95	3.9	-----	
DM		1.6	2.1	2.3	1.6	·0.38	0.38	20.3	18.7	
COM		---	US	1.05	0.7	0.8	10.15	0.7	.-.-	
DM		0.5	0.7	0.203	0.15	4.7	4.7	0.36	0.34	
EM		348.7	·346	576.9	-576	854	-854	1296	·1296	
Elastic shear	317	317	317	317	922.2	922.2	922.2	922.2	1,189.2	1,189.2
Static shew	·87.2	87.2	57.5	57.5	52.6	52.6	52.6	21.6	21.6	-21.6
Total shear	229.5	404.2	259.5	375	869.6	974.8	869.6	974.8	973.2	973.2
Reaction		663.	7		1,244.6		1,844	19	48	
SM		6S		133.1		1,464		1,464	1,195	

e.-

'f'

BENDING MOMENT DIAGRAM

165kN 133.lkN 1464kN 146.6kN 1.195kN

348kN 576.9kN 854kN 1.296kN

SHEAR FORCE DIAGRAM

259.5kN 869.6kN 869.6kN 973.6kN

404.2kN 375kN n4.8kN 974.6kN

REF	CALCULATION	OUTPUT

Design

SPAN A-C= 1,464KNIM

d = 1,41Omm

Assume 2 layer of 20mm diameter with 10mm diameter Stirrup as links:

k=M / bd'fcn = 1,464x10^6 / 25x450x1,410^2

K= 0.06 < 0.156 — kO.06

.:. la= 0.95

Asreq =M__ / 0.87fyz = 1,464x10^6 / 0.87x41Ox1340

=3,063mm^2 — As req=3,063mm^2

Provi e 10 Y20mm bars (3140mm^2)

As prov=3140mm^2/m

SPAN C - H AND SUPPORT B M = 1,195KNIM

k=M / bd^2fcn = 1.195x10^6 / 25x45Ox1,4102

kO.05

.:. la = 0.95

Asreq =M__ / 0.87fyz = 1,195x10^6 / O87x41Ox1340

=2,500mm^2 — As req=2,500mm^2

Provide 8 Y20mm bar (2,51Omm') — As p v=2510mm^2

SUPPORT El M = 1296KNIM

k= 1296x10^6 / 25x450x1 ,410^2

1(= 0.057 < 0.156 — k 0.057

.:. la= 0.95

Asreq =M__ / 0.87fyz = 1296x10^6 / 0.87x41Ox1,340

=2,711mm^2

Provide 9 Y20mm bar (2,830mm^2)

REF	CALCULATION	OUTPUT

CHECKSBEAR

Shear stress v = v/bd

V=974.8

V = 974.8xl0₃ / 450xl,340 = *l.6N/mm2*

100AS / bd = 100x2830 / 450x1340 = *0.47N/mm2*

VC=0.58

ASV / SV = b(v-vc) / 0.87fyv

250(1.6-0.58) / 0.87x250

= l.17

YI0 @ 125 (As prov l.256)

BEAM 7

SPAN5 - 6 = 6.7 = 8-9

Beam self weight = 15.12

From raft slab = % n lx (l_1hk2)

= Y2 x 107.48 x 2.66 (1-1IJC'hooi)

= *13l.5KN/m*

SPAN9-10

Beam self weight = 15.12

From raft = *11h*x 107.48x4 *=143.3KNIM*

= 158.0KNIM

REF CALCULATION OUTPU

BEAM 9

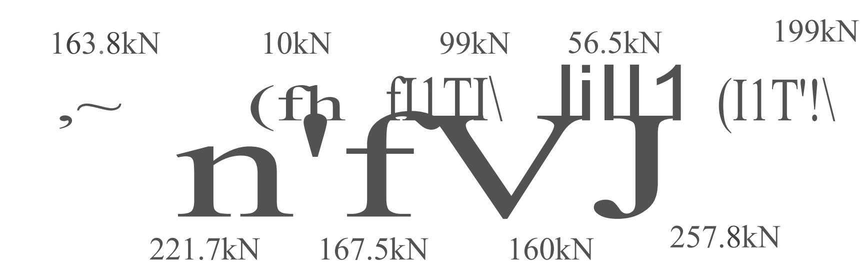
/131.5KNIM .: 15.8KNIM

~r--..~~~r--..~ ~~~

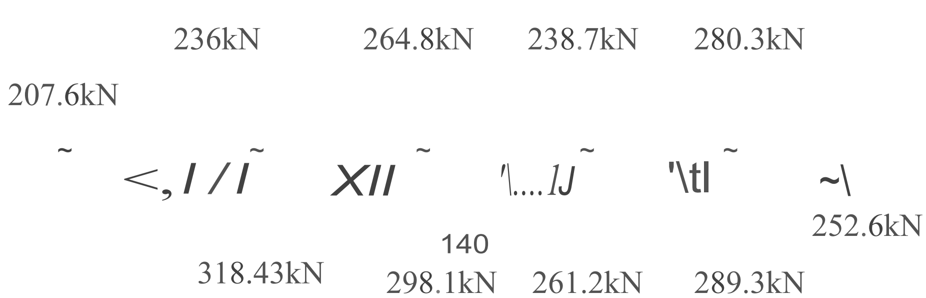
(5) 4m (6) 4m (I) 4m (8) (1O)4m

K	Y.x~-0.188		Y. -0.25		y. -0.25		Y.-0.25		Y.x~=O.l88	
OF	0	0.43	0.57	0.5	0.5	0.5	0.5	0.57	0.43	0
FEM	175	175	-175	175	-175	175	175	175	211	211
OM	175	--	---	--	---	--	---	20.5	15.48	211
COM		87.5	---	--	---	--	10.25	-"-	105.5	
OM		37.6	49.9	--	---	5.13	5.1360.14		45.4	
COM		----	--	25	2.57	--	3007	2.57	----	
OM		---	----	13.79	13.79	15.0	15.0	1.46	1.11	
COM		---	5.61	--	7.7	6.9	0.73	7.7	---	
OM		2.4	3.2	3.85	3.85	3.09	3.09	4.39	3.31	
COM		---	1.92	1.6	1.22	1.93	2.20	1.22	---	
OM		0.82	1.09	141	1.41	0.14	0.14	070	0.52	
EM		221.7	-221.7	1675	-167.4	160.11	-160	257.3	-257.3	
Elastic shear	263	263	263	263	263	263	263	263	316.8	316.8
Static shear	-55.43	65.43	27.1	27.1	1.82	-1.82	24.3	24.3	64.33	-64.33
Total	207.6	318.43	236	290.1	264.8	287.3	238.7287.3		380.3	252.6
Reaction		318	.2	555		499	.9	66	7.6	
SM		163.8		10		99		56.5		199

BENDING MOMENT DIAGRAM



SHEAR FORCE DIAGRAM



REF	CALCULATION	OUTPUT

Design

SPAN 55 - 9; M = 163.8

d= 1,410mm

Assume 2 layer of 20mm diameter with 10mm diameter Stirrup as links.

$k = \frac{M}{bd^2 f_{cu}} = \frac{163.8 \times 10^6}{25 \times 450 \times 1{,}410^2}$

K= 0.07 < 0.156

∴ la = 0.95 Z = 1,340

$As_{req} = \frac{M}{0.87 f_y z} = \frac{163.8 \times 10^6}{0.87 \times 410 \times 1340}$

= 343mm2

Minimum steel = 0.15% bel = 0.25xl02 x450x 1,410 = 95l.75

Provide 5 Y16mm bars - Top (1010mm2)

SP AN 9 - 10 AND SUPPORT 7 M = *199kN/m*

$k = \frac{M}{bd^2 f_{cu}} = \frac{199 \times 10^6}{25 \times 450 \times 1{,}410^2}$

K= 0.08 < 0.156

Support 9

M=257

$k = \frac{M}{bd^2 f_{cu}} = \frac{257 \times 10^6}{25 \times 450 \times 1{,}410^2}$

K= 0.01 < 0.156

Ja = 0.95 Z = 1,340

$As_{req} = \frac{M}{0.87 f_y z} = \frac{257 \times 10^6}{0.87 \times 410 \times 1340}$

,/

Minimum steel = 0.15% bel = 0.15 x 10_2x450 x 1,410 = 95l.75

Provide 5 Y 16mm bars (1010mm2)

REF	CALCULATION	OUTPUT

$$\frac{A_{SV}}{S_V} = \frac{b(v-v_c)}{0.87f_{yv}}$$

$$= \frac{250(0.52-0.15)}{0.87 \times 250}$$

$$=0.43$$

Y 10 @ 225 (As P

CHAPTER FIVE

DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

5.1 Discussion of Results

5.1.1 Design of Roof Slab

In. the roof slab design, panel 1 was designed as 2 way slab, out of which the total dead load was *6.6kN/m2,* live load was *1.5kN/m2,* the designed load on slab was *11.6kN/m2,* and checks including deflection, shear force were carried out in accordance with the appropriate Code of Practice to ensure the safety of the structure.

5.1.2 Design of Roof Gutter

The ultimate designed load for roof gutter was 14.08 *kN/m.*

5.1.3 Ribbed Floor

The ribbed floor design typical floor panel 1 was designed as two way slab but with lesser value the dead load was *5.39kN/m2* while the live load was 2.304kN and the ultimate design load was also *7.71kN/m2.* The ribbed floor was lighter than a solid floor and has advantage of thermal installation, thus the imposed load on the foundation or raft slab is considerably reduced.

5.1.4 Beam

A beam like any other member is designed to resist the ultimate bending moment, shear force and torsion moments if any. Also, serviceability requirement must be adequately considered to ensure the members behave satisfactorily under the working load. However, much emphasis is generally placed on the ultimate limit state requirement of the beams. The ultimate designed load for roof beam 1 was 30.1 *kN/m,* for floor beam 1 was *32.065kN/m* and the designed load for ground beam 1 was also 158.42kN/m

5.1.5 Column Design

Primarily, columns were designed as compression members. Although, some of the columns may be subjected to bending either due to their tenderness or to asymmetrical loading from beams. In a structure, they carried the load from the beam and the slab to the foundation.

From the roofto the sixth floor, the total load on the column was 384.87kN and the total load from the sixth floor to the fifth floor was 908.47kN. From the fifth to fourth floor, the total load was 1,432.07kN and from the fourth to the third floor 1,955.67kN. From third to second floor, the total load was 2,479.27kN and from the second to first floor the total load was 3,002.S7kN. Finally, from the first to ground floor the total load was 3,526.47kN and from the ground floor to the basement the total load was 4,050.07kN.

5.1.6 Design of Foundation

A raft foundation consists of a continuous concrete slab under the whole building, taking all downward loads and distributing them over an area large enough to avoid over stressing the soil beyond its bearing capacity. By the virtue of spreading the load over a very wide area (not less than the area of the building), the raft slab was 500mm, allowable bearing pressure was 150kN/m2,total load was 66,376.2kN and the factored load was 45,776.69kN. Therefore, the required raft area was 301.1Sm2 and the actual floor area was 425.913mm2. Finally, the total designed pressure was l07.4SkN/m2.

5.2 CONCLUSION

Engineering structures consist of materials assembles together in some specified manner for the purpose of carrying external loads. And structural design is carried out to determine the sizes of the component parts so that the resulting structural can support the loads to which they will be subjected. The probabilistic approach for both structural properties and loading conditions has led to the limit state design philosophy which is now universally accepted and adopted in this project design in order to provide the structure a sufficient strength and rigidity which in tum will in turn result to a functional, economical and most importantly safe structure to inhabit or use. Also the design has been in compliance will various accepted standards and relevant codes of practice.

However, if this proposed 6 story office is translated into physical structure, it will stand the test of time, yield an effort to meet the changing standards as they rise all the time, give the customer room for transactions value and also satisfy the needs, priorities, lifestyle, culture and taste of the various people, races and be a source of revenue. Hence, the project work is successful.

5.2 RECOMMENDATION

Although this project work effectively deals with the analysis and design of the super structure, it includes substructure and detailed analysis of the structure. I therefore recommend that this area should be greatly improved upon by students willing to do any project work on design of structure of any kind.

Also in addition the following points are recommended:

1. There should be provision for shear wall in order to cater for the wind effect
2. Ribbed floor is recommended to be used for long slab especially of 6.0m by 7.0m to avoid crossing at the decking level.

3. Ribbed floor is also recommended because of its thermal insulation that is greatly superior to other forms of construction and makes it ideal for Nigeria climate.
4. Relevant textbooks and website should be made available to the students as there are difficulties in sourcing these materials.
5. There should be more students-lecture practical interaction especially for the final year students.
6. More practical exposure to the vanous specializations m the profession should be encouraged.

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