STRUCTURAL DESIGN OF PROPOSED UNITS OF 3BEDROOM MAISONETTE LOKOGOMA FCT ABUJA

BY

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PGD/CIVIL *ENG/07/001*

DEPARTMENT OF CIVIL ENGINEERING

SCHOOL OF ENGINEERING AND ENGINEERING TECHNOLOGY

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DECLARATION

I, Samson Olugbenga victor declare that this project work "Structural Design of two storey residential building { 4 unit of 3 bedroom maisonettes}" was solely carried out by me. All authors from whom vital information were gotten have been duly acknowledged.

10TH MAY 2010

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Samson Olugbenga Victor

Date

PGD/Civii *ENG/07/001*

CERTIFICATION

This is to certify that this project is the original work done by Samson Olugbenga Victor under supervision and accepted by the Civil Engineering Department Federal University of technology Minna.

Engr Oritola S

Date

{Supervisor}

Engr ProfSadiku's

Date

{Head of Department}

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External supervisor

bate

DEDICATION

I d dicated this project to the King of Kings and Lord of Lords, the};tmighty God, wh has seen me through my course of study. Furthermore, to my parents and fridnds who because of their love for education for me, put in all they can to get me educated.

ACKNOWLEDGEMENT

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I ish to express my gratitude and deep appreciation to my supervisor Engr Oritola for the wonderful, careful supervision, invaluable assistance and encouragement recFived in making this project work a reality.

I am particularly indebted to my project supervisor, Engr Oritola S.F who, despite his tight schedule, took pain to go through the manuscript and offered useful suggestions at all stages of this project report

My special thanks goes to the head of department Engr Prof S. Sadiku, Dr Sudo [PGD Coordinator] all lecturers and members Mr P.N Ndoke, Engr Alhassan, Engr Kudu, Mr Saidu, Engr James,Engr [Dr] 0 D Jimoh, Engr Abudullar, Mallam Sule A, their humble guidance, encouragement and assistance towards sharpening the pen of my mind and making this project possible.

I deem it imperative to express my heart felt appreciation to my parent's Mr & Mrs Samson Agunbiade, my lovely wife' Olanike Samson, my precious daughter Oluwaferanmi Samson, lastly my brother and sister Ariyo Samson, Kayode Samson, Mary Samson, for their moral assistance which enable me to carry out the project work easily.

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Finally, to the great I am that I am for his continually sustenance and guidance.

ABSTRACT

This project covers the analysis, design and detailing of proposed 4 units of 3 bedroom maisonette at Logokoma satellite town of Abuja, the project was prepared based on the standard and principle set out by the structural use of concrete B58110 parts 1, 2 and 3 to achieve the desired objectives. The roof members, Beams slabs, stair - case, column and the foundations were analyzed and designed in accordance to B58110. The results were used to produce simple and neat structural detailed drawing to ease estimation and construction of the proposed project.

TABLE OF CONTENT

CHAPTER THREE

CHAPTER FOUR

CHAPTER FIVE

LIST OF NOTATION

As'	Cross sectional of compress reinforcement
As	Cross sectional area of tension reinforcement
Asb	Cross sectional shear reinforcement in the form of bend up
Asc	Cross sectional area of reinforcement in compression
Asv	Cross sectional area of shear reinforcement in the form of links
b	Width of section
bw	Breath of web
d	Effective depth of section
B	modulus of elasticity of concrete
feu	characteristic of concrete cube strength
fs	Service stress of steel
fy	characteristic strength of steel reinforcement
fyv	characteristic strength of link reinforcement
Gk	characteristic dead load
H	overall depth of section
Hf	thickness of flange
I	second moment of inertial
L	length of beam
Lc	effective height of a column of wall
M	bending moment
Mu	ultimate moment of resistance

axial load

characteristic live load

V shear force

V shear stress

Vc ultimate shear stresS

O diameter of sted

Z lever arm Oa1

(

1

CHAPTER TWO

2. LITERATURAL REVIEW

As the title of the project is structural design of two storey residential building {4 unit of 3 bedroom maisonette}. It will be important to discuss prehistory residential building, types of residential building know up to date.

2.1 PREHISTORIC RESIDENDIAL BUILDING

Years ago, early man lived on mountains, in caves for protection against

Unfriendly weather, early detection of enemies and defense of war.

Those choose to live on mountain suffered hot scourge of Sunlight and cold experienced after heavy rainfall because of lack of coverage. Others living in Cave suffered hotness of cave in day and night tirire because some of the cave had no vent or opening to provide ventilation.

After peace reigned for several years, early:man decided to build a structure that has a roof and wall [like cave] and opening to allow fresh air [ventilation] into the structure. The modem types of building today originated from the erection of tall, old historic building of the early man.

2.2 TYPES OF RESIDENTAL BUILDING

(1) BUNGALOW; is a type of modem residential building constructed on on level , that is very wide but not very deep from front to back and has rodf that is very flat without stair or suspended slab.

(2) DUPLEX; is a type of modern residential building constructed into two separate homes having stair and apartment with rooms on two floors.

(3) STOREY BUILDING; is a type of residential building constructed into levels of floor ranging from 2 to 10 with stairs usually with one major entrance.

(4j MAISONETTE; is a type of storey building constructed into apartment with a rooms on two or three floors within and usually with a separate entrance.

2. DESIGN LITERATURE REVIEW

It s important to note that steel and concrete structural member are among the commonly used material in building and construction industries .The design of an engineering structure must ensure that

(1) Under the worst loading, the structure is safe.

(2) During normal working condition, the deformation of the member does not detract from the appearance, durability or performance of the structure. Despite the difficulty in assessing the precise loading and variation in the strength of the concrete and steel, this requirement must be met.

Three basic methods using factors of safety to achieve safe, workable structure have been developed and they are

(a) The permissible stress method; which ultimate strength of the material are divided by factor of safety to provide design stress which are usually within the elastic range

(b) The load factor method; which the working loads are multiplied by a factor of safety

(c) The limit state method; which multiplies the working loads by partial factor of safety and also divides the material ultimate strength by further partial factor of safety.

Construction structural members are;

(1) Roof trusses member carries lateral load and is compose of strut and ti~s which is transfer to roof beam

(2) Beam is a member carrying lateral load in bending and shearing which is transfer to the column.

(3) Column is a member carrying axial load in compression and is often subjected to bending also transfer the entire load of structure to the foundation.

(4) Foundation transfer load from the super structure to sup structure or stable soil in efficient uniform manner.

2.4 DESIGN METHOD

Th limit state method of design is used in this project work, because the design method overcomes many of the disadvantages and inconsistencies of the two mdthods discussed earlier. Limit state method, the design of each individual member or section of a member must satisfy two separate criteria of

* The ultimate limit state which ensure that the probability of failure is acceptably low
* The limit state of serviceability which ensure satisfactory behavior under service (i.e working) load. The principal criteria relating to serviceability are the prevention of excessive vibration, but with certain types of structure and in special circumstances. Other limit state criteria may have to be considered are fatigue, durability and fire resistance.

2.5 DESIGN CODES AND STANDARDS

The project work is done to satisfy the requirement of the B.S 8110; PART 1; 1997, and PART 2; 1995 standard use 0'[concrete.

2.6 DESIGN STRESSES

The project work is concerned with two material namely; concrete and rbd reinforcement (steel). The steel is either mild steel, round bar (R-bar) or high Yilld steel (high tensile) bars (Y-bar). Concrete characteristic strength, feu section 3.1 .2 of B.S 8110 of standard specified minimum of grade 25 (fcu=25 n/mm/vZ) or reinforced concrete. Characteristic strength of reinforcement are given in B.S 4449, BS 4461 and BS 4483.For mild steel round bars, the characteristic strength is 2DO n/mm=z while for high tensile bar is 460 N/mm/\2.Experience has shown, how ever, that a value of 410 *N/mm/\2* is the most appropriate of high tensile bars in this country.

2.7 CONCRETE AND REINFORCEMENT

''

Co rcrete is a composite inert material comprising of a binder course e.g cement an mineral filler (body) or aggregate and water. There are basically two types of co crete, viz

* Dense concrete, is the common form of concrete for reinforced concrete work and the average density is 2400 kg/mA3

* Light weight concrete can be defined as those weighting less than 1920 kg/mA3 and are made in densities down to about 160 kg/mA3

REINFORCEMENT

Section 7, ofBS 8110; part 1, specifies that reinforcement should comply with BS 4449, BS 4462 or BS 4483 and that different types of reinforcement may be used in the same structural member. Hence, for a beam, the main reinforcement might be high yield bars while mild steel bars are used for the links,

CHAPTER THREE

3.1 LOADING

The load on a structure is divided into two ways; dead load and live/imposed load.

3.1.1 DEAD LOAD

Dead load is the load of constant magnitude and that is acting permanently on the structure, including self weight.

3.1.3 LIVEfIMPOSED LOAD

Imposed loads are all the loads without constant magnitude and position of acting e.g man

3.1.3WIND LOAD

This is the load due to forces been exerted on the structure as a result of wind action. Although, the wind load is an imposed load, it is kept in a separate category when its partial factors of safety are specified. And when the load combinations on the structure are being considered.

3.1.4

FACTOR OF SAFETY AND LOAD COMBINATION

Various combinations of the characteristic values of dead load Gk, imposed load Qk, wind load and their partial factor of safety must be considered for the loading of the structure. The partial factors of safety specified by BS 8110 and for the ultimate limit state. The loading combinations are

Dead and imposed load =1.4Gk + 1.6Qk

Dead and wind load = 1.0Gk +1.4Wk

Dead, imposed and wind load = 1.2Gk + 1.2Qk + 1.2Wk

The partial factor of safety specified by BS 8110 and for the serviceability limit sta~e,the load combination is usually Yf 1.0 applied to all load combination.

3.2 ROOF DESIGN

DESIGN INFORMATION

Ref	Intended use of building	Residential
	Relevant codes	BS 6399 part 1;1984 BS 8110 part 1;1997 BS 8110 part 2;1988
	Design stresses	*Fcu=20N/mmA2* *Fy=250N/mmA2*
	Exposure condition	One hour for all element Mild for all element Cover; Slab and stair=20mm Beam & column=25mm Foundation=50mm
	Soil condition	Firm gravely lateriti~ clay. Allowable soil bearing capacity= 1*00KN/mA2* Live 10ad=1.5KN/mA2
	General condition	Roof load(*qk+gk)=1.5KN/mA2* Floor finishing=1.2KN/mA2 Wall and rendering=3.4 *7KN/mA2* Screeding=2.0KN/mA2

~

REF	CALCULATION	OUTPUT

The span of the roof is 11.855 with a slope of *301\0.* Therefore for the building 2.44kg/m1\3 corrugated alluminium sheet will be used. The purlins have a span of 5.93m and are space at 900mm center on the plan or slope of 906mm along the slope.

p

P-Reaction of force transfer to the rafter at the note
Ra & Rb -Reaction of force from span a & b

''

jREF	CALCULATIO~N~	~ __OU_TP_U_T_
BS 6399	LOADING DATA	
	Purlin 50 *75mm at 900 mm *c/c*	
	Rafter 50*100mm at 2000 mm *c/c*	
	Roofing sheet is alluminium=Zi-l-lkg/m=S	
	Timber is o.k. on width density=v'Zokg/m=S	
	Acceleration due to gravity=9.81mI\2/s	
	LOADING ON PURLIN	
	Self weight of alluminium roofmg sheet	
	(2.44*9.81 *0.9)11000=0.0 1*25KN/m*	
	Self weight of purlin ,	
	(976*9.81 *0.05*0.075)1100b _	
	=0.03959KN/m	
	TOTAL DEAD LOAD	
	0.0125+0.03959=0.0484KN/m	*Gk=0.0484KN/m*
	Imposed load on roof without access except for maintenance	
BS 8110	Say *=0.75KN/m'''J*	
	TOTAL IMPOSED LOAD	
	*0.75*0.9=0.675KN/m*	*Qk=0.675KN/m*

lEF	CALCULATION	OUTPUT
BS 8110	WIND LOAD ON ROOF Characteristic wind pressure Q=0.613 Vs/\2 Where Vs=Design wind speed =V *Si *S2 *S3 V=Basic speed of the wind (taken as 35m/s) S l=Topography factor (taken as 1.0) S2=Ground roughness, building size and height above ground factor (taken as 0.74) S3=Expected life of building (taken as 1.0)	
	Therefore, the characteristic Wind~~essure Q=0.613(S2*SI *S3*VY2 0.613(0.74* 1*1*35Y2=0.4113~/m	Wk=0.4113KN/m
	DESIGN LOAD ON ROOF Try different load combination *Dead, imposed and wind load 1.2Gk+1.2Qk+1.2Wk 1.2*0.0484+1.2*0.675+1.2*0.4113 =1.36KN/m	DL=1.36KN/m
BS 8110	*Dead and imposed load l.4Gk + 1.6Qk 1.4*0.0484+ 1.6*0.675=1.15KN/m	
	Design load, DL=1.15KN/m	DL=1.15KN/m

I~F	CALCULATION	OUTPUT
	LOAD ON RAFTER 1.15 kNM 1.5M RA RB RA=RB Where W= Design load, L =length of span of rafter : RA=RB= *WL/2={1.15*1.5]/2* =0.8625 KN This is transfer to the rafter at the' nodes. for Internal nodes P =2*0.8625 =1.725 KN Reaction from the roof truss =6.5P =6.5 * 1.725= 11.213KN	Reaction at the internal node =0.8625 KN Reaction at he external nc de =1.725 KN Reaction from roof truss =11.213 KN
	Number of trusses A TO F =11.855/2 =5.92 = 6 Uniform load on the roof beam {6 * 11.213}/1l885 =5.66 KNM	No of trusses =6 . UL on the roof beam =5.66KNM

CALCULATION OUTPUT

DESIGNED ROOF

1.725KN

.8625 B

A J L G

6 @2m=12meters

;

Design member AB assuming pry condition and long term loading SC3 timber ,

RA = RG = 10.35/2 = 5.175 KN

.8625

FAB

Reaction at A&B = 5.175 KN

A

5.175KN

Rl~F	CALCULATION	OUTPUT
BS 526B	.8625 FAB SIN 30	

1

FAR

AT JOINT A

5.175

EV =0

5.175 + FAB SIN 30 - 0.8625 = 0

FAB =[- 5.175 + *0.8625]/SIN* 30=-9.1KN

-9.1KN [COMPRESSION] -9.1KN COMP

EH=O

FAB COS30+FAR

-9.1COS30=-FAH

FAH=7.88KN[TENSON] 7.88KNTENS

COMPRESSION MEMBER

Rafter and Strut

Effective length, le =IL

1*2.31=2.31 m

Try 100 x 50 SC6 SECTION

BASIC DRY STRESSES

crc.adrn/z =12.5 *N/mm"2*

E min = 11800 *N/mm"2*

"

ɾ c,admll=12.5 * kl * k2 * k3 * k8 * k12

Slenderness ratio, = *Le/b =2310/50*

=46.2

	CALCULATION	OUTPUT
BS 5268	E rnin/rrc.adm/z = *11800/12.5 =852.29*	
	From table k12 =0.196 [interpolation]	K120.196
	lJ *c,admll* = acJ! * k, * k2 * k, * kg * k12	
	12.5 * 1 * 1* 1 *1 *0.196	
	= 2.45 *N/mm/\2*	Permissible stress= *2.4 N/mm/\2*
	Actual compression stress *FORCEI* AREA	
	< lJC, adrnl*1*	
	9.1 * 10/\31100*50= 1.82 Nzmm"Z	Applied stress - *1.82 N/mm/\2*
	1.82 N/mm/\2 < 2.45 Nzmm/'? OK	
	Provide 100 x 50 mm	
		Provide 100*50nun
	TENSON MEMBER	
	Post and Beam tie	
	Try 100 x 50	
	Section properties	
	FAH=7.88KN	
	St, , = 7.S *N /mm2*	Permissible stress *=8.46 N/mm2*
	St, adm, = *St, x k3* x kg x k14	
	=7.5 x 1.0 xl x *[300/100]°·11*	
	=8.46 Nlmm2	
BS 5268	*St, applied,* = *F/A*	
	=7.88 x 10 3 1100x 50	Applied stress *=1.58* Nzrnrrr'
	=1.58 Nlmm2	

Si *applied,* $<$ *St, adm,*

"

Provide 100 x50
Tie beam and post

1.58 *N/mm2* < 8.46 *N/mm2*

PrOvide 100 x 50 tie beam

NOTE

Wind load is neglected in this study because

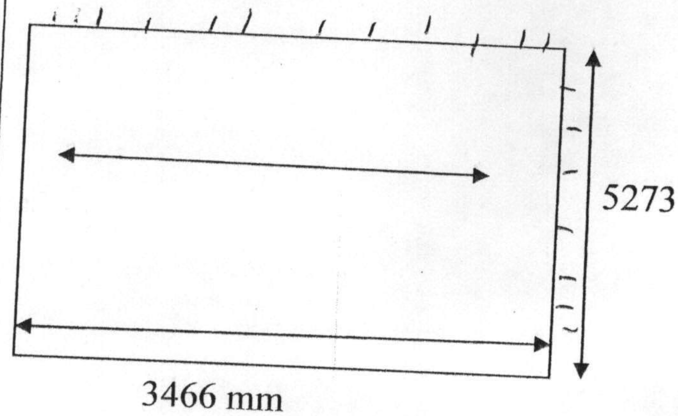
-it effect is horizontal [un necessary] while dead and live load is vertical [important on structure under study]

-Where the structure IS situated wind load obtained from the local wind speedis negligible.

CHAPTER FOUR

4 1.1

REF	CALCULATION	OUTPUT
BS 8110	4.1.1 DESIGN OF SLAB STRUCTURE	
Oyenuga .o	Assumption	
	Fy = 460 Nzmm'.fcu =25 *N/mm/\2* om = 1.15	
	h=150mm, 0= 12mm	
	concrete cover for fire 1hr fire resistance = 25 mm, b =1000 mm	Effective depth d_t = 119 mm
	effective depth in the direction of short span	
BS 6399 part 1 1994 table	*=150-12/2-25=119mm*	d_2 =107 mm
	Long span=150-6-25-12=107mm	
	Fyd = 46011.15 = 400 *N/mm/\2*	*Fy=400N/mm2*
	LOADING	
	Slab self weight = 0.15 *24 =3.6 KNIM2	
	Terrazzo tiles = 0.025 * 22 = 0.55KN1M2	
	Cement mortar = 0.0125 * 20 =; *0-25KNI* m2	DL=6.9KNIM2
	Partition allowance =2.5 Kl~~/m2	
	Total dead load =6.9 *KN/m2*	LL= *1.5KN/M2*
	Imposed load =1.5 *KN/m*	
	Ultimate design load	
	1.4 Gk + 1.6 Qk	
	N =[1.4 * 6.9] + [1.6 * 1.5]=	Design load *12.06 KN/m*
C	*12.06 KN/m2*	

REF	CALCULATION	OUTPUT
BS 8110	SLAB PANEL 1  	
BS 6399 Part 1 1994	Ly/Lx =5273/3466 =1.52 < 2 Two ways spanning	Ly/Lx=1.52
	ULTIMATE MOMENT Msx =$\beta sx\ nL^2$x Msy =$\beta sy\ nL^2$x SHORT SPAN 1.5 - 0.059 1.52 - X 1.75 - 0.065 [1.75 - 1.52]/ [1.75 - 1.5]= [0.065- X] / [0.065 - 0.59] X = 0.059 ; $\beta sx = 0.059$ At mid span Msx = 0.059 * 12.06 * 3.4662 =8.55 KNm Reinforcement K = M/bd^{2}Fcu 8.55 * 10^6/ [1000 * 119^2 *25	

=0.024 < 0.156

BS 8 10

Z = Lad, where La = lever arm table =0.95

Z = Lad = 0.95 * 119 = 113.05mm

As =M *1* [0.87FyZ =

*8.55*10₆1* [0.87* 400*113.05]

=217.33 mm=Z

ProvideY12@300 *C/C* = 377mm2

At edge

1.75 - 0.087

1.52 - x

1.5 - 0.078

=[1.75 - *1.52/1.75* - 1.5]

=[0.087 - *XI* 0.087 - 0.078]

X = 0.079 , *Bsx* = 0.079

Msx =0.079 * 12.06 *3.466~

=11.45 KNM

Reinforcement

K = MuI[bd2Fcu]

=11.45 * *10/\6/[1000* * 119/\2*25

= 0.024 < 0.156

Z =Lad where La=lever arm table

=0.95

Z =Lad = 0.95 *119 =113.05mm

As =M/[0.87FyZ]

*=11.45 *106/[0.87*400*113.05]*

==291.04mm2

ProvideY1*0@200c/top=393mm* 2

CJ-IECKFOR DEFLECTION

Mlbd2=8.55 106/[1000* 1192]*

=0.6

Provide
Y12@300 *C/C* B

Provide
Y10@200 *c/c* T

BS 8 10

Fs =5/8FY Ar/Ap*lIfJ
Fs=5/8*400*[217.33/377]* 111
=144.12 N/m^2
Mf=0.55+[477-Fs]/120[0.9+0.6]
M.F=0.55 + [477-144.12]/180

M.F=2

= 2.4>2
M.F=2

Basic span ratio for continuous slab
=26
Limiting span/depth
=2*26=52
Actual span/depth
=3466/119 =29.13
Actual deflection<limiting deflection
The deflection is ok

Long span
@midspan
Msy=0.034 *12.06 *3.4662
=4.93 KNm
d=-150-6-25-12 =107

Reinforcement
K =M/bd2fcu
= 4.93*106/[1000*1072*25]
=0.017
Z =Lad where La =lever arm table
=0.95
Z = Lad= 0.95 *107 =101.65
As=M/0.87*Fy*Z
=4.93* 106/0.87*400* 101.65
=139.33 mm'
As min=0.13bh/100
0.13 *1000* 1501100= 195mm2
Provide y10 300c/c=262mm2

Provide
YiO @300 c/c B

BS
8110

@Edge

Msy =0.045* 12.06*3.466^2

=6.52KNm

Provide
Y10 @300 c/c T

Reinforcement

K=Mlbd^2

=6.52 * 10^6/[1000*107^2*25]

=0.023<0.156

Z =Lad where la=lever aim table

=0.95* 107 =101.65

As =*MJO.87FyZ*

=6.52* 10^6/[0.87*400* 101.65]

=184.32 mm'

As min=0.13bh/l00

=0.13* 150* 1000/100

=195 mm''

Provide

yl0@300c/top=262 mm^2

,

Ly/Lx=1.52

SLAB PANEL 2

k

BS 8110

5273

~ 3460 ~

ly/lx = *527313460* = 1.52< 2 Two ways slab

ULTIMATE MOMENT

Msx = {3sxnlx2

Msy = {3synlx2

Short sQan;*flsx* or

1.5- 0.055

1.52 - x

1.75 - 0.062

(1.75 - 1.5) / (1.75 - 1.52) = (0.062 - 0.055) /

(0.062 - x)

x = 0.056; {3sx+ = 0.056

Mid span

Msx = {3sxnlx2

= 0.056 x 12.06 x 3.46^2

= 8.09KNm

t

BS 8110

Reinforcement;

$K = Mu / bd^2 feu = 8.09 \times 10^6 / 1000 \times 119^2 \times 25$

$= 0.023 < 0.156$

Z = Lad where la = lever arm table = 0.95

$Z = lad = 0.95 \times 119$ (From lever arm table)

= 113.05mm

$As = M, / 0.87fyZ = 8.09 \times 10^6 / 0.87 \times 400 \times 113.05$

$= 205.64mm^2$

Provide Y12@ 275c/c Bottom ($As = 377mm^2$)

Provide Y12 @275 c/c B

Continuous Edge ; [3s~

1.75 - 0.082

1.52 -- -x

1.5 -- - 0.073

= 1.75 - 1.5 11.75 - 1.52 = 0.082- 0_073/0.082 - x

Therefore, x = -0.074; flsx - ~-0.074

$Msx- = -0.074 \times 12.06 \times 3.46^2$

= 10.68KNm

Reinforcement;

$K = Mu / bd^2 feu = 10.68 \times 10^6 / 1000 \times 119^2 \times 25$

$= 0.03 < 0.156$

Z = Lad where la = lever arm table = 0.95

$Z = lad = 0.95 \times 119 = 113.05mm$

$As = M, / 0.87feuZ = 10.68 \times 10^6 / 0.87 \times 400 \times 119$

$= 271.45mm^2$

Provide Y12@ 300c/c Top ($As = 377mm^2$)

Provide Y12 @ 300 c/c T

CHECK FOR DEFLECTION

~ BS S110

Mlbd2 = 8.09 x 10^6/ 1000 x 119^2= 0.57

Fs = *(5/8)* x (400) x (205.63 *1377)* x 1 /I = *136.36Nlm2*

M.F = 0.55+ [(477 - 136.36) / {120 (0.9 + 0.57)}] <2

= 2.48 > 2.0

SayM.F=2

Limiting span *I* Effective span;

(Allowable span *I* depth ratio) = 2 x 26

=52

Actual span *I* Effective depth = 3460 / 119 = 29.08

Therefore, actual deflection < limiting deflection

The deflection is okay!

M.F=2

Long Span

β₃sx $^+$ = 0.028; *βsx* - = 0.'037

d = 150 - 25 - 126 = 107mm .

since the reinforcement for this span will have a reduce effective depth;

Msx + = 0.028 x 12.06 x 3.46^2= 4.04KNm

Reinforcement

K = M, *I* bd2feu = 4.04 x *10^6/1000* x 107^2x 25 = 0.014 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 107 = 101.65mm

As = M, *I 0.87fyZ* = 4.04 x 10^6 *I* 0.87 x 400 x 101.65 = 114.21mm2

As $_{min}$ =0.13bhl100=195mm^2

Provide YI0@ *300c/c* Bottom (As = 262mm2)

Provide
Y10 @300 c/c B

BS 8 10

Continuous edge

Msx- = 0.037 x 12.06 x 3.462 = 5.34KNm

Reinforcement

K = M, / bd2feu = 5.34 x 10611000 x 1072 x 25 = 0.018 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 107 = 101.65mm

As = M, I *0.87fyZ* = 5.34 x 106 I 0.87 x 400 x 101.65= 150.96mm2

Asmin=0.13 *bh/1* 00= 195mm2

Provide Y10@ *300c/c* TOP (As = 262mm2)

Provide
Y10 @300 *c/c* T

SLAB PANEL 3; P3

BS /273 / / 0J
8110

_ 5266

Ly/Lx=1.00

$lyl\ x = 5273\ /5266 = 1.00 < 2$ Two ways slab

ULTIMATE MOMENT

$\beta sx+ = 0.029$; $\beta sx - = 0.039$

Design load :

Slab selfweight=0.2x24= $4.8KN/m^2$

DESIGN LOAD

=1.4x11.34+ 1.6xl.5=11.34KN

Midspan

$Msx+ = 0.029x\ 13.74\ x\ 5.266^2$

= 11.05KNm

d =200-6-25= 169mm

Reinforcement

$K = M, /bd^2 f_{cu} = 11.05 \times 10^6 / 1000 \times 169^2 \times 25 =$

$0.015 < 0.156$

Z = Lad where la = lever arm from table = 0.95

Z = lad = 0.95 x 169 = 160.6mm

$As = M, / 0.87 fyZ = 11.05 \times 10^6 /$

$0.87 \times 400 \times 160.6 = 197.71mm^2$

Provide Y12@ *300c/c* Bottom (As = 377mm2) Y12 @ 300 *c/c* B

BS 8110

Continuous Edge ; {isx -

f3sx = 0.039

Msx- = -0.039 x 13.74 x 5.2662

= 14.86KNm

Reinforcement

K = M, *I* bd2feu = 14.86 x *106/1000* x 1692x 25 = 0.021 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 169 = 160.6mm

As = M, *I 0.87fyZ* = 14.86 x 106 *I* 0.87 x 400 x 160.6 = 265.89mm2

Provide Y12@ *275c/c* Top (As = 411mm2) Provide Y12 @275 *c/c* T

CHECK FOR DEFL~CTION,

Mlbd2 = 11.05 x 106 *I* 1000 x 1692= 0.39

Fs = *(5/8)* x (400) x (197.71 *1377)* x 1 /1 = *131.12N/m2*

M.F = 0.55+ [(477 - 131.12) *I* {120 (0.9 + 0.39)}] <2 M.F=2

= 2.78> 2.0

SayM.F=2

Limiting span *I* Effective span;

(Allowable span *I* depth ratio) = 2 x 26

= 52

Actual span *I* Effective depth = 5266 *I* 169 = 31.16

Therefore, actual deflection < limiting deflection

The deflection is okay!

Long Span

$\beta_{sx+} = 0.028$; $\beta_{sx-} = 0.037$

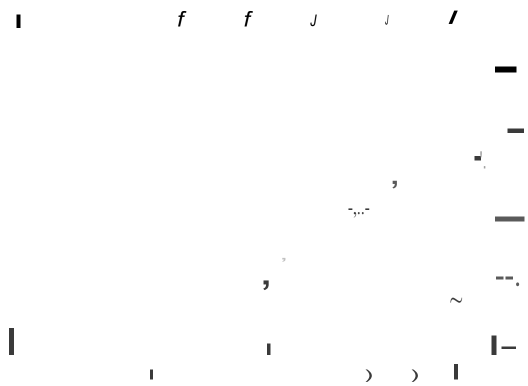
d = 200 - 25 - 126 = 157mm

since, the reinforcement for this span will have a reduce effective depth;

Note: Provide the same reinforcement as above in panel 2

SLAB PANEL 4; P4

3466



2166

Ly/Lx=1.6

l_y/l_x = 3466 / 2166 = 1.6 < 2 Two ways slab

ULTIMATE MOMENT

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

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

Short span; β_{sx+}

1.5- 0.043

1.6-x

1.75- 0.047

(1.75 - 1.5) / (1.75 - 1.6) = (0.047 - x) / (0.047 - 0.043)

x= 0.045

BS~ 110

Mid span

Msx= *psxnlx2*
= 0.045 x 12.06 x $2{,}166^2$
= *2.55KN/m*

Reinforcement

K = M, *1* bd^2feu = *2.S5* x 10^6/1000 x 169^2 x 25 = 0.007 < 0.156
Z = Lad where la = lever arm table = 0.95
Z = lad = 0.95 x 119 = 113.05mm
As = M, *1 0.87fyZ* = 2.55 x 10^6 *1* 0.87 x 400 x 113.05 = $64mm^2$
= As_{min} =0. 13bhll00=$195mm^2$ '"

Provide Y12@ *300c/c* Bottom {As = $377mm^2$)

Provide Y12 @ 300 *c/c* B

Continuous Edge ; *{Js~*

1.5 -- - 0.058
1.6 -- -x
1.75 -- - 0.063
= 1.75 - *1.51* 1.75 - 1.6 = 0.063- 0.058 *1* 0.063-x
Therefore, x = -0.051; *Bsx* - = -0.06
Msx-= -0.06 x 12.06 x 2.166^2
=3.4KNm

Reinforcement;

K = M, *1* bd^2feu = 3.4 x 10^6 *11000* x 119^2 x 25 = 0.009< 0.156
Z = Lad where la = lever arm table = 0.95
Z = lad = 0.95 x 119 = 113.05mm

BS 8 10

$A_s = M_u / 0.87 f_y Z = 3.4 \times 10^6 / 0.87 \times 400 \times 113.05 = 96.11 mm^2$

$A_{s\,min} = 0.13bh/100 = 195 mm^2$

Provide Y10@ 300c/c Top (A_s = 262mm^2)

Provide Y10 @ 300 c/c T

CHECK FOR DEFLECTION,

$M/bd^2 = 2.55 \times 10^6 / 1000 \times 119^2 = 0.18$

$F_s = (5/8) \times (400) \times (64/262) \times 1/1 = 61.07 N/m^2$

M.F = 0.55 [(477 - 61.07) / {120 (0.9 + 0.18)}] < 2

= 3.74 > 2.0

M.F=2

SayM.F=2

Limiting span / Effective span;

(Allowable span / depth ratio) = 2 x 26 = 52

Actual span / Effective depth = 2166 / 119 = 18.20

Therefore, actual deflection <-limiting deflection

The deflection is okay! ~ .

Long Span ~

$\beta_{sx+} = 0.028$; $\beta_{sx-} = 0.037$

Note: Provide the same reinforcement as above in span 4

.-

~

BS8110

Check for shear

V = 108.46 KN

V = V/bd = 108.46 x 103

225 x 407

= *1.18N/mm2*

1.*18<4N/mm2*

100As = 100 x 804 = *0.88N/mm2*

bd 225 x 407

by calculation

Vc = 0.79 (l00AS)1I3 *(400)114*

(bd) (d)

fm of 1.25

V c=O. 79[0.88t33[0.98l25

= *0.75N/mm2*

Shear link

Asv = b [v-vc] = 225 [1.18-0.75]

S, 0.87fyu 0.87 x 250

Asv = 0.44

Sv

Provide R8 links @ 225mm centers

Asu provided = 0.447

Su

Provide RIO @2r)5

Check maximum shear stress

Max shear @ support

Vs = 0.6f - Wu * support width

2

= 0.6 x 180.76 - 34.28 x 0.225

2

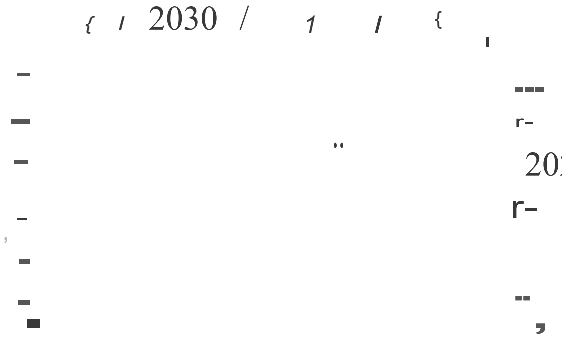
= 108.46 - 3.86 = 104.6KN

Max V = VS = 104.6 x 10^3

bd 225 x 407

=*1.14 N/mm2*

SLAB PANEL 5; P5



Ly/Lx=1

Ultimate ly / lx = 2030 *12021* = 1 < 2 Two ways slab

Moment

{3sx 1=0.03, {3sx 1=0.039

Mid span.

Msx = *Bsxru.x'*

= 0.03 x 12.06 x 2.1212

= *1.63KN/m*

Reinforcement;

K = M, / bd2feu = 1.63x 10611000 x 1192 x 25 = 0.005 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 119 = 113.o5mm

As = M, *I 0.87fyZ* = 1.63 x 106 *I* 0.87 x 499 x 113.05

= 40.67mm2

As min=0.13bhll00=195mm2

Provide YI0@ *300c/c* Bottom (As = 262mm2)

Provide

YI0 @300 *c/c* B

BS S110

Continuous Edge ; {3sx -

/3sx- = 0.039
Msx- = -0.039 x 12.05 x 21212
= 2.1KNm

Reinforcement;

K = M, / bd2feu = 2.1 x 10^6 /1000 x 119^2 x 25 = 0.006 < 0.156
Z = Lad where la = lever arm table = 0.95
Z = lad = 0.95 x 119 = 113.05mm
As = M, / 0.S7fyZ = 2.1 x 10^6 / 0.S7 x 400 x 113.95
= 53.39mm^2
As Min=0.13bh/100=195mm2
Provide Y10@ 300c/c Top (As = 262mm^2)

Provide
Y10 @ 300 c/c T

CHECK FOR DEFLECTION,

Mlbd2 = 1.63 x 10^6 / 1000 x $1i9^2$ = 0.12
Fs = (5/S) x (400) x (53.39 / 262) x 1 // - 50.94N/m^2
M.F = 0.55 +[(477 - 50.94) / {120 (0.9 + 0.12)}] <2
= 4 > 2.0
SayM.F=2
Limiting span / Effective span;
(Allowable span / depth ratio)
=2x26 = 52
Actual span / Effective depth = 2121 / 119 = 17.S2
Therefore, actual deflection < limiting deflection
The deflection is okay!

BS 8110

Long Span

$\beta_{sx}+ = 0.028$; $\beta_{sx}- = 0.037$

Note: Provide the same reinforcement as above in panel 2

SLAB PANEL 6, P6

The panel can be seen as propped cantilever

12.06KN/m

.euom

~1 = $fl^2/10$ = 12.06 x 0.6^2 / 10 = 0.43KNm

Reinforcement; "

d = 150 -25 -6 = 119mm

K = M, / bd^2f_{cu} = 0.43 x $10^6/1000$ x 119^2 x 25 = 0.001 < 0.156 ~

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 119 = 113.05mm

As = M, / $0.87f_yZ$ = 0.43 x 10^6 / 0.87 x 400 x 113.05

= 10.93mm^2

Check for minimum reinforcement

= 0.13bh / 100 = A,

= 0.13 x 1000 x 150 / 100 = 195mm^2

Provide Y12@ *300c/c* Bottom (As = 377mm^2)

Provide the same reinforcement Y12@ *300c/c* as Top and distribution bar for all span

Provide Y12 @ 300 c/c *T/B*

BS 8110

CHECK FOR DEFLECTION,

$M/bd^2 = 0.43 \times 10^6 / 1000 \times 119^2 = 0.03$

$Fs = (5/8) \times (400) \times (195/377) \times 1/1 = 129N/mm^2$

$M.F = 0.55 + [(477 - 129) / \{120 (0.9 + 0.03)\}] < 2$

$= 3.67 > 2.0$

Say M.F=2 — M.F=2

Limiting span / Effective span;

(Allowable span / depth ratio)

$= 2 \times 26 = 52$

Actual span / Effective depth = 600 / 119 = 5.04

Therefore, actual deflection < limiting deflection

The deflection is satisfied!

BS 8 10

Mosh yand Bungyy

4.1.1 DESIGN OF STAIR CASE (TYPICAL)

Oyenuga VO 1999

The stair case plan and cross - section are shown in the architectural drawing

CASE A

250

150 ~

|

-,

150 Waist

Design data;

Waist = 150mm Tread = 250mm

;

Riser = 150mm, cover = 20mm

Feu =25 *N/mm,* F_y =*460N/mm2*

γ_m = 1.15 ~

Total length of goings = 8 x 250

=2000mm

Effective span = L + 0.5 [La + Lb]

La=750mm , Lb=1326

= 2000 + 0.5 (2076)

= 3038mm

D = 150 - 20 - 12/2 = 124mm

Loadings

Waist self weight = 0.15 x 24

=*3.6KNIM2*

Weight of steps = 0.5 x 0.150 x 24

= 1.8 *KN/m2*

Finishing (say) = 1.2 *KN/m2*

Total dead load G_k = 6.6 *KN/m2*

Imposes load Qk = 1.5 *KN/m2*

BS 8 10 Slope factor, .J (2502 + 1502) *I 250*

= 1.166

Slope factor= 1.166

F = (3.6 + 1.2)1.166 +1.8} 1.4 + [1.5]1.6 =12.756 *KN/m2*

Therefore, design load n = *12.756KN/m* 2

Moment

Case a

M=0.125 Fl2

=0.125 x 12.756 x 3.03 82

=14.72KNm

Reinforcement;

K = M, *I* bd2feu= 14.72 x *106/1000* x 1242 x 25 = 0.038 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 124 = 117.&mm -"

As = M, *I 0.87fyZ* = 14.72 x ~06 *I* 0.87 x 400 x 117.8

= 359.07rrun2

Provide Y12@ *200c/c* Bottom (As = 566mm2)

Provide

Y12 @ 200 *c/c* E

CHECK FOR DEFLECTION,

Mlbd2 = 14.72 x 106 *11000* x 1242 = 0.96

Fs = *(5/8)* x (400) x *(359.071* 566) x 1 *II* = *159.44N/nun2*

M.F = 0.55+ [(477 - 159.44) *I* {120 (0.9 + 0.96)}] <2

= 1.97> 2.0

Limiting span *I* Effective span;

(Allowable span *I* depth ratio)

= 1.97 x 20 = 39.4

Actual span I Effective depth = 3038 I 124 = 24.5

BS 8110 Therefore, actual deflection < limiting deflection

Thus, the deflection is satisfied

Second Flight

CASEB

Effective span d = 2000 + 0.5 (1462 + 750) – 3106mm

Moment

Case b = 0.10 Fe

M= 0.10 X 12.756 X 3.1062

= 12.31KNm

Reinforcement;

K=Mu/bd2feu= 12.31 x *106/1000* x 1242x25 = 0.032 < 0.156

Z = Lad where la = lever arm table = 0.95

Z = lad = 0.95 x 124 =117.8rrim

As = M, *I 0.87fyZ* = 12.31 ~ 10^6 I 0.87 x 400 x 117.8

= 301.9mm^2

Provide Y12@ *225c/c* Bottom (As = 502mm2) Provide Y12 @225 B

CHECK FOR DEFLECTION,

Mlbd2= 12.31 x 10^6 *11000* x 1242= 0.8

Fs = *(5/8)* x (400) x *(301.91* 502) x 1 *II* = *150.35N/mm2*

M.F = 0.55+ [(477 - 150.35) *I* {120 (0.9 + 0.8)}] <2

= 2.15 < 2.0

Limiting span *1* Effective span;

BS~ 110 (Allowable span *1* depth ratio)

=2x20 =40

Actual span *1* Effective depth = *31061* 124 = 25.04

Therefore, actual deflection < limiting deflection

Then, the deflection is satisfied

HALF LANDING

Loadings

Span = 750mm

Self weight = 0.150 x 24 x 1.4

= *5.04KN/m* 2

Finishing (say) = 1.2 x 1.4 x1.4

= 2.4 *KN/m2*

Flights = 12.756 x 8 x *0.25/2* x1.4

=*3.29KN/m2*

Live load = 1.5 x 1.4 x 1.6;

= 12.6 *KN/m2*

Total load, W = 23.33 *KN/m2*

23.33KN/m2

lt 750 j

M = w12*l* 8 = 23.33 x 0.752*18*

= 1.64KNm

Reinforcement;

K = l'lu / bd2feu = 1.64 x 106 11000 x 1242x 25 = 0.0043 < 0.156

Z = Lad, lever arm table ,la=0.95

	$Z = 0.95 \times 124 = 117.8$	
BS~ 110	$As = M, / 0.87 f_y Z = 1.64 \times 10^6 / 0.87 \times 400 \times 117.8$	
	$= 40.22 mm^2$	
	$As_{min} = 0.13bh/100 = 195 mm^2$	PrOvide
	Provide Y12@ *300c/c* Bottom and Top	Y12 @ 300 *c/c*
	(As = 377mm2)	B&T

CHECK FOR DEFLECTION,

$M/bd^2 = 1.64 \times 10^6 / 1000 \times 124^2 = 0.11$

$Fs = (5/8) \times (400) \times (40.22 / 377) \times 1 // = 26.67 N/mm^2$

$M.F = 0.55 + [(477 - 26.67) / \{120 (0.9 + 0.11)\}] < 2$

$= 4.27 < 2.0$, M.F=2

Limiting span / Effective span;

(Allowable span / depth ratio)

=2x20 =40; ,

Actual span / Effective depth = *750/* 124 = 6.05

Therefore, actual deflection < limiting deflection

The deflection is satisfied

BS 8 10
Reyn DIdand
SteedPlan
1994

Oyenuga V
1999

4.1.2 DESIGN OF ROOF BEAMS

Assumption / Assignment

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

Fyr = *250N/mm* 2 h = 300mm, 0= 16mm,

om = 1.15, F_{yd}=*400N/mm2*

Concrete cover for 1 hr fire resistance = 25mm

b 225mm, Link = 10mm (say)

d = 300 - 25 - *16/2* - 10 = 257mm

ROOF BEAM

Loadings

Self weight = 0.3 x 0.225 x 24 x 1.4

=*2.27KN/m2*

Load from roof=7.475 x 1.4

=*10.47I<.:1'llm2"*

Rendering & Ceiling hanger

=0.28 x 1.4= 0.392 *KNim2*

Total dead load = *13.13KN/m2~*

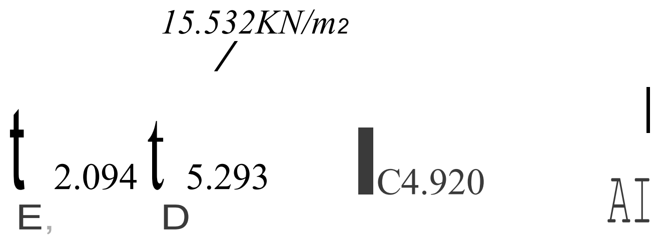
Live load = 1.5 x 1.6 =*2.4KN/m2*

DESIGN LOAD

DL+LL =*15.532KN/m2*

Designed load
=15.532

BSl 110 ROOF BEAM 5



From the diagram, it can be seen by inspection that the beam is symmetrical; both hand side (EDCA) is used for the analysis and design

BENDING MOMENT

At 1st interior support

Mo = Me = -O.llF

Where F =15.532x 5.293=82.21KN

Or 15.532 x 4.920 = 76.42,KN "

Moment at support

Mo =-0.11 x 82.21 x 5.293

=-47.87 KNm

Me= -0.11 x76.42 x 4.92

= - 41.36KNm

Moment at midspan

ME-o=0.09Fl

F =15.532 x 2.094 =32.52KN

ME-o =0.09 x 32.52 x 2.094

=6.13KNm

Mo-e=0.07Fl

=0.07 x 82.21 x 5.293

=30.46KNm

Me-A=0.09Fl

=0.09 x 76.42 x 4.92

=33.84KNm

. SHEAR FORCES

BSI 110

V_E =0.45F=0.45 x 32.52=14.63KN
V_D =0.6F=0.6 x 82.21= 49.33KN
V_C =0.6F=0.6 x 76.42=45.85KN
VA =0.45F=0.45 x76.42 =34.39KN

REINFORCEMENT

At support D , M=-47.87
K=Mlbd2fcu
=47.87 x 10^6/[225 x257^2x 25]
=0.13 < 0.156
No compression steel required
Z =Lad , La from lever arm table
=0.82 x 257 = 210.74 mm'
As -MlO.87FyZ
=47.87 x 10^6/[0.87 x 400 x210.74]
= 552.74 mm2
Provide 3T16 [603 mm"]

Provide
3 Y16 B

At span, M=33.84KNm
bF225 ,b_w=225mm
rectangular beam b=b;

REINFORCEMENT

K=Mlbd2Fcu
= 33.84 x 10^6/[225 x 257^2x 25]
= 0.091
Z =Lad, la from lever arm table
=0.89 x 257 =228.73mm
As =MI[0.87F_yZ]
33.84 x 10^6/[0.87 x 400 x 228.73]
=401.34 mrrr'
Provide 2T16 [402mm2]

Provide
2 Y16T

CHECK FOR DEFLECTION

BS 8110 AT MIDSPAN

Mlbd2= 33.84 xl 06/[225 x 228.732]

=2.87

Fs = 5/8 x Fy x Ar/Ap III

=5/8 x 400 x 425.14/603

=176 N/mm2

M.F=0.55+[477-Fs]1120[0.9+Mlbd2

0.55+[477-176]/120[0.9+2.87]

M.F =1.22<2

Basic span ratio for continues beam =26

Limiting spanld =26 x 1.22 =31.72

M.F=I.22

Actual spanld=4920/210.74

=23.35

Limiting> actual deflection is o.k

CHECK SHEAR

VA= 34.39KN

Shear stress y=v/bd , '=34.39 x 103/225 x257

=0.59N/mm2<0.89\fF cu

1OOAs/bd=100 x 603/225 x 251

=1.04

v; =[0.79[100As/bd] 113[400/d] 1/4]/1.25

=[0.79x 1.041/3x[400/257] 1/4]/1.25

v. =O.38N/mm2

Asv/Sv =b[v-vc]/0.87Fyv

Fyv=250

Asv/Sv =225[0.59-0.38]/0.87x250

=0.22

Provide Rs 300 rom c/c ,

Asv/Sv=0.335

Provide R8 @300 Link

1

BS 8 10

CHECK MAXIMUM SHEAR STRESS at the face of the support

Vs=0.6F -Wu x support widthl2

=0.6 x76.42-15.532xO.225/2

=44.10KN

y=Vs/bd *=44.10/225* x 257

=0.76N/mm < 0.8v'Fcu

O.K

BS 8110

Bending Moment

Code table 3.6 of BS811 0 if the different between the longest and shortest span is not more than 150/0 or providing the dead load is greater than imposed load

MOMENT

M= WL2/8
= 26.89 x 3.462/8
=40. 19KNm

SHEAR FORCE

V =WI/2
=26.89 x 3.46/2
=46.52KN

REINFORCEMENT

Section properties
Overall depth h = 450mn
Web breath {bw} = 225mn
Flange breath bf= bw + 1110 [0.7L] = 225 + 1110 (0.7) [3.46]
= 225.24mn
Effective depth = d = 450 - 25 -(10) - 16/2 = 407mn
Concrete cover = 25mn, fen = 25N/mm2, fy = 460 N/mm2
Partial function steel fm = 1.15
:. fy = 460/1.15 = 400 N/mm2
Durability & fire resistance 1hr
Condition of exposure = mild
M=40.19KNm

$$K = \frac{M}{Bd2fcu} = \frac{40.19 \times 10^6}{225 \{407\}2 \{25\}}$$
= 0.043< 0.156
Z = Lad, La lever arm from table

BS 81 0 4.1.2
MJ Smith
and BJ Bell
1971

Oyenuga V
1999

DESIGN OF FLOOR BEAM

Assumption

$F_y = 460N/mm^2$ steel partial f.s.om = 1.lS , $F_{cu}=25N/mm^2$

b =225mm, h=450mm, (/)= 16mm

$F_{YD}=460/1.15=400N/mm^2$

condition of exposure mild

concrete cover for 1 hr fire resistance= 25mm

d =450-25-10-16/2=409mm

LOADING ON FLOOR BEAM 1

/26.89KNm

-,

3460mm

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 $=1.62KN/m^2$

From panel 2 = 113x 6.9 x 3.46

= $7.96KN/m^2$

From wall = 2.55 KN/m^3 x 3.0

= 7.65 KN/m

Total dead load 17.23 KN/m

Imposed load

From slab panel - 113 x 1.5 x 3.46 = 1.73 KN/m

DESIGN LOAD = 1.4gk + 1.6qk

1.4 x 17.23 + 1.6 x 1.73

=26.89KN/m

BS 8110

Bending Moment

Code table 3.6 ofBS8110 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

MOMENT

M= *WL2/8*

= 26.89 x *3.462/8*

=40. 19KNm

SHEAR FORCE

V =*W1/2*

=26.89 x *3.46/2*

=46.52KN

REINFORCEMENT

Section properties

Overall depth h = 450mn

Web breath {bw} = 225mn

Flange breath bf= bw + 1110[0.7L] = 225 + 1110 (0.7) [3.46]

= 225.24mn

Effective depth = d = 450 - 25 -(10) - *16/2* = 407mn

Concrete cover = 25mn, fen = *25N/mm2,* fy = 460 Nzmm'

Partial function steel fin = 1.15

:. fy = *460/1.15* = 400 N'rnm"

Durability & fire resistance 1hr

Condition of exposure = mild

M=40.19 KNm

$$K = \frac{M}{Bd^2 fcu} = \frac{40.19 \times 10^6}{225 \{407\}^2 \{25\}}$$

= 0.043< 0.156

Z = Lad, La lever arm from table

=0.95 x 407 =386.65

BS 8110 By calculation

AS = 40.19 x 10^6 = 299mu^2

0.87 {400} [386.65}

Provide 3Y16 clc [603] PrOvide 3 y16

Check for shear

V=46.52KN

V = V/bd = 46.52 x 10^3

225 x 407

= *0.51N/rrun2*

100~s = 100 x 402= *0.48N/rrun2*

bd 225 x 407

by calculation

Vc = 0.79 *(100As))/3 (400)114*

(bd) (d)

1.25

= 0.49N/rrun2

Shear link

Asy/sv = b [v-vc] = 225 [0.51-fl.49]

0.87fyu 0.87 x 250 R10@300

Asy = 0.021

Sy

:. No shear reinforcement and nominal links is required

DEFLECTION CHECK

M = 40.19x 10^6 = 1.08

bd2 225 x 407^2

service stress fs

= *5/8* x 400 x 299 x 111

603

=123.96N /rnm'

lS 81 0

$$Mf = 0.55 + \frac{477 - 123.96}{120\,(0.9 + 1.08} = 2.04 > 2$$

m.f=2

Limiting span= $\frac{}{d}$ 2 x 26=52

Actual span = $\frac{3460}{407}$ = 8.50 (d)

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf

= 1.5 x 300 = $450mm^2$

Provide tin @ 250mm *c/c* across the top at the flange to prevent cracking.

BS~110

FLOOR BEAM 2

/26.91KN/m

3466 mm

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m2*

From panel 1 = 1/3 x 6.9 x 3.466

= *7.97KN/m2*

From wall = 2.55 KN/m^3 x 3.0

= 7.65 *KN/m*

Total dead load 17.24 *KN/m*

Imposed load

From slab panel 1 = 113x 1.5 x 3.466

= 1.73 *KN/m*

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 17.24 + 1.6 x 1.73 =*26.91KN/m*

Bending Moment

Code table 3.6 of BS811 0 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

~

B~ 8110

BM

MAXIMUM MOMENT

Mmax=we/8

=26.91 x $3.466^2/8$

=40.41KNm

SHEAR FORCES

V=WI/2

=26.91 x 3.466/2

=46.64KN

REINFORCEMENT

Section properties

Overall depth h = 450mn

Web breath {bw} = 225inn

Flange breath bf= bw + 1I1~ [0.7L]

= 225 + 1110(0.7) [3.466]

= 225.24mn

Effective depth = d = 450 - 25 -(10) -(16/2) = 407mn

Concrete cover = 25mn, fy = 460 $Nhrun^2$

Partial function steel fm = 1.15

:. fy = 460/1.15 = 400 N/mm^2

Durability & fire resistance Ihr

Condition of exposure = mild

M=40.41 KNm

$$K = \frac{M}{Bd^2 fcu} = \frac{40.41 \times 10^6}{225 \{407\}^2 \{25\}}$$

= 0.043 < 0.15

No compression reinforcement

Z = Lad ,La from lever arm table=0.95

Z = 0.95 x 407 =386.65mm

BS 81 0

By calculation

$As = \frac{40.41 \times 10^6}{0.87 \{400\} \{386.65\}} = 300.33$

$= 300.33 mru^2$

Provide 3 y16 *c/c* [603 mrn"]

Provide 3 y16 *T/B*

Check for shear

V=46.64 KN

$V = V/bd = \frac{46.64 \times 10^3}{225 \times 407}$

$= 0.51 N/mru^2$

$0.51 N/mru^2 < 4 N/mru^2$ o.k

$\frac{100As}{bd} = \frac{100 \times 603}{225 \times 407} = 0.66 N/mru^2$

by calculation

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

om of 1.25

$=[0.79[0.66]^{1/3}[0.98]^{1/4}]/1.25$ ~

$= 0.55 N/mru^2$

Since Vc > V Shear reinforcement is not required other than nominal link

Provide R8@225

DEFLECTION CHECK

$\frac{M}{bd^2} = \frac{40.41 \times 10^6}{225.407^2} = 1.08$

service stress fs

$= 5/8 \times 400 \times \frac{300.33}{603} \times 1$

$= 124.51 N/rnnr'$

$Mf = 0.55 + \frac{(477 - 124.51)}{120(0.9 + 1.08)} = 2.03 > 2$

~

BS 81 0 m.f=2

limiting spanld =2 x26 52

Allowable span = 3466 = 8.52
d 407

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = $450mm^2$
Provide R8 @ 250mm c/c across the top at the flange to prevent cracking. R8 @250

BS 8110

FLOORBEAM 3

/31.14KN/m

3460

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 =*L62KN/m2*

w*y=1/2nlx[I-1I3k]*

From panel 4

*Y2*x 6.9 x 3.46 [1-[113 x 1.6]]

= *10.39KN/m2*

From wall = 2.55 *KN/rri3* x 3.U

=*7.65 KN/m*

Total dead load 19.66 *KN/m*

Imposed load

From slab panel 4 =

*Y2*x 1.5 x *3.46[1-[1/3* x 1.6^2]]

=*2.26KN/m*

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 19.66 + 1.6 x 2.26 =*31.14KN/m*

Bending Moment

Code table 3.6 of BS811 0 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BS 8110

BM

MAXIMUM MOMENT

MMAx=W12/8

31.14 x *3.462/8*

=46.60KNm

SHEAR FORCES

Vmax=*WlJ2*

= 31.14 x *3.46/2* = 53.87KN

REINFORCEMENT

Section properties

Overall depth h = 450mn

Web breath {bw} = 225mn

Flange breath bf= bw + *1/10* [0.7L] = 225 + 1110 (0.7) [3.46]

= 225.24mn

Effective depth = d = 450 -.15 -(10) -*(16/2)* = 407mm

Concrete cover = 25mn, fy = 460 *N/mro2*

Partial factor for steel 8m = 1.15

:. fy = 46011.15 = 400 *N/mro2* Provide 3 y161 *IB*

Durability & fire resistance 1hr

Condition of exposure = mild

Mmax=46.60 KNm

$$K = \frac{M}{Bd^2 f_{cu}} = \frac{46.60 \times 10^6}{225\{407\}^2\{25\}}$$

= 0.05 < 0.156

No compression reinforcement

Z =Lad , la from lever arm table

=0.94

Z = 0.94 x 407 =441.8 rom

By calculation

BS ~110

$$A_s = \frac{46.6 \times 10^6}{0.87 \{400\} \{441.8\}} - 303.1mm^2$$

Provide 3 y16 c/c [603mm2]

PrOvide
3 y16 *T/B*

Check for shear

V=53.87KN

$$V = V/bd = \frac{53.87 \times 10^3}{225 \times 407}$$

$= 0.58N/mm^2$

$0.58 < 4N/mm^2$ o.k

$$\frac{100A_s}{bd} = \frac{100 \times 603}{225 \times 407} = 0.66N/mm^2$$

by calculation

$$v_c = 0.79 \left(\frac{100A_s}{bd}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4}$$

om of 1.25

$V_c = [0.79[0.66]^{1/3}[0.98]^{1/4}]/1.25$

$= 0.54N/mm^2$

Shear link

$$\frac{A_{sv}}{S_v} = \frac{b [v-v_c]}{0.87f_{yu}} = \frac{225 [0.58-0.54]}{0.87 \times 250}$$

$$\frac{A_{sv}}{S_v} = 0.04$$

Provide R8 links @ 225mm centers

$$\frac{A_{su} \text{ provided}}{S_u} = 0.447$$

Provide
R8 @225

Nominal link

$$\frac{A_{sv}}{S_v} = \frac{0.4b}{0.87f_{yv}} = \frac{0.04 \times 225}{0.87 \times 250} = 0.41$$

Provide R8 links @ 225mm center

$$\frac{A_{sv}}{S_v} = 0.0447$$

:. No shear reinforcement other than the nominal links is required

BS 110

DEFLECTION CHECK

$\frac{M}{Bd^2} = \frac{46.6 \times 10^6}{225 \times 407^2} = 1.25$

service stress fs

$= \frac{5/8 \times 400 \times 303.1 \times L}{603}$ -

$= 125.66\,N/mm'$

$$Mf' = 0.55 + \frac{477 - 125.66}{120\,(0.9 + 1.25} = 1.91$$

Allowable span = $26 \times 1.91 = 46.66$
d

Actual span = $\frac{3460}{407} = 8.5$
d

limiting> actual hence deflection is on transverse steel

transverse steel required = 15 hf

= $1.5 \times 300 = 450mm^2$

Provide tin @ 250mm *clc* across the top at the flange to prevent cracking.

~

BS 81 0

FLOOR BEAM 4

31.16KN/m

3466

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m* 2

Wy=1/2nlx[I-1I3k]

From panel 4

= *1/2* x 6.9 x 3.466[1-0.33 x1.62]

= *10.40KN/rri2*

From wall = 2.55 *KN/m3* x 3.0

=*7.65 KNlm*

Total dead load *19.67 KN/rtl*

Imposed load

From slab panel 6 =

112x 1.5 x 3.466[1-113 x 1.6] –

2.262 KNlm

DESIGN LOAD = 1.4gk + 1.6qk

= 1.4 x 19.67 + 1.6 x 2.262 =*31.16KN/m*

Bending Moment

Code table 3.6 ofBS81l0 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BS~ 110

BM

MAXIMUM MOMENT

MMAX=We/8

31.16 x *3.466²/8*

=46.79KNm

SHEAR FORCES

V_{MAX}=WI/2

= 31.16 x *3.466/2*

=54KN

MAIN REINFORCEMENT

Section properties

Beam [Beam 3] [panel 4]

Overall depth = 450mm

Web breath, bw = *225mtn*

Flange breath, bf= bw + 1110[0.7L] = 225 + 1110 x 0.7 x 3.466

= 225.24mrn

Effective depth, d = 450-25-t-~
2

= 450-25-10-16/2 = 407mm

Concrete cover = 25mm

fy *=460 N/mm?*

partial factor for steel, fin = 1.15

:. fy = 460 *= 400N/mm2*
1.15

Durability and fire resistance 1hr

C<;mdition of exposure - mild Mmax =46.79KNm

K=M = 46.79 x 10^6 =
Bd^2fcu 225 x 407^2 x25

BS 8 10

005<0.156

No compression reinforcement

Z =Lad , la from lever arm table= 0.94

Z = 0.94 x 407 = 382.58

As

$= \frac{46.79 \times 10^6}{0.87 \times 400 \times 382.58} = 351.44mu^2$

Provide *3T/6 [603]*

Provide 3y16 *T/B*

Check for shear

V=54KN

$V = V/bd = \frac{54 \times 10^3}{225 \times 407}$

= 0.59N/mm2

$\frac{100A_s}{bd} = \frac{100 \times 603}{225 \times 407}$ = *0.69N/mm2*

by calculation

$V_c = 0.79 \left(\frac{100As}{bd}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4}$

om of 1.25

Vc =*[0.79[0.69*^{113} x *0.9814]]11.25*

= 0.69N/mm2

Since V = Vc

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

$\frac{M}{bd^2} = \frac{46.79 \times 10^6}{225 \times 407^2} = 1.26$

service stress fs

$= 5/8 \times \frac{400 \times 351.44 \times 1}{603}$

=145.70 N /mrrr'

~

BS 8110

$$Mf = 0.55 + \frac{(477 - 145.70)}{120\ (0.9 + 1.26)} = 1.83$$

$$\text{Allowable } \frac{\text{span}}{d} = 26 \times 1.83 = 47.58$$

$$\text{Actual } \frac{\text{span}}{d} = \frac{3466}{407} = 8.52$$

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = 450mm^2
Provide R8 @ 250mm c/c across the top at the flange to prevent cracking.

Provide
R8 @250

~

BS8110

FLOOR BEAM 5

34.28KN/m

2094 5273 4920

E b C A

LOADING SPAN DC

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m*$_2$

Wy=1/2nl$_x$[1-1/3k$_2$]

Frompane13

= *1/2* x 6.9 x *5.273[1-113* xf]

= *12.19KN/m*$_2$

From wall = 2.55 *KN/m*$_3$ x 3.0

= 7.65 *KN/m*

Total dead load *21.46 KN/m*

Imposed load

From slab panel 3

= 1I2x1.5x5.273[1-1/3xl] = 2.65 *KN/m*

Design load

1.4x21.46+*1.6x2.65=34.28KN/m*

Mid spans

BS8110 $M_{ED} = 0.09fz$

$=0.09x34.28x2.09^2$ = 13.48 KNm

M_{DC} [interior span] = 0.7fL

= 0.07 x 180.76 x 5.273 - 66.72KNm

M_{CA} = 0.09 fl

$= 0.09 \times 34.28 \times 4.92^2$ = 74.68 KNm

SHEAR FORCES

V_E = 0.45f= 0.45 x 34.28x2.09

= 32.24KN

V_D = 0.6f= 0.6 x 34.28x5.273

=108.46 KN

Vc = 0.6f= 0.6 x 34.28x4.92

= 101.19KN

VA= 0.45f = 0.45 x 34.28x4.92

= 75.90KN

MAIN REINFORCEMENT

Section properties

L Beam [Beam 5] [panel 3]

Overall depth = 450mm ~

Web breath, bw = 225m

Flange breath,bf= bw + *1110* [0.7L] = 225 + *1/10* x 0.7 x 5273

=594.11 m

Effective depth, d = 450-25-t-QL
2

= *450-25-10-16/2* = 407mm

Concrete cover = 25mm

Fcu =*22N/mm2*

fy =*460N/mm2*

partial factor for steel, fin = 1.15

:. fy = 460 / 1.15 =*400N/mm2*

Durability and fire resistance 1hr

BS81 10 Span CA

DEAD LOAD

Self weight of beam *rib=1.62KN/m*

From panel 3

= 112x 6.9 x 4.920[1-113x 12]

= 11.37KN/m

From wall = *7.65KN/m*

Total dead load *=20.64KN/m*

Imposed Load

From panel 3

= *1/2* x 1.5 x 4.92[1-113x 1]

=2.47 KN/m

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 20.64 + 1.6 x 2.47 *=32.85KN/m*

The maximum design load

=34.28KN/m [to be used]

Bending Moment

Code table 3.6 of BS8110 if the different between the longest and shortest span' is not more than 15% or providing the dead load is greater than imposed load

BM

At 1_{st} support of span DC, MD

= O.l1FL

Where f= qL = 34.28 x 5.273

= 180.76KN

MD= -0.11 x 180.76x 5.273

=104.85KNm

Me = -0.11 x 34.28 x 4.92 x 4.92

= *-91.28KN/m*

Condition of exposure – mild

BS BI0 At support D, M

= -104.85KNm

Ultimate moment of resistance

M, = 0.156 bd2fcu

= 0.156 x 225 x 4072 x 25 = 145.37KNm

Mu>Mo

Single reinforcement, No compression reinforcement resistance at

K=M = 104.85 x 10^6 = 0.11

Bdfcu 225x 4072x25

0.11<0.156

Z =Lad ,la from lever arm table

=0.86

Z = 0.86 x 407 =350.02mm

As= 104.85x 10^6 –

0.87x 400 x 350.02

=860.79mm^2

Provide 5 y16 [1010mm^2] Provide 5 y16 T

Mid spans

M = 74.68 KNm, bf= 594.1 hum

K=M = 74.68 x 10^6 =

Dfd2fcu 594.11 x 4072 x 25

0.03<0.156

La from lever arm table=0.95

Z = 0.95d =0.95 x 407 = 386.65mm

As=M_ 74.68x 10^6 =

0.87 fyZ 087 x 400 x 386.65

=555.02mm2

Provide 4y 16 [804mm2] Provide 4 y16 B

1.14N/mm2< 0.8.[lCu = 4N/mm2

BS81 0

End support

Shear distance, d from support face is

:

VA =0.45f-Wu[d + support widthl2]

=0.45xI80.76-34.28[0.407+0.125]

= 81.34 - 18.24 = 63.1KN

V = V = 63.1 x 10^3 = *0.69N/mm2*

bd 225 x 407

Nominal link

&V = O.4b = 0.4 x 225 = 0.41

S, '0.87fyv 0.87 x 250

Provide R81inks @ 225mm center

Asv = 0.447

Sv

Shear resistance of nominal link + concrete i.e

Vn = [Asv 0.87 fyv + bvc] d

[Sv :

=[0.447 x 0.87 x 250 + 225 x 0.75]

407

= 108.78KN > 63.1KN and 108".64KN

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

M = 34.28 x 10^6 = 0.92

Bd^2 $225x407^2$

service stress fs

= *5/8* x 400 x 555.02 xl

804

= 172.58 N /rnm"

~

BS8110

$$Mf = 0.55 + \frac{\sim 477 - 172.58 j}{120 \ (0.9 + 0.92} = 1.94$$

$\frac{\text{Allowable span}}{d} = 1.94x26 = 50.44$

$\frac{\text{Actual span}}{d} = \frac{5273}{407} = 12.96$

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = 450mm2
Provide tin @ 250mm c/c across the top at the flange to prevent cracking.

BS8110 FLOOR BEAM 6

/27.75KN/m

r 4920 t

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m*$_2$

Wy =1I2nlx[1-1I3k]

From panel 2

= *1/2* x 6.9 x 4.92[1-1I3x1.52]

= *8.45KN/m*^{2}

From wall = 2.55 *KN/m*$_3$ x 3.0

=*7.65 KN/m*

Total dead load *17.7ZKNfm*

Imposed load

From slab panel 2

=1/2x1.5x4.92[1-1/3x1.52] = 1.84 *KN/m*

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 17.72 + 1.6 x 1.84 =*27.75KN/m*

Bending Moment

Code table 3.6 of BS8110 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BS8110 BM

Maximum moment

MMAX=wf/8

27.75 x4.922/8

=83.97KNm

SHEAR FORCES

V MAX=W1/2

=27.75 x *4.92/2*

=68.27KN

MAIN REINFORCEMENT

Section properties

[Beam 6] [panel 2]

Overall depth = 450mm

Web breath, bw = 225m

Effective depth, d = 450-25-t-~

2

= *450-25-10-1612* = 409mm :

Concrete cover = 25mm

feu = 25 *N/mm2* ..

fy =460 N/mm_2

partial factor for steel, fin = 1.15

:. fy = 460 = *400N/mm2*

1.15

Durability and fire resistance lhr

Condition of exposure - mild

M*MAX=83.97KN/m*

K=M = 83.97 x 10_6 =0.09

Bd_2fcu 225x 407_2 x25

0.09<0.156

Z= Lad, la from lever arm table

=0.89

Z = 0.89 x 407 =362.23mm

As = 83.97 x 10_6 -

0.87x 400 x 362.23

$=666.13mm^2$

BS811P

Provide 5 y16 [1010mm2]

Provide 5 y16

:

Check for shear

V= 68.27KN

$V = V/bd = \frac{68.27 \times 10^3}{225 \times 407}$

$= 0.75N/mm^2$

$0.75 < 4N/mm^2$ ok

$\frac{100A_s}{bd} = \frac{100 \times 1010}{225 \times 407} = 1.1N/mm^2$

by calculation

Vc = 0.79 (100As)I/3(400i'4

(bd) (d)

om of 1.25

=[0.79[1.1]0.33[0.98]°·25]/1.25

$=0.65N/mm^2$

Shear link ; "

$\frac{Asv}{S_v} = \frac{b\,[v-vc]}{0.87fyu} = \frac{225\,[0.75-0.65]}{0.87 \times 250}$

$\frac{Asv}{Sv} = 0.103$

Provide R8 links @ 225mm centers

PrOvide RIO @225

$\frac{Asu}{Su}$ provided = 0.447

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

$\frac{M}{bd^2} = \frac{83.97 \times 10^6}{225x.407^2} = 2.25$

service stress fs

$= \frac{5/8 \times 400 \times 666.13 \times 1}{1010}$

~

= 164.88 N *Inun*2

BSlII0

$$Mf= 0.55 + \frac{477 - 164.88}{\sim 120 (0.9 + 2.25} j = 1.38$$

Allowable span = 1.38x26 = 35.88
d

Actual span = 4920 = 12.09
d 407

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = 450nun2

Provide tin @ 250nun *c/c* across the top at the flange to prevent cracking.

BS8110

FLOOR BEAM 7

»>: *21.43KN/m*

2094

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 *=1.62KN/m2*

Wy=1I2nlx[I-1/3k]

From panel S

= 112x 6.9 x 2.094[1-l/3xl

= 4.84KN/m2

From wall = 2.55 *KN/m~* x 3.0·

=7.65 KN/m

Total dead load 14.11 *KN/m*

Imposed load

From slab panelS

= l/2x1.5x2.094[1-l/3xl] = 1.05 *KN/m*

DESIGN LOAD = 1.4gk + 1.6qk

= 1.4 x 14.11 + 1.6 x 1.05 *=21.43KN/m*

Bending Moment

Code table 3.6 of BS8110 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BM

BS8110 MAXIMUM MOMENT

MMAX=$Wl^2/8$

=21.43 x $2.094^2/8$

=11.75KNm

SHEAR FORCES

V_{MAX}=W1I2

=21.43x2.094/2

=22.44KN

MAIN REINFORCEMENT

Section properties

[Beam 7] [panel 5]

Overall depth = 450mm

Web breath, bw = 225m

Effective depth, d

= 450-25-10-16/2 = 407mm

Concrete cover = 25mm :

Feu = 25 Nzmm"

:fy =460 N/mm^2

partial factor for steel, fm = 1.15

:. :fy = 460 = $400N/mm^2$

1.15

Durability and fire resistance 1hr

Condition of exposure = mild

MMAX=11.75KN/m

Single reinforcement, No compression reinforcement resistance at

$$K = \frac{M}{Bd^2fcu} = \frac{11.75 \times 10^6}{225 \times 407^2 \times 25} = 0.013$$

0.013<0.156

Z =Lad , la from lever arm table

=0.95

Z =0.95 x 407 =386.65mm

BS81~O

$$A_s = \frac{11.75 \times 10^6}{0.87 \times 400 \times 386.65}$$

$87.33mm^2$

As MfrFO.13bhJ100

=[0.13 x225 *x450]/I00*

$131.53mm^2$

Provide 2 *y16T/B*

Provide 2 y16 [$402mm^2$]

Check for shear

V = 22.44 KN

$$V = V/bd = \frac{22.44 \times 10^3}{225 \times 407}$$

= *0.24N/mm2*

0.24 < *4N/mm2* ok '

$$\frac{100A_s}{bd} = \frac{100 \times 402}{225 \times 407} = 0.44N/n1m2$$

by,calculation

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

Om of 1.25

=[*0.79[0.44t.33[0.98]o.25]11.25*

=0.48N/mm2

Since V < Vc

Provide for nominal link

Nominal link

$$\frac{A_{sv}}{S_v} = \frac{0.4b}{0.87f_{yv}} = \frac{0.4 \times 225}{0.87 \times 250} = 0.41$$

Provide R8 links @ 225mm center

$$\frac{A_{sv}}{S_v} = 0.447$$

~

BS8110

:. No shear reinforcement other than the nominal links is required

·~

DEFLECTION CHECK

$M = \frac{11.75 \times 10^6}{225 \times 407^2} = 0.32$

Bd^2

service stress fs

$= 5/8 \times 400 \times \frac{131.73 \times l,}{402}$

= 81.92 N /mm'

$Mf = 0.55 + \frac{\sim 477 - 81.92)}{120 (0.9 + 0.32} = 3.25 > 2$

Mf=2

Allowable span = 2 x 26 = 52

d

Actual span = $\frac{2092}{407}$ = 5.14

d

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf

= 1.5 x 300 = $450mm^2$

Provide tin @ 250mm *clc* across the top at the flange to prevent cracking.

BS81 10 FLOOR BEAM 8

·It

- > *34.24 KN/m*

5273

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 = *1.62KN/m2*

Wy=1/2nlx[1-1I3k]

From panel 3

= 1I2x6.9x5.273[1-1I3x 1]

= *12.19KN/m2*

From wall = 2.55 *KN/m3* x 3.0

=7.65;KN/irt

Total dead load *21.43 KN/m*

Imposed load

From slab panel 3

= 1/2x1.5x5.273[1-1/3xl] =2.65 *KN/m*

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 21.43 + 1.6 x 2.65 *=34.24KN/m*

Bending Moment

Code table 3.6 ofBS8110 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BS81 0

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

$\frac{M}{bd^2} = \frac{58.76 \times 10^6}{225 \times 407^2} = 1.58$

service stress fs

$= \frac{5/8 \times 400 \times 436.7 \times 1}{603}$

$= 181.05 \, N/mm^2$

$$Mf = 0.55 + \frac{(477 - 181.05)}{120(0.9 + 1.58)} = 1.54$$

Allowable span $= 1.54 \times 26 = 40.04$
d

$\frac{\text{Actual span}}{d} = \frac{4920}{407} = 12.09$

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = $450mm^2$
Provide tin @ 250mm *clc* across the top at the flange to prevent cracking.

BS8 10

FLOOR IN-BEAM 9

CONVERSION OF DROP FLOOR BEAM 8 TO IN-BEAM

Detail information

De&ignload =*34.24KN/m*

Maximum moment =119KNm

Maximum shear force=90.27KN

Shear stress =*0.99N/mm2*

MAIN REINFORCEMENT

Section properties

IN- Beam [Beam 10] [panel 3]

Overall depth = 200mm

Web breath, bw =

Effective depth, d = 200~̧20-t-.§t
2

= *200-20-10-16/2* = 162mm'

Concrete cover = 20mm

feu = 25 N'mm'

fy =460 *N/mm2*

partial factor for steel, fm =1.15

:. fy = 460 = *400N/mm2*
1.15

Durability and fire resistance 1hr

Condition of exposure - mild

As=Area provided=l Zl Omm"

Lever arm Z =333.74mm

K=0.024

K = M2 = 119 x 10₆ = 0.024
Bd feu b x 162₂ x 25

b =[119 x 10^6] 10.024x 25 x162^L

BS8 10

b =756.6mm

b=0.760m

Area required=l Oz-l.e'lmm'

provide 5 y16 @ 150 c/c[1010mm^2]

,

;

Provide

5 yl6T/B @175

~

BS8~10

FLOOR BEAM 11

15.57KN/m

750

LOADING SPAN

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m* 2

Rendering = 0.3 x 0.3

= *0.09KN/m* 2

From wall = 2.55 *KN/m3* x 3.0

= 7.65 *KN/m2*

Total dead load *9.36 KN/m2*

Imposed load

= *1.5KN/m2*

DESIGN LOAD = l.4gk + l.oqk

= 1.4 x 9.36 + 1.6 x 1.5 =*15.57KN/m*

Bending Moment

Code table 3.6 of BS8110 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BM

MAXIMUM MOMENT

MMAX=wf/8

=15.57 x *0.752/8*

=1.09KNm

Provide 2T16 [402mm^2]

BS8 10

Check for shear

V= 5.84KN

V = V/bd = 5.84 x 10^3

225 x 407

= 0.064N/mm2

0.064 <4N/mm^2

100A$_s$ = 100 x 402 = 0.44N/mm^2

bd 225 x 407

by calculation

Vc = 0.79 (100As)l/3(400)114

(bd) (d)

om of 1.25

=[0.79[0.44]°·33[0.98]°·25]/1.25 '"

= 0.47

Since V < Vc

Shear link

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

M = 1.09 x 10^6 = 0.029

bd2 225.x407^2

service stress fs

= 5/8 x 400 x 131.63 x],

402

Provide 2 y16 TIB

= 81.86 N Imm^2

Mf= 0.55 + ~ 477 - 81.86 ~ =4.09>2

120 (0.9 + 0.029

Mf=:2

BS8 10

SHEAR FORCES

$V_{MAX}=Wl/2$

$=15.57 \times 0.75/2$

$=5.84KN$

~

MAIN REINFORCEMENT

Section properties

Beam [Beam 11]

Overall depth = 450mm

Web breath, bw = 225m

Effective depth, d = 450-25-t-~
2

= 450-25-10-16/2 = 407mm

Concrete cover = 25mm

feu $=25N/mm^2$

fy $=460N/mm^2$

partial factor for steel, fm = 1.15

:. fy = 460 = $400N/llll11_2$
1.15

Durability and fire resistance UIT

Condition of exposure – mild

Mmax=1.09KNm

M = 1.09KNm, bw = 225mm

K=M = 1.09×10^6 = 0.001
Bd2fcu $225 \times 407^2 \times 25$

Zr=Lad, la from lever arm table
=0.95

Z = 0.95d = 0.95 x 407 = 386.65mm

A_s=M_ = 1.09×10^6 =
0.87 fyZ 087 x 400 x 386.65

= $8.10mm^2$

$A_{S\,MIN}$ =0.13bh/100

=[0.13 x 225 x450]/I00

$131.63mm^2$

MAXIMUM MOMENT

$M_{MAX} = Wl^2/8$

$= 34.24 \times 5.273^2/8$

$= 119KN/m$

SHEAR FORCES

$V_{MAX} = Wl/2$

$= 34.24 \times 5.273/2$

$= 90.27KN$

MAIN REINFORCEMENT

Section properties

Beam [Beam 8] [panel 3]

Overall depth = 450mm

Web breath, bw = 225m

Effective depth, d = 450-25-t-.0/2

= 450-25-10-16/2 = 407mm:

Concrete cover = 25mm

feu = 25 N/mm^2 ~

fy =460 N/mm^2

partial factor for steel, fin = 1.15

:. fy = 460/1.15 = 400N/mm^2

Durability and fire resistance lhr

Condition of exposure - mild

Mmax ~119KNm

Single reinforcement, No compression reinforcement resistance at

$K = \frac{M}{Bd^2fcu} = \frac{119 \times 10^6}{225 \times 407^2 \times 25} = 0.13$

$0.13 < 0.156$

Z Lad, la from lever arm table

=0.82

BS81 10

$$\frac{\text{Allowable span}}{d} = 2 \times 26 = 52$$

$$\frac{\text{Actual span}}{d} = \frac{750}{407} = 1.84$$

limiting> actual hence deflection is on transverse steel

transverse steel required = 1.5 hf
= 1.5 x 300 = 450mm2
Provide tin @ 250mm *c/c* across the top at the flange to prevent cracking.

Provide
RIO @225

BSS1 0

Z= 0.S2 x 407 = 333.74 mm

As = 119 x 10^6 -
0.S7x 400 x 333.74
1024.61mm^2

Provide 6 y16[1210]

Provide 6y16

Check for shear stress
V=90.27KN
V = V/bd = 90.27 x 10^3
225 x 407
= *0.99N/mm2*
100A~= 100 x 1210= *1.32N/mm*^{2}
bd 225 x 409

by calculation
Vc = 0.79 (l00AS)1I3*(400)1/4*
(bd) (d)
om of 1.25

Vc=[*0.79[1.32]0.33[0.9S]0.25]/1.25*
=0.*69N/mm* ^{2} ~

Shear link
Asv = b [v-vc] = 225 [0.99-0.69]
S, 0.S7fyu 0.S7 x 250

Asv = 0.31
Sv

Provide RS links @ 225mm centers
Asu provided = 0.447
Su

Provide
RIO @225

Nominal link
&V = O.4b= 0.4 x 225 = 0.41
S, 0.S7fyv 0.S7 x 250

~ Provide RSlinks @ 225mm center

BS8110 $\frac{Asv}{Sv} = 0.447$

:. No shear reinforcement other than the nominal links is required

DEFLECTION CHECK

$\frac{M}{Bd^2} = \frac{119 \times 10^6}{225 \times 407^2} = 3.19$

service stress fs

$= 5/8 \times 400 \times \frac{1024.61}{1210} \times 1$

= 211.70 N /mrn'

$Mf = 0.55 + \frac{(477 - 211.70}{120 (0.9 + 3.19} = 1.09$

Allowable $\frac{span}{d} = 1.09 \times 26 = 28.35$

Actual $\frac{span}{d} = \frac{5273}{407} = 12.96$:

limiting > actual hence deflection is on transverse steel

transverse steel required = 1.5 hf

= 1.5 x 300 = 450mm^2

Provide tin @ 250mm *clc* across the top at the flange to prevent cracking .

~

BS81 10 FLOOR BEAM 10

/7KN/m

b 1500 F 3773 C 4920 A

LOADING SPAN FC

Self weight of beam rib

0.225 x 0.3 x 24 =*1.62KN/m2*

Wy =1/2nlx[1-1I3k]

From panel 2

= 112x 6.9 x *3.773[1-1/3x1.6]*

= 6.*14KN/m2*

From wall = 2.55 *KN/m3* x 3.0

=*7.65 KN/m*

Total dead load *15.41 KN/m*

Imposed load

From slab panel 2 ;

= *1/2* x 1.5 x *3.773[1-1/3x1.6]*

1.34 KN/m

Span CA ~

DEAD LOAD

Self weight of beam *rib=I.62KN/m*

From pane12

= 112x 6.9 x 4.92[1-1I3x1.6]

= *8.01KN/m*

From wall = *7.65KN/m*

Total dead load = *17.28KN/m*

Imposed Load

From panel 2

= *1/2* x 1.5 x 4.92[1-1I3x1.6]

= 1.74 *KN/m*

DESIGN LOAD = l.4gk + 1.6qk

= 1.4 x 17.28 + 1.6 x 1.74 =*26.97KN/m*

Bending Moment

BS811 0

Code table 3.6 of BS811 0 if the different between the longest and shortest span is not more than 15% or providing the dead load is greater than imposed load

BM

At 1st support of span FC, MF

= O.l1FL

Where f= qL = 26.97 x 3.773

= 101.76KN

f 1=26.97 x 4.92=132.69KN

MF= -0.11 x 101.76 x 3.773

= *-42.23KN/m*

Mc = -0.11 x 132.69 x 4.92

= *-71.81KN/m*

Mid spans

MEF= 0.09fz

= 0.09 x 26.97 x 1.5^2 =5.46KNm

MFc [interior span] = 0.7FI

= 0.07 x 101.76 x 3.773 = 26.88KNm

MCA= 0.09 fl

= 0.09 x 132.69 x 4.92 = 58.76 KNm

SHEAR FORCES

VE= 0.45f= 0.45 x 40.46

= 18.21KN

VF= 0.6f= 0.6 x 101.76

=61.06 KN

Vc = 0.6f= 0.6 x 132.69

= 79.61KN

VA=0.45f= 0.45 x 132.69

= 59.71KN

=61.57KN

BS8 10 MAIN REINFORCEMENT

Section properties

L Beam [Beam 9] [panel 2]

Overall depth = 450mm

Web breath, bw = 225m

Flange breath, bf= bw + 1110 [0.7L] = 225 + *1/10* x 0.7 x 4920

= 569.4m

Effective depth, d = 450-25-t-QL
2

= *450-25-10-16/2* = 407mm

Concrete cover = 25mm

Feu =*25N/mm2*

fy =*460N/mm2*

partial factor for steel, fm =1.15

:. fy = 460 = *400N/mm2*
1.15

Durability and fire resistance lhr

Condition of exposure - mild

At support C, M

= -71.81KNm

Ultimate moment of resistance

Mu = 0.156 bd^2

= 0.156 x 225 x 407^2 x 25 = 145.36KNm

Mu>Me

Single reinforcement, No compression reinforcement resistance at

K=M = 71.81 x 10^6 = 0.08
Bd'fcu 225 x407^2 x25

Z=Lad ,la from lever arm table

=0.90

Z =0.90 x 407 =366.3mm

As = 71.81 x 10^6 -
0.87x 400 x 366.3

563.34mm^2

BS8110 Provide 4 y16 [804] Provide 4y16 T

Mid spans

M = 58.76 KNm, bf= 569.4mm

$K = \frac{M}{Dfd^2fcu} = \frac{58.76 \times 10^6}{569.4 \times 407^2 \times 25} =$

=0.23

Z=Lad ,la from lever arm table

=0.95

Z = 0.95d = 0.95 x 407 = 386.65mu

$As = \frac{M}{0.87\ fyz} = \frac{58.76 \times 10^6}{0.87 \times 400 \times 386.65}$

436.70mm2

Provide 3T16 [603mm2]

Provide 3Y16 B

Check for shear

Vc = 79.61 KN

$V = V/bd = \frac{79.61 \times 10^3}{225 \times 407}$

= *0.87N/irun2*

$\frac{100A_s}{bd} = \frac{100 \times 603}{225 \times 407}$ = *0.69N/mm2*

by calculation

$Vc = 0.79\ \left(\frac{100As}{bd}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4}$

0m of 1.25

=*[0.79[0.69]°·33[0.98]°·25]/1.25*

=0. *56N/mm2*

Shear link

$\frac{Asv}{S_v} = \frac{b\ [v-vc]}{0.87fyu} = \frac{225\ [0.87-0.56]}{0.87 \times 250}$

$\frac{Asv}{Sv} = 0.32$

BS81 10 Provide R8links @ 225mm centers

Asu provided = 0.447

Su

PrOvide RIO @225

Check maximum shear stress

Mmaxshear @ support

Vs = 0.6f - Wu * support width

2

= 0.6x 132.69 - 26.97 x 0.225 x 0.5

= 79.61 - 3.03 = 76.58KN

Max V = VS = 76.58 x 10^3

bd 225 x 407

=0.84 N/mm2

0.84 N/mm2< 0.8.ffCU = 4N/mm2

End support

Shear distance, d from support face is

VA=,0.45f - Wu [d + support widthl2]

0.45xI32.69-26.97[0.407+ 0.125]

= 59.71 - 14.35 = 45. 3QKN

V = V = 45.36 x 10^3 = *0.50N/mm2*

bd 225 x 407

Nominal link

&v = O.4b = 0.4 x 225 = 0.41

Sv 0.87fyv 0.87 x 250

Provide R8 links @ 225mm center

Asv = 0.447

Sv

Shear resistance of nominal link + concrete i.e

Vn = [Asv 0.87 fyv + bvc] d

[Sv

=[0.447 x 0.87 x 250 + 225 x 0.56]

407

= 90.85KN > 45.36KN and 79.61KN

BS811)

Oyenu]aV
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Reynold &
Steedman

4.1.3 DESIGN OF COLUMNS

Design data

r, $=460N/mm^2$

$F_{cu}=25N/mm^2$

b=230mm

h=450mm

concrete cover=25mm

LOADING

COLUMN 1

[Ground floor to roof level]

Second floor level to roof level

Floor area =2.5 x 3=75m^2

Load from rooftruss=l1.213KN

Load from roofbeam=17.5KN

Column self weight

=0.45 x 0.23 x 24 x 1.4 x 3 :

=10.43KN

TOTAL=39.14KN

First floor level-second floor level

Load from the above =39.14

Load from slab=7.5x12.06=90.45KN

Load from beam=5.5x17.5=96.25KN

Column selfweight= 10.43KN

TOT AL=236.27KN

Ground floor level-first floor level

Load from the above=236.27

Load from slab=7.5x12.06=90.45KN

Load from beam=5.5x17.5=96.25KN

Column selfweight=10.43KN

TOT AL=433 .2KN

LOADING

COLUMN 2

[Ground floor to roof level]

BS8110

Second floor level to roof level

Load from rooftruss=l1.213KN

Load from roofbeam=30.156KN

Column self weight

=0.45 x 0.23 x 24 x 1.4 x 3

=10.43KN

TOTAL=51.80KN

First floor level-second floor level

Load from the above =51.80KN

Load from slab =90.45KN

Load from beam =226.17KN

Load from cantilever=5.5x15.52

=85.36KN

Column selfweight= 10.43~

TOTAL=464.21KN

Ground floor level-first floor level

Load from the above=464.21KN

Load from slab=90.45KN

Load from beam=226.1KN

Load from cantilever=85.36KN

Column selfweight=10.43KN

TOTAL=876.55KN

Base on the above loading , the following columns are designed as representative been the must critical in each group

	Type	Col class	Col ID NO	Loading [KN]	Height
BS8 10	1	biaxial	C,	39.14, 236.27 433.2	GL RL
	2	axial	C_2	51.80 464.21 876.55	GL RL

COLUMN TYPE 1

This is an unbraced biaxial column. The column is 225 x 225 square in cross section

Check for slenderness

Lexlh $=3000/225<10$

=13.33<10[not satisfiedl.The'" column can be adjusted as short or be left without adjustment if considered as unbraced ,the loading from practical point of view may necessitate size increment.

fixed end moment

Slab load

Concrete load =0.15 x24=3.6KN/m^2

Finishes =1.2KN/m2

Partition = 1.00KN/m2

TOTAL G_K = 5.8 KN/m2

QK = 1.5 KN/m2

Design load

5.8 x1.4 +1.6 x1.5 =10.52KN/m2

Beam load

Own load=0.45xO.23x24=2.484KN/m

Finishes =1.00KN/m2

Wall =2.55 x 3 =7.65KN/m2

TOTAL G_K= 11.*13KN/m*

Factored load

=1.4x11.13=15.5SKN/m2

beam 1 and 6 are incidental to biaxial column 1

floor beam [FEM]

BSS 10

The 4.S2m beam external

$W_x=1/3wl_x$

Wy=1/2wlx[1-1/3k2]

Slab load=10.52xO.5x4.92[1-0.231]

=19.90KN/m

Wallibeam load=15.5SKN/m

TOTAL *=35.4SKN/m*

FEM=1I12x4.922x35.4S=71.57KNm

The 4.92m beam -internal

Slab load 2x19.90 *=39.SKN/m*

Beam load *=15.5SKN/m*

TOTAL *=55.3SKN/m*

FEM=1I12x4.922x55.3S=111 :71KNm

The 3.46m beam-external

Slab load= 10.52xO.33x3.46

=12.01KN/m

Beam load=15.5SKN/m

TOTAL *=27.59KN/m*

FEM=1112x3.462x27.59=27.52KNm

The 3.46m beam internal

Slab load *=2x12.01=24.02KN/m*

Beam 10ad=15.5SKN/m

TOTAL *=39.60KN/m*

FEM *=1/12X3.462x39.6=39.51KNm*

i

ROOF BEAM

Roof load =1.5 x1.5 =2.25KN/m

Beam load=2.484xl.4=3 .48KN/m

BS8 10 The 4.92m beam-external

Roof load=

2.25x 0.5x4.92[1-0.231]=4.26KN/m

Beam load =3.48KN/m

TOTAL =7.74KN/m

FEM= 1/12X 4.92₂x7.74=15.61KNm

The 4.92beam-internal

Roof=2x4.26= 8.52KN/m

Beam =3.48KN/m

TOTAL =12KN/m

FEM=1I12X4.922xI2=24.21KN/m

The 3.46m beam- external'

Roofload=

2.25xO.333x3.46 =2.59KN/m-

Beam load =3.48KN/rrt

TOTAL =6.07KN/m

FEM=1I12x3.462x6.07=6.06KNm

The 3.46m beam -internal

Roof load =2.59x2 = 5.18KN/m

Beam load =3.48KN/m

TOTAL =8.66KN/m

FEM=1112x3.462x8.66 =8.64KN/m

Column stiffness

Ie $=bh^3/12$ =[225 $x225^3$]/12

=213574 $x10^6$

K_{col} [2^{nd}-Roof]=213574 x $10^6/3000$

=71.191 x 10^3

K_{col} [floor-floor]=213574 x $10^6/3150$

=67.801 x10^3

BS81,0

BEAM STIFFNESS

Roof beam 1=225 x $450^3/12$

=1708.59 x 10^6

K_{beam}[3.46m]=1708.59 x $10^6/3.46$

=493.812 x 10^3

K_{beam}[4.92m]=1708.59 x $10^6/4.92$

=347.274 x 10^3

Floor beams -3460mm

Flange width =725mm[external]

Flange width=1255mm[internal]:

Ht/h =*150/600 =0.25*

Bw/b =0.31[external]=0.184[inte"mal]

1=0.136[225][600^3]=6609.6 x 10^6E

=0.163[225][600^3]=7921.8 x $10^6$1

Floor beam -4920mm

Flange width =825mm[external]

Flange width=1455mm[internal]

Htih =*150/600 =0.25*

Bw/b =0.273[external]=0.158[internal]

1=0.143[225][600^3]=6949.8 x 10^6E

=0.170[225][600^3]=8262.0 x $10^6$1

MOMENT ON COLUMNS

COLUMN 1

Roof level moment

Mx_x=6.06KNm My_v=15.61KNm

BS81 10 Kcol=71.191

L K Beam=3417+284.8=626.5

Mco1[x_x]=[6.06x71.191]171.191+626.5

= 1.62KNm

MCo1[y_y]=0.62x15.6116.06

=1.98KNm

Second floor level

Mx_x=15.58KNm My_y=71.57KNm

Kcol=71.191,67.801

Kbeam=1322,1168

Moment on foot of column

Mco1[x-x]=

[15.S8x71.191]

71.191 +61.8+ 1322+ 1168

= 1.42KNm

Moment on top of column

Mco1[y-y]=0.42x67.801171.191

=1.4KNm

First floor level

In the x-x direction moment on foot and top of column are the same

Mco1[x-x]=

15.58x67.801

67.801 +61.80+ 1322+ 1158

=1.34KNm

Moment in y-y direction

Mt on roof of col second floor

M=0.42x71.57 *115.58*= 1.93KNm

Others =1.34x71.57/1S.58=1.56KNm

According to the standard, design moment on column

M=0.4M]+0.6M2

MJ is the smaller of the two moment

BS8 10 M2 is the higher foot and top

j

2nd roof Mxx=0.4xl.62+0.6xl.42

=1.492KNm

Myy=O.4xl.6+0.6xl.56

=1.58KNm

1st _2nd Mxx=Myy

0.4xl.4+0.6x1.34=1.364KNm

COLUMN 2

This is a truly central column in which all the opposite beams have their moments balancing out

The design summary

COL	Design parameter	Roof level	2 floor	1 floor
ColI	LoadKN	39.14	236.3	433.2
	M[x-x]	1.4	1.42	1.34
Col 2	load	51.8	464.2	876.6
	M[x-x]	1.4	1.422	1.34

4.1.3 DESIGN OF FOUNDATION

BS8110 Partl~2&3 1997

Oyenuga V

Based on the column with similar loadings, the following foundations basis are design as representative being the most critical in each group

Base type	Col. ID No	Loads (KN)	B - classification
1	CL 1	433.2	Square Pad
2	CL2	876.55	Square Pad

DESIGN DATA

fy = *460N/mm2,* *feu* = *25N/mm2*

b = 230mm *pb* = *150KN/m2*

BASE TYPE-l

N= 433.2KN @ Ultimate limit st~te
h=400mm
column size = 225 x 450
d =400 - 50 - 10 = 340mm
For serviceability limit state;
Design axial load = *433.21 1.46*
= 296.71KN

Allowing for 10% of design load as the self weight of the base.

Therefore, self weight of base =296.71 x 10% - 29.67KN

Therefore, total load - 296.71 + 29.67 =326.38KN

Required base area = Axial load *1 pb* = 326.38 *1* 150 = 2.18m2

~

1st_2nd floor level, N=464.21KN

shear reinforcement

Provide 6-y16 ,RIO@150mm c/c

Provide 6y16
RIO@ 150mm

BS8 10

Ground floor-I st floor N=876.55KN

$$Asc = \frac{N-0.35Fcubh}{0.7Fy-0.35Fcu}$$

$$\frac{876.55x10^3-0.35x25x225x225}{0.7x410 -0.35x25}$$

Asc=1652.68mm2

Provide 6-y16 bars[1884mm2]

Provide RIO @200mm c/c

Provide 6 y16
RIO @200mm

A_{req} =2.18m

Provide 1.5 x 1.5 x *OAm,* (Area = 2.25m2)
For the ultimate limit state,

Design axial load = 326.38KN

Earth pressure = axial load *l* area provided - *326.38/2.25* = *145.06KN/m2*

BS8110

at the column face, shear stress =
NICol. Perimeter x d < *0.8-J[eu* = 326.38 x 10^3
1 1350 x 340 < *0.8-J25*
=0.71<4

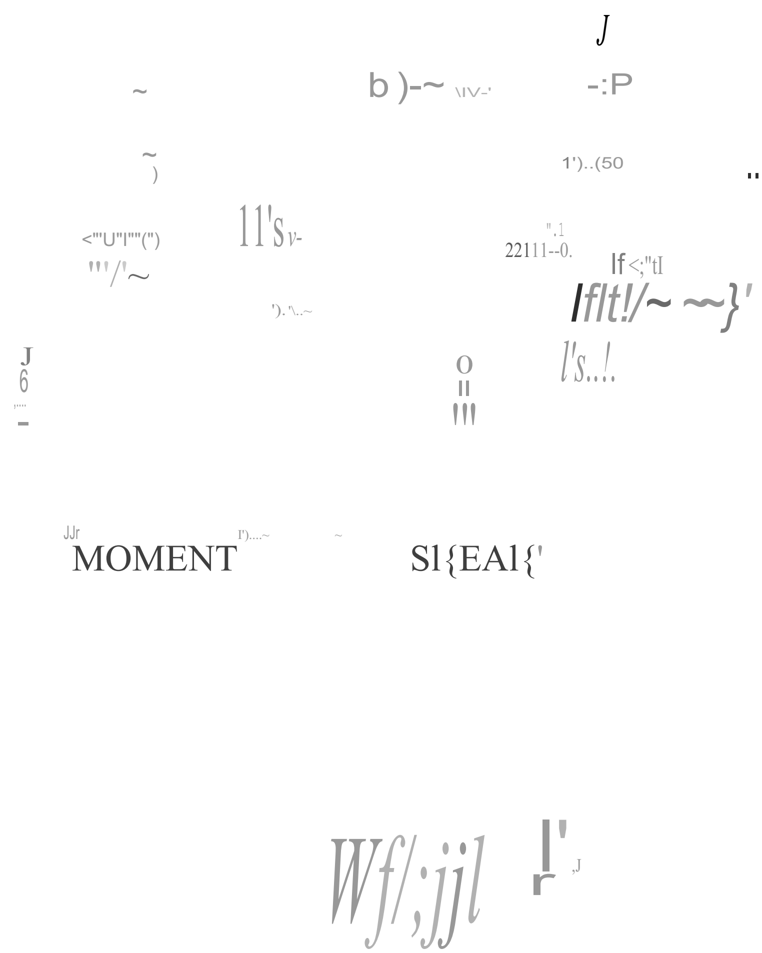
CHECK FORPUNCHITNGSHEAR

Critical perimeter =
(column perimeter) + (8 x 1.5d)
= (1350) + (8 x 1.5 x 340)
= 5430mm.
Area within perimeter = (450+ 3d)2
= (450 + *1020i*
= 2.2 x 10^6 mm^2
Punching Shear force, V =
145.06 (Base Area +Area within per.]
= 145.06 (1.5x1.5 - 2.2)
=7.25KN

Therefore, punching shear stress, v = VIperimeter xd
= 7.5 x 10^3 *1* (1350 x 340)
= *0.016N/mm2*

Comparing this value with the permissible shear of *0.34N/mm2* of table 3.9 of BS 8110; part 1; 1985 (Assuming the minimum 0.15% steel), shows that the base is safe against punching shear with a depth of 400mm

BS8110



Moment

At the column face, span=1.45mm

M=wL2/2 $= 145.06 \times 1.45^2 / 2 = 152.49$KNm per mrun

Reinforcement

$K = M/bd^2f_{cu} = 152.29 \times 10^6 / 1000 \times 340^2 \times 25$
$= 0.053 < 0.156$
$Z = 0.94d = 0.94 \times 340$ (from table of lever arm)
$= 319.6mm$

$As = M/0.95fZ$
$= 152.49 \times 10^6/0.95 \times 460 \times 319.6$

BS8110 $= 1091mm^2$

Minimum steel = 0.13%bh

$0.0013 \times 1000 \times 400 = 520mm^2$

Provide Y16 @ 150 P bottom ($As = 1340mm^2$)

Provide y16 @150B

FINAL CHECK OF PUNCHING SHEAR
$100 As / bd = 100 \times 1340 / 1000 \times 340 = 0.39$
$V_c = 0.63 N/mm^2$ (by interpolation)

Punch shear stress was $0.016 N/mm^2$; therefore, a 400mm thick pad is adequate.

SHEAR STRESS

At critical section for shear, 1.0d from the column face.
$V = 145.06 \times 0.75 = 108.8KN$
$N = V/bd = 108.8 \times 10^3 / 10^3 \times 340 = 0.32 N/mm^2$

Therefore, $0.32 N/mm^2 < 0.63$

The section is adequate in shear

BASE TYPE - 2

N = 876.55KN @ Ultimate limit state
h=400mm
Column size = 450 x 225
D = 400 - 50 -10 = 540rmn

For serviceability limit state;

BS8110 Design axial load = 876.55 *I 1046*
= 600.37KN

Allowing for 10% of design load as the self weight of the base.

Therefore, self weight of base = 600.3 x 10%
=60.04KN

Therefore, total load = *660041KN*

Required base area = Axial load *I pb*
= *660041 I* 150 = *404m2*

Provide 2.1 x 2.1 x *Oo4m,* (Area ~ *4041m2)*
For the ultimate limit state,
Earth pressure=

Design axial load *I* area provided = *660041 I 4.41*
= *149.75KN/m2*

At the column face, shear stress
= *NICol.* Perimeter x d <0.8-1*feu*
= *660041* x 10^3 *12* (450 + 225) x 340 < 0.8-125
= *1044* < 4 *N/mm2*

CHECK FORPUNCfITNG SHEAR

Critical perimeter = (Column Perimeter) + (8 x 1.5d)

~

=[2(450 +225) + (8 x 1.5 x 340)
=5430mm.
Area within perimeter = (450 + 3di
= (450 + 1020/
= 2.2x 10^6 mm'
Punching Shear force,
V = 149.75 (Base Area +Area within p
= 149.75[4.41-2.2]
=330.95KN

BS8110 Punching shear stress v=
V/perimeter x d
=330.95 x$10^3$11350x340

=0.*72N/mm2*

Comparing this value with the permissible shear of *0.34N/mm2* of table 3.9 of BS 8110; part 1; 1985 (Assuming the rmrumum 0.15% steel), shows that the base is safe against punching shear with a depth of 400mm

/ / / / / 17// || / / I / /

BS8110

MOMENT SHEAR

Moment

At the column face, span=1.5mm
M=wL2/2 = 660.41 x 1.12*I* 2 = 363.23KNm per mrun

Reinforcement

K=M/bd$_2$ *feu* =
363.23 x 10^6 / 1000 x 340^2 x 25
= 0.13 < 0.156

Z = 0.95d = 0.95 x 340 (from table oflever arm)
=323mm

$A_8 = M / 0.95 f_y Z =$

$363.23 \times 10^6 / 0.95 \times 460 \times 323$

$= 2573.34mm$

Minimum steel = 0.13%bh

$0.0013 \times 1000 \times 400 = 520mm^2$

Provide Y20 @ 125 P bottom ($A_s = 2510mm^2$) Provide y20 @125 b

BS8110

FINAL CHECK OF PUNCHING SHEAR

$100 A_s / bd = 100 \times 2510 / 1000 \times 340 = 0.74$

$v_c = 0.72N/mm^2$ (by interpolation)

from table

Punch shear stress was 0.72N/mm2; therefore, a 400mm thick pad is adequate.

SHEAR STRESS

At critical section for shear, 1.0d from the column face.

$V = 660.41 \times 0.19 = 125.48KN$

$v = V / bd = 125.48 \times 10^3 / 10^3 \times 340 = 0.36N/mm^2$

Therefore, $0.36N/mm2 < 0.72$

The section is adequate in shear

CHAPTER FIVE

5.0 ANALYSIS AND DISCUSSION OF RESULTS

.1 ANALYSIS OF RESULTS

5.1.1 ROOF

The critical dead and impose loads on the roof is *1.5KN/m* which is the higher force in the roof members governs the design. This load is acting on member AB and number of trusses is 6

The reaction at AB members is 5.175KN with magnitude of permissible stress of *2.4N/mm2* and applied stress of *1.82N/mm2*

However, the critical load that governs the design of strut is 12.213KN acting on members. Similarly the uniform moment acting on the roof beam 5.66KNm.provision of 100 x 50mm strut was made

Meanwhile the effect of wind load was not taken into consideration as the slope angle calculated is 28.3 ° < 30°. Satisfy the condition of not determine the wind ~~e~d

5.1.2 TYPICAL FLOOR SLAB

All the slabs are design typically as two - ways spanning slab except the propel cantilever slab spanning 2m as P6, slab with PI having ly/lx = 4.5/4 = 1.13 and the mid span moment for short and long span of 13.18KNM and 10.18KNM respectively.

Continuous edges have moment of 17.37KNM and 13.48KNM respectively. This implies that the moment in longer direction is move critical than in longer direction. Also slab panel P2 with ly/lx = *514* = 1.25 have mid span moment of 13.48KNM and 8,39KNM continuous edges have moment of 17.67KNM and 11.08KNM hogging respectively which also indicates that the moment in shorter direction is greater than that in longer direction. This behavior is similar to that of o her slab panels while that of cantilever slab is 7.488KNM running through the s .ab panel as uniform distributed load (udl).

5ľ1.3 ROOF BEAM

The critical roofbeam - 5 is two-span continuous beam but symmetrical and hence assumed simply supported. It has the maximum span moments 14.60KNM, 47.87KNM and 34.94KNM at span AC, CD and DE respectively while the maximum shear force at each span are; 26.02KN, 22.53KN and 19.31KN for span AC, CD and DE respectively.

However, because the critical beam which is roof beam 5 IS nominal reinforcement, other roof beam members are generally taken to be nominal reinforcement

5.1.4 FLOOR BEAMS

Floor beam - 1 is a three-span continuous beam but symmetrical and he tee assumed simply supported. The maximum span moments for spans, AC, CD d DE are 171.47KNM, 318.20KNM and 605.90KNM respectively. Shear force and bending moment at supports A, C, D and E are 87.70KN, 284.24KN, 304.46KN, 161.95KN, 0, 91.36KNM, 201.92KNM and 0 respectively.

Floor beam -2 is also a three-span continuous beam having maximum span moments for spans AC, CD, and DE as 218.88KNM, 384.77KNM and 698.02KNM respectively.

The shear force and bending moment at support is such that, at A, C, D and E are 1l0KN, 349, 8KN, 355.94KN, 185.03KN 0, 118.35KNM 234.55KNM, 0 respectively.

imilarly roof beam 3 is a two span continuous beam. The maximum span oments for span 1-4, *4-S,* are I88.42KNM and 102.49KNM respectively. Shear 'orces and bending moment at support, (1), (4), and *(S)* are; IS8.S9KNM 24.89KN, 16~.96KN, 100.44KNM and 0 respectively.

Floor beam - 4 and 7 are typical two-span continuous beam. The maximum span moment that governs the design for span 1-4 and 4-*S* are *2S1.08KNM* and *136.S6KNM* respectively. Shear force and bending moments at support (1), (4) and *(S)* are 2I1.32KN, 166.41KN, *ISS.8SKN,* I33.84KNM and 0 respectively.

For floor beam - Sand 6 are typical two-span continuous beams having the maximum span moments that governs the design for span respectively. The shear forces and bending moment at support (1), (4) and *(S)* are 244.23KN,]92.33KN, 180.I2KN, *lS4.68KNM* and 0 respectively.

However, the floor beam - 8 also a two-span continuous having the maximum span moments governs the design for span 1-4 and *4-S* as *172.0SKNM* and 93.44KNM respectively. The shear forces and bending moment vat support (1), (4), and (5) are 144.7KN, 113.76KN, 106.64KN; 91.58KNM and 0 respectively.

5.1.5 COLUMNS

Column Al starts from foundation and extends to the roof floor level. The bi xial loaded at the loads acting on the column are 84.16KN, *210.62SKN,* *1S7.0* KN, *193.SSKN* respectively. (third floor level to roof level, second floor level tofirst floor level).

Column C2 is axially loaded and starts from foundation and terminates at roof level. The axial loads acting on the column are 101.986KN, 164.892KN, 227.798KN, and 290.740KN respectively, (third floor level to roof level, second floor level to third floor level, first floor level to second floor level, ground floor level to first floor level to second floor level, ground floor level to first floor level).

olumn Al is biaxial and starts from foundation and terminates at roof level. The a ia1 loads and moments acting on the column are 84.16KN, Mxx = 79.00KNM, yy = 86.42KNM (third floor to roof level), 120.625KN and MXX = 59.60KNM, yy = 63.13KNM (first floor to second floor level), 290KN, and Mxx = 59.60KNM, Myy = 63.13KNM (ground floor to first floor level) respectively. Similarly column G1 is also biaxial and starts from foundation and terminates at roof level. The axial loads and moments acting on the column are 96.50IKN, Mxx = 91.34KNM, Myy = 91.65KNM (third floor to roof level), 135.l82KN, Mxx = 59.60KNM, Myy = 59.80KNM (first floor level to second floor level), 212.54KN, Mxx = 59.60KNM, Myy = 56.80KNM (ground floor level to first floor level) respectively.

5.2.0 DISCUSSION OF RESULTS

I was observed from the above floor analysis that the combination of dead and imposed loads is more critical as the wind load is not considered. This may be as result of difference in the magnitude of the force acting on the truss. Then, the dead and impose load is used in the design analysis.

For the analysis of roof beam, it was observed that there is comparatively large difference in moment which may be due to significant difference in span. Therefore, each span was designed separately.

For the floor beams analysis of a continuous beam, the differences in moments may be due to difference in loads as well as in span. While some have a very close span moment value, the latter was used for the design of reinforcement in the span and some are design separately because of large difference in moment.

However, for the column design, for the axially loaded column, it was observed that the difference in axial loads may be due to the member of floors the incidental beams are carrying. For the biaxial type column. The difference in axial loads may also be due to the member of floor~the incidental beams are carrying.

5.3 CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

A structure is designed to perform a certain function. To perform this function satisfactorily it must have sufficient strength and rigidity. Certain-factors must, therefore, ne taken into consideration when designing structure if any kind, especially a public structure. These factors are safety, economy, durability, fire resistance etc. it is against this background that all these factors are exclusively and strictly bear in mind when the design aspect of this project thesis was been done.

However, detailed structural drawing of various structural elements such as roof, slabs, beams, columns, and foundations were successful1yproduced for use by the construction team such as contractor responsible for the supply of steel (reinforcement), iron fixer (or bender) quantity surveyor in the production of biII of quantities etc.

Therefore, if this proposed residential building structure given the architectural and structural drawings specifications is translated into physical structure, will stand the test of time.

RECOMMENDATIONS

lthough this thesis effectively dealt with the analysis and design of both super nd substructures, it does not include detailed analysis of the nature of the soil strata in the project site with a view to determing the bearing capacity of the soil, 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.

The bearing capacity of the soil used for this design was assumed to be *150KN/m2.* For further study, I hereby recommend that adequate soil analysis should be carried out before preliminary assumption of the bearing capacity of the proposed site.

For economical factor, I hereby recommend that the roof design can be carried out using timber material.

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APPENDIX

APPENDIX

A. BASIC LOADS

Concrete specific weight	24.0 kN/m^3
Light - weight concrete	7.0 - 18.0 kN/m^3
225mm block work	*2.87 kN/m^2*
150mm block work	*2.15 kN/m^2*
Wall finishes - both sides	*0.60 kN/m^2*
13mm rendering	0.30 kN/m^2
37mm screeding	*0.80 kN/m^2*
Terrazko paving	0.022kN/in2 per mm.
Roofing felt and screed	*2.00 kN/m2*
Roof live load (No access)	0.25 kN/m^2
Wood (average)	8.00 kN/m^3
Asbestos roofingsheet, sheeting rails and nails	0.40 kN/m^2
Amiatus and nails	*0.30 kN/m^2*

B. REINFORCEMENTS

B.1 Reinforcement Stresses

High yield deformed bars	215
Cold worked,	21
bars	215

Note: It is advisable to adopt the following characteristic stresses for steel in this country, based on experience.

- Mild Steel, f_y = 250 N/mm²
- High tensile, f_y = 410 N/mm²

APPENDIX

B.2 Reinforcement Properties

Bar Size (mm)	[illegible]	[illegible]	[illegible]	16	[illegible]	25	32	[illegible]
Area (mm²)	50.3	78.5	113.1	201.1	314.2	490.9	804.2	1257
Perimeter (mm)	25.j	41.4	37.6	50.2	62.8	78.5	[illegible]	125
Weights (kg/m)	0.395	0.616	0.888	1.579	2.466	3.854	6.313	9.864
Lengths per tonne - Mild steel	270	178	122	68	44	28	[illegible]	[illegible]
Lengths per tonne - High tensile	206	132	92	52	33	21	13	8

B.3 Areas for spacing bar spacing - slab, foundation

Size (mm)	Spacing of bar in mm								
	100	125	150	175	200	225	[illegible]	[illegible]	300
8	503	402	335	287	252	223	201	182	16.8
10	785	628	523	449	393	349	314	285	262
12	1130	905	754	646	56b	502	452	[illegible]	377
16	2010	1610	1340	1150	1010	893	804	731	67.0
20	3140	2510	2090	1800	1570	1396	1260	1142	1050
[illegible]	4910	3930	3270	2810	2450	2181	1960	1784	1640
[illegible]	NR	6430	5360	4600	4020	3574	3220	2924	2680
[illegible]	NR	NR	8380	7180	0280	5585	5030	4569	4190

Note: Intermediate values can be pro-rated

N.R - Spacing not recommended

B.4 Areas for specific bar groups - beams Columns

Bar size (mm)	Numbers of bars									
	1	2	3	4	5	6	7	8	9	10
8	50	101	151	201	252	302	352	402	453	503
10	78	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	[illegible]	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	[illegible]	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	[illegible]	7240	8040
40	1260	2570	3770	5080	6280	7540	8800	10100	11300	12600

two way spanning Bending Moment Coefficients for restrained slab.

Case		Short span coefficients for Ratio of ly/lx: 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long Span Coeff. All
L:i	Edge, -ve	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0,032
	Span, +ve	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
	Edge -ve	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
	Span +ve	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0,050	0.028
	Edge -ve	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.037
C	Span +ve	0.03!}	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
	Edge -ve	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.045
	Span +ve	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.034
b	Edge -ve	0.046	0.050	0.054	0.057	0.060	0:062	0.067	0.070	-
	Span +ve	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
	Edge -ve	-	-	-	-	-	-	-	-	0.045
	Span +ve	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
	Edge -ve	0.0~7	0.065	0.071	0.076	0.081	0.084	0.092	0.099	-
	Span +ve	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0,044
J	Edge -ve	-	-	-	-	-	-	-	-	0.056
	Span +ve	0.04~	0.054	0.063	0.071	0.075	0.084	0.096	0.105	0.044
	Span +ve	0.055	0.065	0.074	0.081	0.087	0.092	0.103	0,111	0.056

Note:- ~ Mean1 conti~uOUS edge

hort Spanning (J.s'y

Spanning (J.sy 0.046 0.037 0.029

ote: ~ll Moments here ~e Po~it\ve.

E3. Column Design Chart 3

C. S. means Increase the section of the column.

Asc (Mild Steel Bars) = 0.0042 u fcubh.

Asc (High Tensile Bars) = 0.0026 u fcubh