Computer Aided Analysis and Design of Multistoreyed Apartment

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Project Report

Submitted by

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1. A C K N O W L E D G E M E N T

As the old saying says "Ingratitude Pierces the heart more than a spear, we will be piercing our hearts, if we fail to thank such of those whom our thanks are really due"

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2. INTRODUCTION

GENERAL:

Construction technique and culture of Architect has been very much improved nowadys. However due to radical changes takes place in the civilization and etc., other factors such as reduction in residential area, day by day in urban zones cost factors, tax pattern and facilities in and around the vicinity aestabetic sence etc., has justified the performance of multistoreyed apartments.

With the increase in the numbers of stories, R.C.C farms are designed to take all the loads with the opening between columns enclosed by brick panel walls. This type of construction considerably reduces the weight of the foundations. The earth quake stresses which proportional to the weight of the structure become small.

The design deals with a variety of structural eelements like slabs, beams, staircase, water tank etc. The procedurre followed here in generally is in accordance with Indian standard specification and National Building code.

Scope of Present Work

To cater the need of housing work increasing population, a project consist of an apartment with five storeys is planned and design in the present Project

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PLINTH AREA DETAILS

Ground Floor	= 444.0 Sq.m
First Floor	= 444.0 Sq.m
Second Floor	= 444.0 Sq.m
Third Floor	= 444.0 Sq.m
Fourth Floor	= 444.0 Sq.m

Totally 20 Flats, 4 in each floor are provided in this appartment

ANALYSIS AND DESIGN

The entire design of frames,footings are based on **"LIMIT STATE DESIGN"** as per relevant codes viz. IS:456-1978 and Design Aids to IS:456-1978 etc., Designed mix M15 for concrete and steel grade of Fe415. The analysis is done by **DIRECT STIFFNESS METHOD** and the design has been done by **LIMIT STATE METHOD.**

ORIENTATION OF BUILDING

The orientation of the building should be such that it attains maximum benifit from the nature and at the same time it is protected from harmful effects.

VENTILATION

The object of ventilation is to maintain air circulation in the rooms and in the building for proper ventilation windows should have minimum area of 1/8th of floor area of the room and the area of doors and windows should not be less than 1/4th of the floor area of the room.

DRAWING ROOM

Drawing room should be well lighted and ventilated and this room services as a recreation room, study room, entertaining room etc.

DINING ROOM

Dining room is provided with a cup-board and a wash basin.

BED ROOM

Bed rooms should be located so that they are ventilated well and at the same time provide privacy.

KITCHEN

It should be provided in rear corners of the building. It should be connected with dining room and well ventilated. Sink should be provided for washing facilities and sufficient number of shelves should also be provided.

BATH AND WATER CLOSET

Bath and water closet should be provided in rear~~y~~ of the building seperately so that the two can be used at a time. Good ventilation should be provided and should be fitted with shower,wash basin,shelves,brackets etc.

STAIRCASE AND LIFT

The staircase are well ventilated and lighted and are constructed for easy passage of the inmat~~e~~es. As in this project,the number of stor~~i~~eyes are five,lifts are also provided

WATER SUPPLY AND DRAINAGE

An elevated water tank is provided.Water supply is given to each flat by proper network of pipe lines.

A septic tank is provided for satisfactory drainage of sewage. Harmful waste water such as water from kitchen can be satisfactorily disposed.

ELECTRICAL WIRING AND DISTRIBUTION

The exact position of all mains,plug points etc.should be determined in advance so that holes,silts etc. can be left in structural units. Recessed conduct type is suggested for wiring.

FLOORING

Now a days it is advisable to use mosaic tile flooring for better performance and appearance. The type ,dimension and laying is accorrding to IS :8Ø9-1957.

DRAWINGS

The detailed drawings showing floor plans, sectional, elevational and structural drawings are also attached.

3. S Y N O P S I S

The project deals with structural design elements.

a] Structural planning

b] Design of structural members

1] Design of slabs

2] Estimation of loads

3] Analysis of structures

4] Design of beams

5] Design of columns

6] Design of footings

7] Design of staircase

8] Design of septic tank

9] Design of elevated water tank

c] Structural and architectural drawing and approximate estimation of the multistoreyed apartment.

The multistoreyed residential flat is a typical reinforced concrete framed structure with a total number of 20 flat.

Each flat of the building with a living room, 2 bedrooms, dining room, kitchen and toilet.

Car parking is provided in the backside of ground floor.Ducts are provided for service line and wastes.A staicase is also provided.

The analysis has been done by **DIRECT STIFFNESS METHOD** and the design has been done by **LIMIT STATE METHOD**. The project has been computerized and the prrogram is written in **FORTRAN - 77**. **M15** concrete and **Fe415** steel bars have been used.

An approximate detailed estimation of the multistoreyed residential flat is also made.

4. STRUCTURAL DESIGN

The structural design of members is done under the following stages.

(a) Structural planning

(b) Design of structural member

- (i) Design of slab
- (ii) Estimation of load
- (iii) Analysis of structure
- (iv) Design of beams
- (v) Design of columns
- (vi) Design of footing
- (vii) Design of staircase
- (viii) Design of septic tank
- (ix) Design of water tank

(c) Structural drawing and detailing

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(a) STRUCTURAL PLANNING

This involves determination of the form of the structure, the materials for the same, the structural system, the layout of its components, the method of analysis and the philosophy of structural design.

After getting an architectural plan of the building, the structural planning of the building frame is done. This involves determination of the following.

(a) Column position

(b) Beam location

(c) Spanning of slabs

(d) Layout and planning of staircase

(e) Type of footing

(b) DESIGN OF STRUCTURAL MEMBERS

In our project, we have chosen the limit state method for the design of structural members.

The main object of limit state method is to achieve an acceptable probability that a structure will not become unserviceable in its life, i.e. it will not reach a limit state. A structure with appropriate degree of reliability should be able to withstand safely all loads that are liable to act on it throughout its life and it should also satisfy the serviceability requirements, such as, limitations on deflection and cracking.

In the limit state design, the loads, permissible stress and factor of safety are determined. The design loads are obtained including the load factors. The partial safety factors are based on statistical and probabilistic grounds. These all factors are based on statistical and probabilistic grounds. These all factors have been considered and the design program is written in FORTRAN-77. Thus this method is a more scientific approach for the design of reinforced concrete structures.

[1] DESIGN OF SLABS

INTRODUCTION

Slabs are designed by limit state method using IS: 456--1978. All the slabs are monolithic with the beams on all the four edges.

Liveloads for slabs are taken from IS: 875 - 1978 The bending moments for two way and one-way continuous slabs are determines using the bending moment co-efficient given in IS: 456--1978.

ROOF SLAB:

SLAB: 1 (3.66m x 2.44m)

Assume overall thickness of slab as 140mm and effective depths as 120mm.

$$L_x = 2.44 + 0.12 = 2.56m$$

$$L_y = 3.66 + 0.12 = 3.78m$$

$$\frac{L_y}{L_x} = \frac{3.78}{2.56} = 1.48 < 2$$

Here, two way slab.

TO FIND TOTAL FACTORED LOAD:

Self weight of slab = 0.14 x 25	=	3.50 KN/m^2
Weight of weathering course	=	2.25 KN/m^2
Total dead load	=	5.75 KN/m^2
Live Load (access provided)	=	1.50 KN/m^2
Total Load	=	7.25 KN/m^2
Total factored load	=	1.5 x 7.25
	=	10.875 KN/m^2

SHORT SPAN COEFFICIENT:

Negative moment at support	=	0.057
Positive moment at midspan	=	0.043

LONG SPAN COEFFICIENT

Negative moment at support	=	0.063
Positive moment at midspan	=	0.043

DESIGN MOMENT

	α	$M = WLx^2$
Long span		
- Support	0.053	3.72×10^6 N -mm
- midspan	0.043	3.02×10^6 N-mm
Short Span		
- Support	0.057	4.00×10^6 N-mm
- Midspan	0.043	3.02×10^6 N-mm

For M15 - Fe 415

$$Ru\ max = 0.36\ \sigma ck\ xumax\ (1-0.42\ xumax)$$

$$= 2.07\ N/mm^2$$

$$Mu\ max = Rumax \times b.d^2 = 2.07 \times 1000 \times 120^2$$

$$= 29.81 \times 10^6\ N.mm > 4 \times 10^6 Nmm$$

$$(Mu\ max = 4.00 \times 10^6\ N.mm)$$

O.K for strength

$$\text{Effective depth } d = \left[\frac{BM}{0.138\ \sigma ckB}\right]^{1/2}$$

$$= \left[\frac{4 \times 10^6}{0.138 \times 15 \times 1000}\right]^{1/2}$$

$$= 43.95$$

$$d = 44$$

Provide effective depth as 120 overall depth as 140mm

d = Provided = 120mm > d required

Hence O.K.

Main steel area is obtained using equation:

$$Ast = \frac{0.5\ \ ck}{Fy}\left[1 - \sqrt{1 - \frac{4.6M}{\sigma ckbd^2}}\quad b.d\right]$$

LONG SPAN

Area of steel @ mid span

$$Asty = \frac{0.5 \times 15}{415}\left[1 - \sqrt{\frac{1 - 4.6 \times 3.02 \times 10^6}{+ 15 \times 1000 \times 140^2}}\right] \times 1000 \times 14$$

$$= 60.5mm^2$$

Provide 8mm ϕ bars,

$$\text{Spacing of bar} = \frac{\pi/4 \times 8^2}{60.5} \times 1000$$

$$= 830.8mm$$

The spacing of bars 3d or 450mm (3d=360 or 450) whichever is less.

Provided 8mm ϕ bar @ 360mm c/c spacing

$$\text{Asty provided} = (1000 \times \pi/4 \times 8^2)/360$$

$$= 139.63\ mm^2 > 60.5mm2$$

Hence safe

Distribution Steel

$$Ast = 0.12\%bd$$

$$= 0.12/100x1000x140$$

$$= 168mm^2$$

Provide 6mm ϕ bar

$$\text{Spacing} = \frac{\pi/4x6^2}{168} \times 1000 = 168.3mm$$

This Reinforced is provided in the edge strips @ spacing not greater than 5d or 450 whichever is less.

Provided 6mm ϕbars @ 165mm c/c spacing

Ast provided = $171.36mm^2 > 168mm^2$

CHECK FOR SHEAR

$$\text{max SF} = \frac{WLx}{2} = \frac{10.875 \times 2.56}{2}$$

$$= 13.92 \text{ KN}$$

$$\text{Nominal shear stress} = V = \frac{13.92 \times 10^3}{+\ 1000 \times 120}$$

$$= 0.116 \text{ N/mm}^2$$

$$\text{Percentage of steel} = \frac{100 \text{ Ast}}{1000 \times 120}$$

$$= 0.116\%$$

Permissible shear stress c = 0.35N/mm^2

Shear Strength = K. τc

K for D = 120mm = 1.3

Shear strength = 1.3 x 0.35 = 0.455 N/mm^2 is greater than - **OK**

CHECK FOR DEVELOPMENT LENGTH @ SHORT EDGE

$$M1 = 0.8 \text{ fy} \times \text{Ast} \left[d - \frac{\text{Fy Astx}}{\sigma \text{ckxb}} \right]$$

$$= 0.87 \times 415 \times 139.63 \left[120 - \frac{415 \times 139.63}{15 \times 1000} \right]$$

$$= 5.85 \times 10^{6} \text{ N-mm}$$

$$SF = 13.93 \times 10^3 \text{ N}$$

Anchorage length of bars @ 90 including 60mm straight length

$L = 60 + 8\phi = 124\text{mm}$

Development length $Ld = 56\,\phi = 448\text{mm}$

$$56\phi < \frac{1.3 \text{ M1}}{v} + \text{Lo}$$

$$56\phi < 491.4$$

SLAB 6 (6.7m x 2.44m)

Assume thickness of slab is 120mm and overall thickness is 140mm

$$\frac{Ly}{Lx} = \frac{6.82}{2.56} = 2.66>2m$$

Hence one way slab

Total factored load = 10.875 KN/m^2

$$\text{Minimum depth of steel } d = \frac{L}{\alpha\beta\tau\delta\lambda}$$

Let $\alpha=20$, $\beta=1$, $\tau=1$, $\delta=1$ & $\lambda=1$

$$d = \frac{2440}{20} = 122mm$$

Let us adopt overall depth D = 150mm

Total factored load of slab = 10.875 KN/m^2

$$\text{Max BM} = \frac{Wulx^2}{+8}$$

$$= \frac{10.875 \times 2.56^2}{8}$$

$$= 8.9 \text{ KN m}$$

Assume steel consists of 10mm bars with 15mm clear cover.

Eff. depth = 150-15-5 = 130mm

$$\text{Max S.F.} = \frac{Wulx}{+2}$$

$$= 13.3KN$$

Depth of the slab is,

$$BM = 0.138 \ \sigma ck \ bd^2$$

$$d = \sqrt{\frac{8.9 \times 10^6}{0.138 \times 15 \times 1000}}$$

$$= 65.57mm.$$

Adopt eff. depth = 70mm and

D = 90mm

Area of tension is

$$M = 0.87 \times fy \times Ast \times \left[d - \frac{Fy \times Astx}{\sigma ck \times b} \right]$$

$$8.9 \times 10^{6} = 0.87 \times 415 \times Ast \left[120 - \frac{415 \times Ast}{15 \times 1000} \right]$$

$Ast = 216.2mm^2$

Use 10mm ϕbar @ 360mm c/c

$Ast = 218.20 > 216.21mm^2$

Hence O.K.

Temperature Reinforcement equal to 0.15% of the gross concrete area will be provided in the long direction.

$$= \frac{0.15}{100} \times 1000 \times 90$$

$= 135mm^2$

Use 6mm ϕ M.S bars @ 205 mm c/c

$= 28.27 \times 1000/205 = 137.9 > 135mm^2$

Hence O.K.

CHECK FOR SHEAR

$$\% \text{ of tension steel} = \frac{100 \ Ast}{bd}$$

$$= \frac{100 \times 137.9}{1000 \times 90}$$

$= 0.153\%$

Shear strength of concrete for 0.153% steel

$$c = 0.35 N/mm^2$$

$$c = K.\ c = 1.25 \times 0.35 = 0.44 N/mm^2$$

$$v = \frac{Vu}{bd} = \frac{13.3 \times 10^3}{1000 \times 90} = 0.147\ N/mm^2$$

$$0.44 > 0.147\ N/mm^2$$

Hence O.K.

The slab is safe in shear.

CHECK FOR DEVELOPMENT LENGTH

Movement of resistance offered by

10mm ϕbars @ 720 mm c/c

$$M1 = 0.87\ Fy\ Ast \left[d - \frac{Ast \quad x \quad Fy}{Fck.b} \right]$$

$$= 0.87 \times 415 \times 137.9 \left[90 - \frac{137.9 \times 4.15}{15 \times 1000} \right]$$

$$= 4.29 \times 10^6 Nmm$$

$$V = 13.3 \times 10^3 N$$

Let us assume anchorage length Lo = 0

$$Ld < 1.3 \frac{M1}{V} = 56\phi < \frac{1.3 \times 4.29 \times 10^6}{13.3 \times 10^3}$$

$$\phi < 7.49mm$$

Hence O.K.

CHECK FOR DEFLECTION

% of steel at midspan $= \dfrac{100\ Ast}{bd}$

$= \dfrac{100 \times 137.9}{1000 \times 90}$

$= 0.153\%$

Modification factor $\tau = 1.6$

(page 230 Ashok K.Jain)

(Fe 415)

Allowable L/d = $20 \times 1.6 = 32$

Actual L/d = $2560/90 = 28.44 < 32$

Hence O.K.

DESIGN OF FLOOR SLABS

Minimum live load of floor area is taken as 2.ØØ KN/m^2.

SLAB 1 (3.66 X 2.44m)

Assume thickness of the slab as 12Ømm

$$\frac{Ly}{Lx} = 1.48 < 2$$

Two way slab.

Here the slab is discontinuous on one edge.

TO FIND TOTAL FACTORED LOAD.

Self wt of slab	=	Ø.14 x 25 =	3.5KN/m^2
Wt. of weathering coarse	=	2.25KN/m^2	
Live load	=	2.ØØ KN/m^2	
		7.75 KN/m^2	
Total factored load	=	1.5 x 7.75	
	=	11.63 KN/m^2	

Short span coefficient

Positive moment at midspan = Ø.Ø43

Long span coefficient

Positive moment at midspan = Ø.Ø43

Design Moment

Short Span	α	M= WLx^2
-Mid span	Ø.Ø43	3.277 x $1Ø^6$ Nmm
Long Span		
-Mid span	Ø.Ø43	3.277 x $1Ø^6$ Nmm

Bending moment = 3.277×10^{6} N mm

For M_{15} - Fe_{415}

Ru max = $2.07 N/mm_{2}$

Mumax = Rumax bd^{2}

= $2.07 \times 1000 \times 120^{2}$

= 29.81×10^{6} Nmm > 3.277×10^{6} Nmm

Hence it is O.K. for strength

Required effective depth $d = \sqrt{\dfrac{BM}{0.138 \quad ckb}}$

$$= \sqrt{\frac{3.277 \times 10^{6}}{0.138 \times 15 \times 1000}}$$

d = 39.8mm

Provide effective depth as = 120mm

Overall depth D = 140mm

Hence 'd' provided is greater than 'd' required.

Hence O.K.

Short Span

Area of steel required at midspan

$$Astx = \frac{0.5 \times 15}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 3.277 \times 10^{6}}{15 \times 1000 \times 120}}\right] \times 1000 \times 120$$

$= 77.04 \text{ mm}^{2}$

Provide 8mm ϕ bar.

Spacing = 652.5mm

The spacing of bars should be 3d (360mm) or

450mm which ever is less as per IS code.

Provide 8mm ϕ bar @ 360mmc/c spacing.

Ast provided = $1000 \times \pi/4 \times 8^{2}$ = $139.63 > 77.04 mm^{2}$ OK

Long Span

Area of steel at Asty = $77.04 mm^2$

Provide 8mm ϕ@ 360mmc/c spacing.

Astx provided = $139.63 > 77.04 mm^2$

Hence safe

Distribution steel

Provide 6mm ϕbar 165mm c/c spacing

At corners with one edge discontinuous

$$At = \frac{163}{2} = 84 mm^2$$

Provide 6mm ϕ bar @ 335 c/c spacing

At (Provided) = $84.4 mm^2$

Hence safe

Check for shear

$$\text{Max shear force} = \frac{WLx}{2}$$

$$= \frac{11.63 \times 2.56}{2}$$

$$= 14.89 KN$$

$$\text{Nominal shear stress} \quad v = \frac{14.89 \times 10^3}{1000 \times 120}$$

$$= 0.124 N/mm^2$$

$$\% \text{ of steel} = \frac{100 \text{ Ast}}{bd} = 0.116$$

Permissible shear stress c = $0.35 N/mm^2$

K for D = 120mm = 1.3

Shear strength = 1.3×0.35

= $0.455 N/mm^2$

> $0.128 N/mm^2$ Hence safe

Check for development length at short edge.

$$M1 = 0.87\ fy\ Ast \left[d - \frac{Fy\ Astx}{ckb} \right]$$

$$= 0.87 \times 415 \times 139.63 \left[120 - \frac{415 \times 139.63}{15 \times 1000} \right]$$

$$= 5.85 \times 10^{6} Nmm$$

Shear force = 14.89×10^{3}N

Anchorage length of bars at 90 including 60mm straight length

$$Lo = 124mm$$

Development length Ld = $56\ \phi$ = 448mm

$$56\ \phi < 1.3M1/V+Lo$$

$$\phi < 11.33\ mm$$

Hence safe.

Slab 6 (6.7 m x 2.44m)

Assume thickness of slab as 120mm

$$\frac{Ly}{Lx} = 2.66 > 2mm$$

One way slab

Total factored load = 11.63 KN/m^2

Minimum depth D = 150mm

Total factored load of slab = 11.63 KN/m^2

$$\text{Max BM} = \frac{WuLx^2}{8}$$

$$= \frac{11.63 \times 2.56^2}{8}$$

$$= 9.53KNm$$

Assume steel consists of 10mm ϕbar with 15mm clear cover.

Effective depth = 130mm

$$\text{Max shear Force} = \frac{WuLx}{2} = 14.89KN$$

Depth of the slab

$$BM = 0.138\ \sigma ck\ bd^2$$

$$d = \frac{9.53 \times 10^6}{0.138 \times 15 \times 1000}$$

$$= 67.85mm$$

Adopt effective depth = 70mm

and overall depth D = 90mm

Area of tension is,

$$M = 0.87 \times fy \times Ast \times \left[d - \frac{FyAstx}{ckb} \right]$$

$$9.53 \times 10^6 = 0.87 \times 415 \times Ast \left[120 - \frac{415 \times Ast}{15 \times 1000} \right]$$

$$Ast = 232.4\ mm^2$$

Use 10mm ϕ bar @ 335 mmc/c

Ast (Provided) = $234.45^2 > 232.41mm^2$

Hence O.K.

Temperature reinforcement equal to 0.15% of the gross concrete area will be provided in the long direction

$$= \frac{0.15}{100} \times 1000 \times 90$$

$$= 135mm^2$$

Use 6mm ϕ MS bars @ 250 mm c/c

Area provided = $137.9 > 135mm^2$ Hence O.K.

Check for Shear

$$\% \text{ of tension steel} = \frac{100Ast}{bd}$$

$$= 0.153\%$$

$$= 0.35N/mm^2$$

$$c = K. \quad c = 1.25 \times 0.35$$

$$= 0.44/mm^2$$

$$\frac{Vu}{bd} = \frac{14.89 \times 10^3}{1000 \times 90}$$

$$= 0.165n/MM^2$$

$$0.44 > 0.165n/MM^2$$

Hence O.K. (Safe in shear)

Check for development length

Moment of resistance offers by 10mm bar @ 720mm c/c ($Ast = 137.9mm^2$)

$$M1 = 4.29 \times 10^6 \text{ Nmm}$$

$$V = 14.89 \times 10^3 \text{ N}$$

Let us assume $Lo = 0$

$$\phi < 6.68mm$$

Hence O.K.

Check for Deflection

$$\% \text{ of steel at midspan} = \frac{100 \ Ast}{bd}$$

$$= \frac{100 \times 137.9}{1000 \times 90}$$

$$= 0.153\%$$

SCHEDULE OF R.C.C SLAB (ROOF SLAB)

SLAB NO.	DEPTH(mm)	SHORT SPAN STEEL (mm)	LONG SPAN STEEL(mm)	REMARKS
ROOF SLAB:				
S1	140	8 at 240	8 at 240	Two way slip
S2	140	8 at 360	8 at 360	Two way slab
S3	140	8 at 360	8 at 360	Two way slab
S4	140	8 at 260	8 at 385	Two way slab
S5	140	8 at 260	8 at 385	Two way slab
S6	150	8 at 360	8 at 205 (Dist:steel)	One way slab
S7	140	8 at 310	8 at 316	Two way slab

SLAB NO.	DEPTH(mm)	SHORT SPAN STEEL (mm)	LONG SPAN STEEL(mm)	REMARKS
FLOOR SLAB				
S1	140	8 at 240	8 at 360	Two way slip
S2	120	8 at 360	8 at 360	Two way slab
S3	120	8 at 360	8 at 360	Two way slab
S4	120	8 at 170	8 at 360	Two way slab
S5	120	8 at 170	8 at 360	Two way slab
S6	150	10 at 335	6 at 205 (Dist:steel)	One way slab
S7	120	8 at 170	8 at 250	Two way slab

[2] LOAD DATA

ESTIMATION OF LOADS

The loads acting in beams are the shear at the end of the slab. If the beams are supporting two-way slabs, the load distribution is trapezoidal on long beams and triangular on short beams with base angle of 45. The ordinates of trapezoidal and triangular load is qLx/2.

In the case of intermediate beam, it is the sum of slab shears on two sides of the beam. The triangular and trapezoidal loads are converted into equivalent uniformly distributed loads.

In case of oneway slab the load transferred to the beam along the short edge may be approximately taken equal to the area of the triangle having height equal to Lx/4.

In case of slabs supported on secondary beams it is always on the safer side to consider load on secondary beams to be that over rectangular area instead of that over trapezoidal area.

The load separation diagram for each beam member is shown in the drawing.

LOAD CALCULATION

FOR ROOF SLABS:

BEAM B1

Min. Live access provided is 1.5 KN/M2

Area of Triangle	=	1/2 x 1.83 (tan 45 x 1.83/2)
	=	$0.84 m^2$
Live Load	=	$1.50\ KN/m^2$
	=	1.50 x 0.84 = 1.26 KN/m
L.L/m	=	1.26/1.83 = 0.69KN/m
Slab load	=	D.L. of slab + Wt of weathering course
	=	$3.5 + 2.25 = 5.75\ KN/m^2$
	=	$5.75 \times 0.84 = 4.83\ KN/m^2$
Slab Load	=	4.83/1.83 = 2.64 KN/m
Thickness of main wall	=	230mm
Main wall load/m	=	19.2 x 0.23 x 3
	=	10.08 KN/m
25% reduction	=	$\frac{25}{10} \times 10.08$
	=	2.52 KN/m

Load of main wall after reduction

= 10.08-2.52 = 7.56 KN/m

D.L = 2.64 + 7.56 = 10.20 KN/m

Total L.L = 0.69 KN/m

ssume of beam as 150mm x 400 mm

L = 150mm, D = 400mm

Dr = Depth of rib = D-D (Depth of slab)

= 400 - 100

= 300mm

Self weight of beam = 25 x 0.15 x 0.3

= 1.125 KN/m

Total D.L = 10.2 + 1.125

= 11.325 KN/m

Total load over the beam

BM = 11.325 + 0.69

= 12.015 KN/m

Total ultimate load (Mu) = 1.5 x 12.015

= 18.0225 KN/m

BEAM B5

Tan 45 = x/1.28m

Area of Triangle I = 1/2 x 2.56 x x

= 1/2 x 2.56 x 1.28

= $1.64m^2$

Area of Triangle II = 1/2 x 2.56 x 1.28

= $1.64m^2$

L.L/m = 1.5 x 3.28/2.44

= 2.02 KN/m

Slob load = (3.5 + 2.25) x 3.28

= 2.02 KN/m

Slab load = (3.5 + 2.25) x 3.28

= 18.83 KN

Slab load/m = 18.83/2.44 = 7.72 KN/m

Main wall load/m	=	19.2 x 0.23 x 3
	=	10.08 KN/m
25% reduction	=	2.52 KN/m
Load of main wall after reduction	=	7.56 KN/mD.L
D.L	=	7.72 + 7.56 = 15.28 KN/m
Total L.L	=	2.02 KN/m

Assume the size of the beam as 150 x 400 mm

Dr	=	400 - 100 = 300
Self weight of the beam	=	25 x 0.15 x 0.3
	=	15.28 + 1.125
Total D.L	=	16.41 KN/m
Total load over the beam B5	=	16.41 + 2.02
	=	18.43 KN/m
Total ultimate load(Mu)	=	1.5 x 18.43
	=	27.65 KN/m

BEAM B2

Length of the smaller side of trapezium = 4.27 - (1.28 x 2)

= 1.71 m

Area o Trapezium $= \dfrac{1.71 + 4.27}{2} \times 1.28$

$= 3.88\ m^2$

L.L/m $= \dfrac{1.5 \times 3.88}{4.27} = 1.37$ KN/m

Slab load/m = 5.75/4.27 = 1.35 KN/m

Load of main wall after reduction = 7.56 KN/m

DL = 1.35 + 7.56 = 8.91 KN/m

ssume size of Beam as 15Ømm x 4ØØmm

Self weight beam = 25 x Ø.15 x Ø.3

= 1.125 x KN/m

Total D.L = 8.91 + 1.125 = 1Ø.Ø35 KN/m

Total ultimate load (Mu) = 1.5 x (1Ø.Ø35 + 1.37)

= 17.12 KN/m

BEAM B3 & B4

Length of the smaller side of trapezium = 3.47 - (2 x 1.28)

= Ø.91 m

Area of Trapezium $= \dfrac{(0.91 + 3.47)}{2} \times 1.28$

$= 2.81\ m^2$

L.L/m $= \dfrac{1.5 \times 2.81}{2} \times 1.28$

= 1.22 KN/m

Slab load/m $= \dfrac{5.75}{3.47}$ = 1.66 KN/m

Load of main wall after reduction = 7.56 KN/m

D.L = 1.66 + 7.56

= 9.23 KN/m

Therefore Total D1 = 9.23 + 1.125 =1Ø.36 KN/m

Total ultimate load (Mu) = 1.5 x 11.58

= 17.37 KN/m

BEAM B6, B1Ø & B15

Area of Two Triangles = 2 (1/2 x 1.95 x tan45 x Ø.975)

= 1.91 m^2

L.L/m = $\frac{1.91 \times 1.5}{1.95}$ = 1.47 KN/m

Slab load = 5.75 x 1.95

= 1Ø.99 KN/m

Slab Load/m = 1Ø.99/1.95 = 5.64 KN/m

Load of main wall after reduction = 7.56 KN/m

Total D.L = (5.64 + 7.56) + 1.125

= 14.325 KN/m

Total ultimate load (Mu) = 1.5 x (14.325 + 1.47)

= 23.7 KN/m

BEAM B7 & B11

Length of smaller side of trapezium = 4.39 - 2 x (Ø.67)

= 3.Ø5m^2

Area of trapezium = $\frac{3.05 + 4.39}{2}$ x Ø.67 + 3.88

= 6.37 m^2

L.L/m = $\frac{6.37 \times 1.5}{4.39}$ = 2.17 KN/m

Slab load/m = $\frac{5.75 \times 6.37}{4.39}$ x 8.34 KN/m

(Note: There is no wall. Hence no calculation of main wall load)

EAM B6, B10 & B15

Area of Two Triangles = 2 (1/2 x 1.95 x tan45 x 0.975)

= 1.91 m^2

L.L/m = $\frac{1.91 \times 1.5}{1.95}$ = 1.47 KN/m

Slab load = 5.75 x 1.95

= 10.99 KN/m

Slab Load/m = 10.99/1.95 = 5.64 KN/m

Load of main wall after reduction = 7.56 KN/m

Total D.L = (5.64 + 7.56) + 1.125

= 14.325 KN/m

Total ultimate load (Mu) = 1.5 x (14.325 + 1.47)

= 23.7 KN/m

BEAM B7 & B11

Length of smaller side of trapezium = 4.39 - 2 x (0.67)

= 3.05m^2

Area of trapezium = $\frac{3.05 + 4.39}{2}$ x 0.67 + 3.88

= 6.37 m^2

L.L/m = $\frac{6.37 \times 1.5}{4.39}$ = 2.17 KN/m

Slab load/m = $\frac{5.75 \times 6.37}{4.39}$ x 8.34 KN/m

(Note: There is no wall. Hence no calculation of main wall load)

D.L = 8.34 KN/m

Total D.L = 8.34 + 1.125 = 9.47 kn/M

Total ultimate load (Mu) = 1.5 x (2.17 + 9.47)

= 17.46 KN/m

BEAM B8 & B9

Area of two trapezium = 2.81 x 2

= 5.62 m^2

L.L/m = $\frac{5.62 \times 1.5}{3.47}$

= 2.43 KN/m

Slab load/m = $\frac{5.75 \times 5.62}{3.47}$

= 9.31 KN/m

(NOTE : NO WALLS)

Total D.L = 9.31 + 1.125

= 10.44 KN/m

Total ultimate load (Mu) = 1.5 x (10.44 + 2.43)

= 19.31 KN/m

BEAM B12 B13 & B17

Area of two trapezium = 5.62 M^2

L.L/m = $\frac{5.62 \times 1.5}{3.47}$

= 2.43 KN/m

Slab load/m = $\frac{5.75 \times 5.62}{3.47}$

= 2.43 KN/m

Load of wall after reduction = 7.56 KN/m

D.L = 9.99 KN/m

Total D.L = 9.99 + 1.125 = 11.12 KN/m

Total ultimate load (Mu) = 1.5 x (11.12 + 2.43)

= 20.33 KN/m

BEAM B14

Area of two trapeziums $= 2 \times \dfrac{3.05 + 4.39}{2}\ 0.67$

$= 4.98\ m^2$

L.L/m $= \dfrac{4.98 \times 1.5}{4.39} = 1.71$ KN/m

Slab load $= \dfrac{5.75 \times 4.98}{4.39} = 6.53$ KN/m

Load of wall after red action = 7.56 KN/m

D.L = 6.53 + 7.56

= 14.09 KN/m

Total D.L = 14.09 + 1.125

= 15.22 KN/m

Total ultimate load (Mu) = 1.5 x (15.22 + 1.71)

= 25.4 KN/m

BEAM B16

Area of two trapeziums = 3.88 x 2

$= 7.76 m^2$

L.L/m $= \dfrac{1.5 \times 7.76}{4.39}$

= 2.65KN/m

Slab load/m $= \dfrac{5.75 \times 7.76}{4.39}$

= 10.17 KN/m

Load of wall after reduction = 7.56 KN/m

D.L = 10.17 + 7.56

= 17.73 KN/m

Total D.L = 17.73 + 1.125

= 18.86 KN/m

Total ultimate load (Mu) = 1.5 x (18.86 + 10.17)

= 43.56 KN/m

BEAM B18

Area of triangle $= 1/2 \times 1.5 \times \tan 45 \times 0.75$

$= 0.5625\ m^2$

L.L/m $= \dfrac{0.5625 \times 1.5}{1.5} = 0.5625$

Slab load/m $= \dfrac{5.75 \times 0.5625}{1.5}$

$= 2.16$ KN/m

Load of wall after reduction $= 7.56$ KN/m

Total D.L. $= 9.72$KN/m $+ 1.125$ KN/m

$= 10.85$ KN/m

Total ultimate load (Mu) $= 1.5 \times (0.5625 + 10.25)$

$= 17.15$ KN/m

BEAM B24, B23 & B22

Length of smaller side $= 2.56 - 2(0.975)$

$= 0.61$

Area of trapezium $= \dfrac{0.61 + 2.56}{2} \times 0.975$

$= 1.55\ m^2$

L.L/m $= \dfrac{1.55 \times 1.5}{2.56} = 0.91$ KN/m

Slab load/m $= \dfrac{5.75 \times 1.55}{2.56}$

$= 3.48$ KN/m

Load of wall after reduction $= 7.56$ KN/m

$$\text{Total D.L.} = 11.04 + 1.125$$

$$= 12.17 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (12.27 + 0.19)$$

$$= 19.62 \text{ KN/m}$$

BEAM B27

$$\text{Total area} = 1.55 + 1/2 \times 2.56 \times (\tan 45 \times \frac{2.56}{2}$$

$$= 3.19\text{m2}$$

$$\text{L.L/m} = \frac{1.5 \times 3.19}{2.56} \times 1.8 \text{ KN/m}$$

$$\text{Slab load/m} = \frac{5.75 \times 3.19}{2.56}$$

$$= 7.17 \text{ KN/m}$$

$$\text{Load of wall after reduction} = 7.56\text{KN/m}$$

$$\text{Total D.L} = 14.73 + 1.125$$

$$= 15.86\text{KN/m}$$

$$\text{Total ultimate load (Mu)} = 1.5 \times (15.86 + 1.87)$$

$$= 26.59\text{KN/m}$$

BEAM B25

$$\text{Area of triangle} = 1/2 \times 2.56 \times \frac{2.56}{2} = 1.64$$

$$= \frac{(2.56 - 2 \times 0.975) + 2.56}{2} \times 0.975$$

$$= 1.55 \text{ m}^2$$

$$L.L/m = \frac{1.5 \times 3.19}{2.56} \times 1.87 \text{ KN/m}$$

$$\text{slab load/m} = \frac{5.75 \times 3.19}{2.56} = 7.17 \text{ KN/m}$$

Load of wall after reduction = 7.56KN/m

Total D.L = 7.17 + 7.56 = 14.73 KN/m

Total ultimate load (Mu) = 1.5 x (14.73 + 1.87)

= 24.9 KN/m

BEAM B26

Total area = 3.19 + 2 x (1/2 1.28 x .64)

= 4.Ø1m2

$$L.L/m = \frac{1.5 \times 4.01}{2.56} = 2.35 \text{ KN/m}$$

$$\text{Slab load/m} = \frac{5.75 \times 4.01}{2.56} = 9.01 \text{ KN/m}$$

Load of wall after reduction = 7.56KN/m

Total D.L = 17.7 KN/m

Total ultimate load (Mu) = 1.5 x (17.7 + 2.35)

= 3Ø.Ø75KN/m

EAM B29

Total area = $Ø.64 + 2 \times (1/2 \times 1.28 \times Ø.64)$

= $1.44\ m^2$

L.L/m = $\frac{1.5 \times 1.44}{2.56} = Ø.84$ Kn/m

Slab load/m = $\frac{5.75 \times 1.44}{2.56} = 3.23$ KN/m

Load of wall after reduction = 7.56KN/m

Total D.L = $(3.23 + 7.56) + 1.125$

= 11.92 KN/m

Total ultimate load (Mu) = $1.5 \times (11.92 + Ø.84)$

= 19.14 KN/m

BEAM B3Ø & B31

Area of triangle = $Ø.64 \times 2 = 1.28\ m^2$

L.L/m = $\frac{1.5 \times 1.28}{2.56} = Ø.75$ KN/m

Slab load/m = $\frac{5.75 \times 1.28}{2.56} = 2.875$KN/m

(NOTE : NO WALLS)

Total D.L = $2.875 + 1.125 = 4.Ø$ KN/m

Total ultimate load (Mu) = $1.5 \times (4.Ø + Ø.75)$

= 7.13 KN/m

BEAM B21

Area of triangle = $\emptyset.64 \times 2 = 1.28\ m^2$

$$L.L/m = \frac{1.5 \times 1.28}{2.56} = \emptyset.75\ KN/m$$

$$\text{Slab load/m} = \frac{5.75 \times 1.28}{2.56} = 2.875\ KN/m$$

(NOTE : NO WALLS)

Total D.L = $2.875 + 1.125 = 4.\emptyset\ KN/m$

Total ultimate load (Mu) = $1.5 \times (4.\emptyset + \emptyset.75)$

= $7.13\ KN/m$

LOAD CALCULATIONS FOR FLOOR SLABS:

TAKE MINIMUM LIVE LOAD IN FLOOR AREA AS 2.0 KN/m^2

BEAM B1

Area of triangle = 1/2 x 1.83 x (tan 45 x 1.83/2)

= 0.84 m^2

L.L. = 2.0 KN/m^2

L.L/m = $\frac{1.68}{1.83}$

= 0.92 KN/m

Slab Load = D.L of slab + wt. of weathering coarse

= 0.14 x 25 x 2.25

= 5.75 x 0.84

= 4.83 KN/m^2

Slab load/m = $\frac{4.83}{1.83}$

= 2.64 KN/m

Load of main wall after reduction = 7.56 KN/m

Total D.L = (2.64 + 7.52) + 1.125

= 11.325 KN/m

Total ultimate load (Mu) = 1.5 x (11.325 + 0.92)

= 18.38 KN/m

BEAM B5

L.L. = $2 \times 3.28 = 6.56$ KN/m^2

L.L/m = $\dfrac{6.56}{2.44} = 2.69$ KN/m

Total D.L = 16.41 KN/m

Total ultimate load (Mu) = $1.5 \times (16.41 + 2.69)$

= 28.65 KN/m

BEAM B2

L.L/m = $\dfrac{2.0 \times 3.88}{4.27}$

= 1.82 KN/m

Total D.L. = 10.035 KN/m

Total ultimate load(Mu) = $1.5 \times (10.035 + 1.82)$

= 17.79 KN/m

BEAM B3 & B4

L.L/m = $\dfrac{2.0 \times 2.81}{3.47}$

= 1.62 KN/m

Total D.L. = 10.36 KN/m

Total ultimate load(Mu) = $1.5 \times (10.36 + 1.62)$

= 17.97 KN/m

BEAM B6, B10 & B15

$$L.L/m = \frac{2.0 \times 1.91}{1.95}$$

$$= 1.96 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (14.325 + 1.96)$$

$$= 24.44 \text{ KN/m}$$

BEAM B7 & B11

$$L.L/m = \frac{2.0 \times 6.37}{4.39}$$

$$= 2.91 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (2.91 + 9.47)$$

$$= 18.57 \text{ KN/m}$$

BEAM B12, B13 & B17

$$L.L/m = \frac{2.0 \times 5.62}{3.47}$$

$$= 3.24 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (11.12 + 3.24)$$

$$= 21.54 \text{ KN/m}$$

BEAM B14

$$L.L/m = \frac{2.0 \times 4.98}{4.39}$$

$$= 2.27 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (15.22 + 2.27)$$

$$= 26.24 \text{ KN/m}$$

BEAM B26

$$L.L/m = \frac{2.0 \times 4.01}{2.56}$$

$= 3.14$ KN/m

Total ultimate load(Mu) $= 1.5 \times (17.7 + 3.14)$

$= 31.26$ KN/m

BEAM B25

$$L.L/m = \frac{2.0 \times 3.19}{2.56}$$

$= 2.50$ KN/m

Total ultimate load(Mu) $= 1.5 \times (14.73 + 2.50)$

$= 25.85$ KN/m

BEAM B28

$$L.L/m = \frac{2.0 \times 1.28}{2.56}$$

$= 1.0$ KN/m

Total ultimate load(Mu) $= 1.5 \times (11.57 + 1.0)$

$= 18.86$ KN/m

BEAM B32, B33 & B34

$$L.L/m = \frac{2.0 \times 0.64}{2.56}$$

$= 0.5$ KN/m

Total ultimate load(Mu) $= 1.5 \times (10.44 + 0.5)$

$= 16.41$ KN/m

BEAM B21

$$L.L/m = \frac{2.0 \times 1.28}{2.56}$$

$$= 1.0 \text{ KN/m}$$

$$\text{Total ultimate load(Mu)} = 1.5 \times (4.0 + 1.0)$$

$$= 7.50 \text{ KN/m}$$

[3] A N A L Y S I S O F S T R U C T U R E S

The analysis of frames has been done using **DIRECT STIFFNESS METHOD.** The analysis of frames is computerised and the programme is done in **'FORTRAN 77'**. The bending moments diagrams are shown in the structural drawing part.

```
**************************************************************************
   PROGRAM FOR ANALYSIS OF PLANE FRAMES USING DIRECT
   STIFFNESS METHOD
**************************************************************************
    DIMENSION X(75),Y(75),S(75,75),AML(75,6),F(75),IP(75),IQ(75
    DIMENSION A(50),AI(50),U(75),BK(6,6),P(50),D(100)
    OPEN(1,FILE='NMRA.IN')
    OPEN(2,FILE='MEA.OUT')
    READ(1,*)NN,NE,NBC,E
    WRITE(2,410)
0   FORMAT(5X,60('*'),//,15X,'WELCOME TO THE PLANE FRAME ANALYSIS',
  * //,5X,60('*'))
    WRITE(2,411)NN,NE,NBC,E
1   FORMAT(//,5X,'NUMBER OF NODES =',I2,//,5X,'NUMBER OF ELEMENTS
  * =',I2,//,5X,'NUMBER OF BOUNDARY CONDITIONS =',I2,//,5X,
  *'ELASTIC MODULUS =',E10.4//)
    WRITE(2,412)
.2  FORMAT(5X,'NODE NUMBER',15X,'X COORDINATE',12X,'Y COORDINATE')
    DO 10 I=1,NN
    READ(1,*)J,X(J),Y(J)
)   WRITE(2,413)J,X(J),Y(J)
13  FORMAT(/,9X,I2,20X,F10.4,14X,F10.4//)
    WRITE(2,414)
14  FORMAT(/,1X,'MEMBER',3X,'BACK',7X,'FORE',7X,'AREA',6X,'MOMENT OF
  * INERTIA',6X,'LENGTH'/,1X,'NUMBER',3X,'NODE',7X,'NODE'//)
    DO 15 I=1,NE
    READ(1,*)J,IP(J),IQ(J),A(J),AI(J)
    N=NN*3
    DO 20 I=1,N
.0  F(I)=0.0
    DO 24 I=1,NE
    DO 24 J=1,6
24  AML(I,J)=0.0
    READ(1,*)NJL
    IF(NJL.EQ.0)GO TO 12
    DO 25 I=1,NJL
    READ(1,*)LJN
    N1=LJN*3-2
    N2=N1+1
    N3=N2+1
25  READ(1,*)F(N1),F(N2),F(N3)
12  READ(1,*)NML
    IF(NML.EQ.0)GOTO 34
    DO 30I=1,NML
30  READ(1,*)J,(AML(J,MN),MN=1,6)
34  DO 35 I=1,N
    DO 35 J=1,N
35  S(I,J)=0.0
C   GENERATION OF STIFFNESS MATRIX
    DO 40 M=1,NE
    NB=IP(M)
    NF=IQ(M)
    H=X(NF)-X(NB)
    V=Y(NF)-Y(NB)
    AL=SQRT(H*H+V*V)
    CA=H/AL
    SA=V/AL
    C2=CA**2
```

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```
      T1=E*A(M)/AL
      T2=12.0*E*AI(M)/AL**3
      T3=4.0*E*AI(M)/AL
      T4=6.0*E*AI(M)/AL**2
      IB=3*NB-2
      IF=3*NF
      WRITE(2,344)M,IP(M),IQ(M),A(M),AI(M),AL
4     FORMAT(3X,I2,5X,I2,9X,I2,5X,F10.5,5X,F13.4,10X,F10.5,. )
      S(IB,IB)=S(IB,IB)+(C2*T1+S2*T2)
      S(IB,IB+1)=S(IB,IB+1)+CS*T1-CS*T2
      S(IB,IB+2)=S(IB,IB+2)+SA*T4
      S(IB,IF-2)=S(IB,IF-2)-C2*T1-S2*T2
      S(IB,IF-1)=S(IB,IF-1)-CS*T1+CS*T2
      S(IB,IF)=S(IB,IF)+SA*T4
      S(IB+1,IB+1)=S(IB+1,IB+1)+S2*T1+C2*T2
      S(IB+1,IB+2)=S(IB+1,IB+2)-CA*T4
      S(IB+1,IF-2)=S(IB+1,IF-2)-CS*T1+CS*T2
      S(IB+1,IF-1)=S(IB+1,IF-1)-S2*T1-C2*T2
      S(IB+1,IF)=S(IB+1,IF)-CA*T4
      S(IB+2,IB+2)=S(IB+2,IB+2)+T3
      S(IB+2,IF-2)=S(IB+2,IF-2)-SA*T4
      S(IB+2,IF-1)=S(IB+2,IF-1)+CA*T4
      S(IB+2,IF)=S(IB+2,IF)+0.5*T3
      S(IF-2,IF-2)=S(IF-2,IF-2)+C2*T1+S2*T2
      S(IF-2,IF-1)=S(IF-2,IF-1)+CS*T1-CS*T2
      S(IF-2,IF)=S(IF-2,IF)-SA*T4
      S(IF-1,IF-1)=S(IF-1,IF-1)+S2*T1+C2*T2
      S(IF-1,IF)=S(IF-1,IF)+CA*T4
      S(IF,IF)=S(IF,IF)+T3
      S(IB+1,IB)=S(IB,IB+1)
      S(IB+2,IB)=S(IB,IB+2)
      S(IF-2,IB)=S(IB,IF-2)
      S(IF-1,IB)=S(IB,IF-1)
      S(IF,IB)=S(IB,IF)
      S(IB+2,IB+1)=S(IB+1,IB+2)
      S(IF-2,IB+1)=S(IB+1,IF-2)
      S(IF-1,IB+1)=S(IB+1,IF-1)
      S(IF,IB+1)=S(IB+1,IF)
      S(IF-2,IB+2)=S(IB+2,IF-2)
      S(IF-1,IB+2)=S(IB+2,IF-1)
      S(IF,IB+2)=S(IB+2,IF)
      S(IF-1,IF-2)=S(IF-2,IF-1)
      S(IF,IF-2)=S(IF-2,IF)
      S(IF,IF-1)=S(IF-1,IF)
      F(IB)=F(IB)+(CA*AML(M,1)-SA*AML(M,2))
      F(IB+1)=F(IB+1)+CA*AML(M,2)+SA*AML(M,1)
      F(IB+2)=F(IB+2)+AML(M,3)
      F(IF-2)=F(IF-2)+CA*AML(M,4)-SA*AML(M,5)
      F(IF-1)=F(IF-1)+SA*AML(M,4)+CA*AML(M,5)
      F(IF)=F(IF)+AML(M,6)
40    CONTINUE
C     APPLICATION OF BOUNDARY CONDITIONS
      DO 60 I=1,NBC
      READ(1,*)JB,SB
      S(JB,JB)=1.0
      F(JB)=SB
      DO 70 J=1,N
```

```
      CONTINUE
      CONTINUE
      CALL GAUSS(S,F,U,N)

      WRITE(2,201)
1     FORMAT(//,5X,'NODAL DISPLACEMENTS IN GLOBAL COORDINATE SYSTEM',/)
      WRITE(2,347)
7     FORMAT(/,5X,'NODE',7X,'HORIZONTAL',7X,'VERTICAL',7X,'ROTATION',
     */,16X,'DEFLECTION',7X,'DEFLECTION',5X,'(IN RADIANS)',/)
      DO 90 I=1,NN
      WRITE(2,202)I,U(I*3-2),U(I*3-1),U(I*3)
2     FORMAT(5X,I2,8X,F10.6,6X,F10.6,5X,F10.6,/)
      WRITE(2,234)
4     FORMAT(//,2X,'MEMBER FORCES IN LOCAL COORDINATE SYSTEM',/)
      WRITE(2,*)' MEMBER   NODE    AXIAL FORCE      SHEARFORCE         BENDING'
                                                                        MOMENT
      WRITE(2,*)'
      CALCULATION OF MEMBER END FORCES
      DO 100 I=1,NE
      NB=IP(I)
      NF=IQ(I)
      H=X(NF)-X(NB)
      V=Y(NF)-Y(NB)
      AL=SQRT(H*H+V*V)
      CA=H/AL
      SA=V/AL
      C2=CA*CA
      S2=SA*SA
      CS=CA*SA
      T1=E*A(I)/AL
      T2=12.0*E*AI(I)/AL**3
      T3=4.0*E*AI(I)/AL
      T4=6.0*E*AI(I)/AL**2
      BK(1,1)=T1
      BK(2,1)=0.0
      BK(3,1)=0.0
      BK(4,1)=-T1
      BK(5,1)=0.0
      BK(6,1)=0.0
      BK(1,2)=0.0
      BK(2,2)=T2
      BK(3,2)=-T4
      BK(4,2)=0.0
      BK(5,2)=-T2
      BK(6,2)=-T4
      BK(1,3)=0.0
      BK(2,3)=-T4
      BK(3,3)=T3
      BK(4,3)=0.0
      BK(5,3)=T4
      BK(6,3)=0.5*T3
      DO 110 II=1,6
      BK(II,4)=-BK(II,1)
      BK(II,5)=-BK(II,2)
110   BK(II,6)=BK(II,3)
      BK(3,6)=0.5*T3
      BK(6,6)=T3
      IB=3*NB-2
```

```
      D(3)=U(IB+2)
      D(4)=CA*U(IF-2)+SA*U(IF-1)
      D(5)=-SA*U(IF-2)+CA*U(IF-1)
      D(6)=U(IF)
      DO 120 L=1,6
      P(L)=0.0
      DO 120 J=1,6
0     P(L)=P(L)+BK(L,J)*D(J)
      DO 130 L=1,6
0     P(L)=P(L)-AML(I,L)
      WRITE(2,206)I,NB,P(1),P(2),P(3)
      WRITE(2,207)NF,P(4),P(5),P(6)
07    FORMAT(9X,I3,3(4X,F14.4),/)
06    FORMAT(1X,I3,5X,I3,3(4X,F14.4),/)
00    CONTINUE
      STOP
      END
      GAUSSIAN ELIMINATION
      SUBROUTINE GAUSS(A,C,X,N)
      DIMENSION A(75,75),C(75),X(75)
      DO 10 I=1,N-1
      DO 10 J=I+1,N
      D=A(J,I)/A(I,I)
      DO 30 K=1,N
30    A(J,K)=A(J,K)-A(I,K)*D
      C(J)=C(J)-C(I)*D
10    CONTINUE
      DO 40 I=N,1,-1
      TOT=0.0
      DO 50 J=I+1,N
50    TOT=TOT+A(I,J)*X(J)
      X(I)=(C(I)-TOT)/A(I,I)
      TOT=0
40    CONTINUE
      RETURN
      END
```

INPUT DATA FOR THE ANALYSIS OF PLANE FRAMES USING DIRECT STIFFNESS - METHOD

1. Enter the no of nodes, no of elements, no.of boundary conditions and youngs modulus of elasticity.

2. Enter the mode number, x-Co. ordinate and Y-Co. ordinate.

3. Enter the member number back-mode, fore-node, area and moment of inertia.

4. Enter the no.of joint loads.

5. Enter the joint number

6. Enter the horizontal load, vertical load and moment

7. Enter the no. of number loads

8. Enter the member number

9. Enter the horizontal load, vertical load and moment

10. Enter the boundary conditions.

T DATA
****
E No:4 ROOF SLAB
****
) 15 1
3.5
.66 3.5
.93 3.5
1.28 3.5
2.78 3.5
2.78 0
1.28 0
.93 0
.66 0
0 0
2 1 1
2 3 1 1
3 4 1 1
4 5 1 1
5 6 1 1
4 7 1 1
3 8 1 1
2 9 1 1
1 10 1 1

-43.37 26.46 0 -43.37 -26.46

-93 66.19 0 -93 -66.19

-27.94 15.6 0 -27.94 -15.6

-12.21 3.05 0 -12.21 -3.05
6 0
7 0
8 0
9 0
20 0
21 0
22 0
23 0
24 0
25 0
26 0
27 0
28 0
29 0
30 0

FRAME No:4 ROOF SLAB
****

****

WELCOME TO THE PLANE FRAME ANALYSIS

****

NUMBER OF NODES =10

NUMBER OF ELEMENTS = 9

NUMBER OF BOUNDARY CONDITIONS =15

ELASTIC MODULUS =0.1000E+01

NODE NUMBER	X COORDINATE	Y COORDINATE
1	0.0000	3.5000
2	3.6600	3.5000
3	7.9300	3.5000
4	11.2800	3.5000
5	12.7800	3.5000
6	12.7800	0.0000
7	11.2800	0.0000
8	7.9300	0.0000
9	3.6600	0.0000
10	0.0000	0.0000

MEMBER NUMBER	BACK NODE	FORE NODE	AREA	MOMENT OF INERTIA	LENGTH
1	1	2	1.00000	1.0000	3.66
2	2	3	1.00000	1.0000	4.27
				1.0000	3.35

5	5	6	1.00000	1.0000	1.50000
6	4	7	1.00000	1.0000	1.50000
7	3	8	1.00000	1.0000	1.50000
8	2	9	1.00000	1.0000	1.50000
9	1	10	1.00000	1.0000	3.[illegible]

EMBER FORCES IN LOCAL COORDINATE SYSTEM

EMBER	NODE	AXIAL FORCE	SHEARFORCE	BENDING MOMENT
1	1	10.9351	53.1747	-38.6391
	2	-10.9351	33.5653	2.7538
2	2	22.7397	91.4058	-36.8876
	3	-22.7397	94.5942	43.6949
3	3	7.5551	17.5047	6.9987
	4	-7.5551	38.3753	33.9594
4	4	0.8989	5.0901	-5.0318
	5	-0.8989	19.3299	15.7118
5	5	19.3299	0.8989	-15.7127
	6	-19.3299	-0.8989	11.5666
6	4	43.4653	6.6563	-20.9265
	7	-43.4653	-6.6563	5.6296
7	3	112.0990	15.1845	-44.6936
	8	-112.0990	-15.1845	-8.4523
8	2	124.9710	-11.8046	34.1336
	9	-124.9710	11.8046	7.1823
9	1	53.1748	-10.9351	18.6392
	10	-53.1748	10.9351	-0.3665

```
T DATA
*******
E No:4 FLOOR SLAB
******************

14 30 1
 3.5
.66 3.5
.93 3.5
1.28 3.5
2.78 3.5
2.78 0
1.28 0
.93 0
.66 0
 0 0
 0 7
 3.66 7
 7.93 7
 11.28 7
 12.78 7
 1 2 1 1
 2 3 1 1
 3 4 1 1
 4 5 1 1
 5 6 1 1
 4 7 1 1
 3 8 1 1
 2 9 1 1
 1 10 1 1
0 1 11 1 1
1 2 12 1 1
12 3 13 1 1
13 4 14 1 1
14 5 15 1 1
0
4
1
0 -44.73 27.28 0 -44.73 -27.28
2
0 -71.74 51.05 0 -71.74 -51.05
3
0 -27.94 15.6 0 -27.94 -15.6
4
0 -10.94 2.73 0 -10.94 -2.73
16 0
17 0
18 0
19 0
20 0
21 0
22 0
23 0
24 0
```

0
0
0
0
0
0
0
0
0
0
0
0

FRAME No:4 FLOOR SLAB
****

****

WELCOME TO THE PLANE FRAME ANALYSIS

****

NUMBER OF NODES =15

NUMBER OF ELEMENTS =14

NUMBER OF BOUNDARY CONDITIONS =30

ELASTIC MODULUS =0.1000E+01

NODE NUMBER	X COORDINATE	Y COORDINATE
1	0.0000	3.5000
2	3.6600	3.5000
3	7.9300	3.5000
4	11.2800	3.5000
5	12.7800	3.5000
6	12.7800	0.0000
7	11.2800	0.0000
8	7.9300	0.0000
9	3.6600	0.0000
10	0.0000	0.0000
11	0.0000	7.0000
12	3.6600	7.0000

14	11.2800	7.0000
15	12.7800	7.0000

ER ER	BACK NODE	FORE NODE	AREA	MOMENT OF INERTIA	LENGTH
	1	2	1.00000	1.0000	[illegible].66013
2	2	3	1.00000	1.0000	4.27000
3	3	4	1.00000	1.0000	1.35000
4	4	5	1.00000	1.0000	1.50000
5	5	6	1.00000	1.0000	3.50000
6	4	7	1.00000	1.0000	3.50000
7	3	8	1.00000	1.0000	3.50000
8	2	9	1.00000	1.0000	3.50000
9	1	10	1.00000	1.0000	3.50000
10	1	11	1.00000	1.0000	3.50000
11	2	12	1.00000	1.0000	3.50000
12	3	13	1.00000	1.0000	3.50000
13	4	14	1.00000	1.0000	3.50000
14	5	15	1.00000	1.0000	[illegible].50000

MEMBER FORCES IN LOCAL COORDINATE SYSTEM

MEMBER	NODE	AXIAL FORCE	SHEARFORCE	BENDING MOMENT
1	1	0.0000	53.3770	-41.6678
	2	0.0000	36.0830	10.0197
2	2	0.0000	70.1111	-39.6725
	3	0.0000	73.3689	40.6279
3	3	0.0000	17.2875	[illegible].8212
	4	0.0000	38.5925	34.8646
4	4	0.0000	-2.1869	[illegible].6985
			[illegible]	[illegible]7.8916

	6	-12.0334	-3.8554	-4.4979
	4	18.2028	7.8350	-18.2816
	7	-18.2028	-7.8350	-9.1408
7	3	45.3282	10.1677	-23.7246
	8	-45.3282	-10.1677	-11.8623
8	2	53.0970	-6.3542	14.8264
	9	-53.0970	6.3542	7.4132
9	1	26.6885	-8.9288	20.8339
	10	-26.6885	8.9288	10.4170
10	1	-26.6885	-8.9288	20.8339
	11	26.6885	8.9288	10.4170
11	2	-53.0970	-6.3542	14.8264
	12	53.0970	6.3542	7.4132
12	3	-45.3282	10.1677	-23.7246
	13	45.3282	-10.1677	-11.8623
13	4	-18.2028	7.8350	-18.2816
	14	18.2028	-7.8350	-9.1408
14	5	-12.0334	3.8554	-6.9959
	15	12.0334	-3.8554	-4.4979

[4] DESIGN OF BEAMS

With the exception of pre cast system reinforced concrete floors, roofs, decks etc., are always monolithic. Beam stirrups and bent bars extend up into the slab.

Hence a part of the slab will act with the upper part of the beam to resist longitudinal compression. The resulting beam cross section is T-shaped rather than rectangular. The slab forms the beam flange, while the part of the beam projecting below the slab forms what is called the web or stem.

In the present design of reinforced concrete structures, provisions is made to resist hypothetical overloads. The required strength is found by applying load factors r, greater than unity.

```
************************************************************************
    RECTANGULAR BEAM DESIGN
************************************************************************

     ASSUME THE SIZE OF THE BEAM AS 300mmx450mm
     INTEGER DIAM ,DS
     REAL M,L,W
     WRITE(5,*)'ENTER THE MOMENT'
     WRITE(5,*)'ENTER THE VALUE OF Fck'
     WRITE(5,*)'ENTER THE VALUE OF Fy'
     WRITE(5,*)'ENTER THE DIA OF THE BAR'
     WRITE(5,*)'ENTER THE LENGTH AND UNIFORMLY DIST_LOAD'
     WRITE(5,*)'ENTER THE DIA OF STIRRUP'
     OPEN(5,FILE='RB.IN')
     OPEN(6,FILE='RB.OUT')
     READ(5,*)M,Fck,Fy,DIAM,L,W,DS
     B=300.
     DE=500.
     C=50.
     D=DE-C
     IF(FY.EQ.250)XUMAXBYD=0.53
     IF(FY.EQ.415)XUMAXBYD=0.48
     IF(FY.EQ.500)XUMAXBYD=0.46
     MULIM=0.36*XUMAXBYD*(1-0.42*XUMAXBYD)*B*D*D*FCK
     DO1000I=1,20
000  WRITE(*,*)
     IF(M.LT.MULIM)N=1
     IF(M.EQ.MULIM)N=2
     IF(M.GT.MULIM)N=3

     IF(N.EQ.1)THEN
C    SINGLY REINFORCED SECTION
     BFCK=B*FCK
     XYZ=0.87*FY
     ANM=SQRT((XYZ*D)**2-4*M*(XYZ*FY/BFCK))
     AS1T=(-XYZ*D+ANM)/(-2*XYZ*FY/BFCK)
     ENDIF

     IF(N.EQ.2)THEN
     AS2T=XUMAXBYD/(0.87*FY/0.36/FCK/B/D)
     ENDIF

     IF(N.EQ.3)THEN
C    DOUBLY REINFORCED SECTION
     MU2=M-MULIM
     XUMAX=XUMAXBYD*D
     STRAIN=(XUMAX-C)*0.0035/XUMAX
     IF (FY.EQ.250)THEN
     IF (STRAIN .LE. 0.0125)FSC=((250.0/0.0125)*STRAIN)/1.15
     IF(STRAIN .GT. 0.0125 )FSC=250/1.15
     ENDIF
     IF (FY.EQ.415)THEN
     IF (STRAIN .LE. 0.002075)FSC=((415.0/0.002075)*STRAIN)/1.15
     IF(STRAIN .GT. 0.002075)FSC=415/1.15
     ENDIF
     IF (FY.EQ.500)THEN
     IF (STRAIN .LE. 0.0025)FSC=((500.0/0.0025)*STRAIN)/1.15
     IF (STRAIN .GT. 0.0025 )FSC=500/1.15
```

```
      BFCK=B*FCK
      XYZ=0.87*FY
      ANM=SQRT((XYZ*D)**2-4*MULIM*(XYZ*FY/BFCK))
      AS3T1=(-XYZ*D+ANM)/(-2*XYZ*FY/BFCK)
      AS3T=AS3T1+AS3T2
      ENDIF

      SUMMARY
      WRITE(6,10002)
02    FORMAT(75('*'))
      WRITE(6,*)'BEAM No.                 SUMMARY OF THE DESIGN'
      WRITE(6,*)'
      WRITE(6,*)'THE MOMENT IS:',M ,'N.mm'
      WRITE(6,*)'THE LENGTH OF BEAM IS:',L ,'mm'
      WRITE(6,*)'THE UNIFORMLY DISTRIBUTED LOAD:',W,'N/mm'
      WRITE(6,10003)
      IF(N.EQ.1)THEN
      WRITE(6,*)'               SINGLY REINFORCED SECTION'
      WRITE(6,*)'THE REINFORCEMENT IS:',AS1T,'mm²'
      NOBARS=AS1T/(3.141592654*((DIAM/2)**2))
      NOBARS=NINT(NOBARS+0.5)
      IF(NOBARS.LT.2.0)THEN
      NOBARS=NOBARS+2.0
      ENDIF
      WRITE(6,10004)NOBARS,DIAM
0004  FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR')
      ENDIF

      IF(N.EQ.2)THEN
      WRITE(6,*)'               BALANCED REINFORCED SECTION'
      WRITE(6,*)'THE REINFORCEMENT IS:',AS2T,'mm²'
      NOBARS=AS2T/(3.141592654*((DIAM/2)**2))
      NOBARS=NINT(NOBARS+0.5)
      IF(NOBARS.LT.2.0)THEN
      NOBARS=NOBARS+2.0
      ENDIF
      WRITE(6,10005)NOBARS,DIAM
10005 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR')
      ENDIF

      IF(N.EQ.3)THEN
      WRITE(6,*)'               DOUBLY REINFORCED SECTION'
      WRITE(6,*)'THE TOTAL REINFORCEMENT IS:',AS3T,'mm²'
      WRITE(6,*)'THE TENSION REINFORCEMENT IS:',AS3T1,'mm²'
      WRITE(6,*)'THE TENSION REINFORCEMENT IS:',AS3T2,'mm²'
      WRITE(6,*)'THE TENSION REINFORCEMENT IS:',AS3T2,'mm²'
      NOBARS1=AS3T1/(3.141592654*((DIAM/2)**2))
      NOBARS1=NINT(NOBARS1+0.5)
      IF(NOBARS1.LT.2.0)THEN
      NOBARS1=NOBARS1+2.0
      ENDIF
      NOBARS2=AS3T2/(3.141592654*((DIAM/2)**2))
      NOBARS2=NINT(NOBARS2+0.5)
      IF(NOBARS2.LT.2.0)THEN
      NOBARS2=NOBARS2+2.0
      ENDIF
      WRITE(6,10006)NOBARS1,DIAM
10006 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR AS
     *     REINFORCEMENT')
```

```
      WRITE(6,10003)
3     FORMAT(75('*'))
      ENDIF
C     CHECK FOR SHEAR
      IF(N.EQ.1)AST=AS1T
      IF(N.EQ.2)AST=AS2T
      IF(N.EQ.3)AST=AS3T
      SF=W*L/2
      TOWVEE=SF/B/D
      PERSTEEL=AST*100/B/D
      IF(FCK.EQ.15)THEN
      IF(PERSTEEL.LE.0.25)P=0.35
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     * P=0.35+(0.11/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     * P=0.46+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     * P=0.54+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     * P=0.6+(0.04/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     * P=0.64+(0.04/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LT.1.75))
     * P=0.68+(0.03/0.25)*(PERSTEEL-1.5)
      IF(PERSTEEL.GE.1.75)P=0.71
      ENDIF
C     PERCENTAGE OF STEEL FOR M20 MIX
      IF(FCK.EQ.20)THEN
      IF(PERSTEEL.LE.0.25)P=0.36
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     * P=0.36+(0.12/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     * P=0.48+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     * P=0.56+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     * P=0.62+(0.05/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     * P=0.67+(0.05/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LE.1.75))
     * P=0.72+(0.03/0.25)*(PERSTEEL-1.5)
      IF((PERSTEEL.GT.1.75).AND.(PERSTEEL.LE.2.0))
     * P=0.75+(0.04/0.25)*(PERSTEEL-1.75)
      IF((PERSTEEL.GT.2.0).AND.(PERSTEEL.LE.2.25))
     * P=0.79+(0.02/0.25)*(PERSTEEL-2.0)
      IF((PERSTEEL.GT.2.25).AND.(PERSTEEL.LT.2.5))
     * P=0.81+(0.01/0.25)*(PERSTEEL-2.25)
      IF(PERSTEEL.GE.2.50)P=0.82
      ENDIF
      IF(P.LE.TOWVEE)THEN
      WRITE(6,*)'THE SHEAR REINFORCEMENT IS PROVIDED'
      Vc=P*B*D
      Vs=SF-Vc
      ASV=3.14*DS**2/4
      SPACING=0.87*FY*2*ASV*D/Vs/100
      SP=NINT(SPACING-10)
      WRITE(6,*)'PROVIDE 2 LEGGED',DS,'mm STIRRUP @',SP,'mm C/C'
```

```
STOP
END
```

MAXIMUM BENDING MOMENT FOR THE BEAMS

FRAME NO	LENGTH(l)	Uniformly load (W) Distributed (N/mm)		Bending moment (N-mm)	
		Roof	Floor	Roof	Floor
1.	4270	15.05	17.79	22.34x10	25.84x10
2.	4270	24.31	25.52	23.06x10	28.94x10
3.	4270	23.7	24.44	16.16x10	18.51x10
4.	4270	28.29	33.6	34.43x10	18.51x10
5.	2440	19.62	20.09	7.6x10	11.31x10
6.	2440	30.075	31.26	17.34x10	22.89x10
7.	2440	32.63	33.65	14.86x10	15.4x10
8.	2440	7.13	7.5	2.87x10	4.15x10
9.	2440	16.23	16.41	6.17x10	9.32x10

Here the moment acting on the water tank is also considered in the design.

Bending moment of the tank = 5.00×10^{6} N-mm

INPUT DATA FOR THE DESIGN OF RECTANGULAR BEAM

Enter the value of moments FcK, FY- diameter of the bar, length and uniformly distributed load, diameter of the stirrup.

T DATA
****
NO: 1 ROOF SLAB
****

0000 15 415 20 4270 15.05 8

TPUT DATA
****

AM No: 1 ROOF SLAB
****

****
SUMMARY OF THE DESIGN

HE MOMENT IS: 0.2234000E+008N.mm
HE LENGTH OF BEAM IS: 4270.0000000mm
HE UNIFORMLY DISTRIBUTED LOAD: 15.0500000N/mm
****
SINGLY REINFORCED SECTION

THE REINFORCEMENT IS: 141.6098000mm^2
PROVIDE 3 NOS OF 20 mm DIA BAR
THE SECTION IS SAFE AGAINST SHEAR
****

UT DATA
****

AM NO:1 FLOOR SLAB
****

840000 15 415 20 4270 17.79 8

UTPUT DATA
****

BEAM NO: 1 FLOOR SLAB
****

****
SUMMARY OF THE DESIGN

THE MOMENT IS: 0.2584000E+008N.mm

THE LENGTH OF BEAM IS: 4270.0000000mm

THE UNIFORMLY DISTRIBUTED LOAD: 17.7900000N/mm

****
SINGLY REINFORCED SECTION

THE REINFORCEMENT IS: 164.5943000mm²

PROVIDE 3 NOS OF 20 mm DIA BAR

THE SECTION IS SAFE AGAINST SHEAR

****

****
****
```
    DESIGN OF T-BEAM
**********************************************************************************
      COMMON FCK,BW,BF,EFFD,MDU,DF,N,XU1,XU2,XU3,L,W
      INTEGER DB,DBc
      OPEN(5,FILE='BEAM1.IN')
      OPEN(8,FILE= 'BEAM1.OUT')
      READ(5,*)FCK,FY,BW,BF,DF,EFFD,EFFCO,MDU,W,L
      IF(FY.EQ.250)XUMAX=0.53*EFFD
      IF(FY.EQ.415)THEN
      XUMAX=0.48*EFFD
      ELSE
      XUMAX=0.46*EFFD
      ENDIF
      MUMAX=0.36*FCK*XUMAX*BW*(EFFD-0.42*XUMAX)+0.45*FCK*(BF-BW)*
     1 DF*(EFFD-DF/2)
      IF(MDU.LE.MUMAX)GOTO 20
      IF(MDU.GT.MUMAX)GOTO 10
10    WRITE(8,*)'DOUBLY REINFORCED SECTION'
      MU1=MUMAX
      WRITE(8,*)MU1,MUMAX,MDU
      MU2=MDU-MU1
      AST1=((0.36*FCK*XUMAX*BW)+(0.45*FCK*(BF-BW)*DF))/(0.87*FY)
      STRAIN=(XUMAX-EFFCO)*0.0035/XUMAX
      IF (FY.EQ.250)THEN
      IF (STRAIN .LE. 0.0125)FSC=((250.0/0.0125)*STRAIN)/1.15
      IF(STRAIN .GT. 0.0125 )FSC=250/1.15
      ENDIF
      IF (FY.EQ.415)THEN
      IF (STRAIN .LE. 0.002075)FSC=((415.0/0.002075)*STRAIN)/1.15
      IF(STRAIN .GT. 0.002075)FSC=415/1.15
      ENDIF
      IF (FY.EQ.500)THEN
      IF (STRAIN .LE. 0.0025)FSC=((500.0/0.0025)*STRAIN)/1.15
      IF(STRAIN .GT. 0.0025 )FSC=500/1.15
      ENDIF
      ASC=MU2/(FSC*(EFFD-EFFCO))
      AST2=(ASC*Fsc)/(0.87*FY)
      AST=AST1+AST2
C     WRITE(8,*)'DIAMETER OF BAR TO BE USED AS TENSILE REINFORCEMEN
      READ(5,*)DB
      AREA=3.14*(DB**2)/4.0
      ANB=AST/AREA
      NB=NINT(ANB)
      IF(NB.LT.2.0)THEN
      NB=NB+2.0
      ENDIF
C     WRITE(8,*)'ENTER DIAMETER OF COMPRESSION BAR'
      READ(5,*)DBc
      AREA 1=3.14*(DBC**2)/4.0
      ANCB=ASC/AREA 1
      NCB=NINT(ANCB)
      IF(NCB.LT.2.0)THEN
      NCB=NCB+2.0
      ENDIF
      WRITE(8,*)'SUMMARY OF DESIGN'
      WRITE(8,*)'THE TOTAL TENSILE REINFORCEMENT:',       AST,'mm
      WRITE(8,*)'THE TOTAL COMPRESSION REINFORCEMENT:',   ASC,'mm
      WRITE(8,*)'THE TOTAL ... REINFORCEMENT:',           DB, 'mm'
```

```
      WRITE(8,*)'NO OF COMPRESSION BAR:',
      STOP
      WRITE(8,*)'SINGLY REINFORCED SECTION DESIGN'
      XU=DF
      MU1=0.36*FCK*XU*BF*(EFFD-0.42*XU)
      IF(MDU.LT.MU1)THEN
      N=1
      CALL QUAD
      AST=(0.36*FCK*XU1*BF)/(0.87*FY)
      WRITE(8,*)'DIAMETER OF BAR TO BE USED AS TENSILE REINFORCEMENT'
      READ(5,*)DB
      AREA=3.14*(DB**2)/4.0
      ANB=AST/AREA
      NB=NINT(ANB)
      IF(NB.LT.2.0)THEN
      NB=NB+2.0
      ENDIF
      ELSE
      XU=(7./3.0)*DF
      MU2=0.36*FCK*XU*BW*(EFFD-0.42*XU)+0.45*FCK*(BF-BW)*
     1 DF*(EFFD-DF*0.5)
      IF(MU2.GT.MDU)THEN
      N=2
      CALL QUAD
      YF=0.15*XU2+0.65*DF
      AST=((0.36*FCK*XU2*BW)+(0.45*FCK*YF*(BF-BW)))/(0.87*FY)
      WRITE(8,*)'DIAMETER OF BAR TO BE USED AS TENSILE REINFORCEMENT'
C     READ(5,*)DB
      AREA=3.14*(DB**2)/4.0
      ANB=AST/AREA
      NB=NINT(ANB)
      IF(NB.LT.2.0)THEN
      NB=NB+2.0
      ENDIF
      ELSE
      N=3
      CALL QUAD
      AST=((0.36*FCK*XU3*BW)+(0.45*FCK*DF*(BF-BW)))/(0.87*FY)
      WRITE(8,*)'DIAMETER OF BAR TO BE USED AS TENSILE REINFORCEMEN
C     READ(5,*)DB
      AREA=3.14*(DB**2)/4.0
      ANB=AST/AREA
      NB=NINT(ANB)
      IF(NB.LT.2.0)THEN
      NB=NB+2.0
      ENDIF
      ENDIF
      ENDIF
      WRITE(8,*)'SUMMARY OF DESIGN'
      WRITE(8,*)'THE TOTAL TENSILE REINFORCEMENT:',      AST,'mm2'
      WRITE(8,*)'DIAMETER OF TENSILE REINFORCEMENT:',    DB,  'mm'
      WRITE(8,*)'NO OF TENSILE REINFORCEMENT BAR:',      NB

C     CHECK FOR SHEAR
C     IF(N.EQ.1)AST=AS1T
C     IF(N.EQ.2)AST=AS2T
      IF(N.EQ.3)AST=AS3T
```

```
      IF(PERSTEEL.LE.0.25)P=0.35
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     *   P=0.35+(0.11/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     *   P=0.46+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     *   P=0.54+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     *   P=0.6+(0.04/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     *   P=0.64+(0.04/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LT.1.75))
     *   P=0.68+(0.03/0.25)*(PERSTEEL-1.5)
      IF(PERSTEEL.GE.1.75)P=0.71
      ENDIF
      PERCENTAGE OF STEEL FOR M20 MIX
      IF(FCK.EQ.20)THEN
      IF(PERSTEEL.LE.0.25)P=0.36
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     *   P=0.36+(0.12/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     *   P=0.48+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     *   P=0.56+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     *   P=0.62+(0.05/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     *   P=0.67+(0.05/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LE.1.75))
     *   P=0.72+(0.03/0.25)*(PERSTEEL-1.5)
      IF((PERSTEEL.GT.1.75).AND.(PERSTEEL.LE.2.0))
     *   P=0.75+(0.04/0.25)*(PERSTEEL-1.75)
      IF((PERSTEEL.GT.2.0).AND.(PERSTEEL.LE.2.25))
     *   P=0.79+(0.02/0.25)*(PERSTEEL-2.0)
      IF((PERSTEEL.GT.2.25).AND.(PERSTEEL.LT.2.5))
     *   P=0.81+(0.01/0.25)*(PERSTEEL-2.25)
      IF(PERSTEEL.GE.2.50)P=0.82
      ENDIF
      IF(P.LE.TOWVEE)THEN
      WRITE(8,*)'THE SHEAR REINFORCEMENT IS PROVIDED'
      Vc=P*BW*EFFD
      Vs=SF-Vc
c     WRITE(*,*)'ENTER THE DIA OF STIRRUP'
      READ(5,*)DS
      ASV=3.14*DS**2/4
      SPACING=0.87*FY*2*ASV*EFFD*1000/Vs
      SPACING=NINT(SPACING-5)
      WRITE(8,*)'PROVIDE 2 LEGGED',DS,'mm STIRRUP @',SPACING,'mm C
      GO TO 2000
      ENDIF
      WRITE(8,*)'THE SECTION IS SAFE AGAINST SHEAR'
2000  STOP
      END

      SUBROUTINE QUAD
      COMMON FCK,BW,BF,EFFD,MDU,DF,N,XU1,XU2,XU3
      IF (N.EQ.1)THEN
```

```
      DISC=SQRT(B*B-4.0*A*C)
      ROOT1=(-B+DISC)/(2.0*A)
      ROOT2=(-B-DISC)/(2.0*A)
      XU1=ROOT1
      IF(XU1.GT.ROOT2)XU1=ROOT2
      ENDIF
      IF(N.EQ.2)THEN
      A=(0.1512*FCK-(0.0051*FCK*(BF-BW)))
      B=(-0.36*FCK*BW*EFFD)+(0.067*FCK*EFFD*(BF-BW))-
     *(0.1682*FCK*DF*(BF-BW))
      C=MDU+0.095*FCK*(BF-BW)*(DF**2)-0.2925*FCK*(BF-BW)*EFFD*DF
      DISC=SQRT(B*B-4.0*A*C)
      ROOT1=(-B+DISC)/(2.0*A)
      ROOT2=(-B-DISC)/(2.0*A)
      XU2=ROOT1
      IF(XU2.GT.ROOT2)XU2=ROOT2
      ENDIF
      IF(N.EQ.3)THEN
      A=0.1512
      B=-0.36*FCK*BW*EFFD
      C=MDU-0.45*FCK*(BF-BW)*DF*(EFFD-DF*0.5)
      DISC=SQRT(B*B-4.0*A*C)
      ROOT1=(-B+DISC)/(2.0*A)
      ROOT2=(-B-DISC)/(2.0*A)
      XU3=ROOT1
      IF(XU3.GT.ROOT2)XU3=ROOT2
      ENDIF
      RETURN
      END
```

INPUT DATA FOR THE DESIGN OF T BEAM

1. Enter the value of Fck, Fy, Breadth of web, Breadth of flange, Depth of flange, effective depth, effective cover, MDU, W and length.
2. Enter the diameter of bar to be used as tensile reinforcement.
3. Enter the diameter of compression bar.
4. Enter the diameter of stirrup.

UT DATA
****

M NO:6 ROOF SLAB
****

415 300 1830 120 570 40 23060000 24.31 4270

TPUT DATA
****

AM NO:6 ROOF SLAB
****

****

SUMMARY OF DESIGN

****

SINGLY REINFORCED SECTION DESIGN

HE TOTAL TENSILE REINFORCEMENT: 112.3905000mm²
IAMETER OF TENSILE REINFORCEMENT: 20mm
O OF TENSILE REINFORCEMENT BAR: 2
HE SECTION IS SAFE AGAINST SHEAR

****

T DATA
****
NO:6 FLOOR SLAB
****

415 300 1830 120 570 40 28940000 25.52 4270

TPUT DATA
****
EAM NO:6 FLOOR SLAB
****
****
SUMMARY OF DESIGN
****

SINGLY REINFORCED SECTION DESIGN

THE TOTAL TENSILE REINFORCEMENT: 141.1591000mm²
DIAMETER OF TENSILE REINFORCEMENT: 20mm
NO OF TENSILE REINFORCEMENT BAR: 2
THE SECTION IS SAFE AGAINST SHEAR
****

OUTPUT DETAILS OF BEAMS

Beam No.	Type of Beam	Type of reinforcement Sec.	Diameter of tensile reinforcement(mm)	No. of bars	Section is safe against shear or not.
1. Roof	Rectangular	singly	20	3	safe
Floor	Rectangular	singly	20	3	safe
2. Roof	Rectangular	singly	20	3	safe
Floor	Rectangular	singly	20	3	safe
3. Roof	Rectangular	singly	20	3	safe
Floor	Rectangular	singly	20	3	safe
4. Roof	Rectangular	singly	20	3	safe
Floor	Rectangular	singly	20	3	safe
5. Roof	T-Beam	singly	16	2	safe
Floor	T-Beam	singly	16	2	safe
6. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	20	2	safe
7. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	20	2	safe
8. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	20	2	safe
9. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	20	2	safe

Beam No.	Type of Beam	Type of reinforcement Sec.	Diameter of tensile reinforcement(mm)	No. of bars	Section is safe against shear or not.
10. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
11. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
12. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
13. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
14. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
15. Roof	T-Beam	singly	20	2	safe
Floor	T-Beam	singly	16	2	safe
16. R	T-Beam	singly	16	3	safe
F	T-Beam	singly	16	3	safe
17. R	T-Beam	singly	16	3	safe
F	T-Beam	singly	16	3	safe
18. R	T-Beam	singly	16	3	safe
F	T-Beam	singly	16	3	safe
19. R	T-Beam	singly	16	3	safe
F	T-Beam	singly	16	3	safe

Beam No.		Type of Beam	Type of reinforcement Sec.	Diameter of tensile reinforcement(mm)	No. of bars	Section is safe against shear or not.
20.	R	T-Beam	singly	16	3	safe
	F	T-Beam	singly	16	3	safe
21.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	2	safe
22.	R	T-Beam	singly	20	3	safe
	F	T-Beam	singly	20	3	safe
23.	R	Rectangular	singly	20	3	safe
	F	Rectangular	singly	20	3	safe
24.	R	Rectangular	singly	20	3	safe
	F	Rectangular	singly	20	3	safe
25.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	3	safe
26.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	3	safe
27.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	3	safe
28.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	2	safe

Beam No.		Type of Beam	Type of reinforcement Sec.	Diameter of tensile reinforcement(mm)	No. of bars	Section is safe against shear or not.
29.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	2	safe
3Ø.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	2	safe
31.	R	T-Beam	singly	16	2	safe
	F	T-Beam	singly	16	2	safe
32.	R	Rectangular	singly	16	3	safe
	F	Rectangular	singly	16	3	safe
33.	F	Rectangular	singly	16	3	safe
	F	Rectangular	singly	16	3	safe
34.	R	Rectangular	singly	16	3	safe
	F	Rectangular	singly	16	3	safe

[5] D E S I G N O F C O L U M N S

In general column sections are subjected to Axial and iaxial bending moments. To arrive at the column dimension, the oments on the columns are not known at the initial stage. Therefore usually the direct load carrying above a certain section of the column is computed, which are accounting for the bending moment action on the section. More dimension can be provided along the plan of greater bending moment.

Dimensions of the columns are calculated assuming two percent of reinforcement of the gross sectional area of the columns. Limit state design method is adopted for calculating the dimensions of the columns. The design is computerised and the programme is written in FORTRAN - 77.

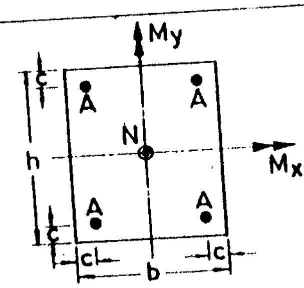
```
*************************************************************************
      COLUMN DESIGN USING DESIGN CHARTS
*************************************************************************
       ASSUME COLUMN SIZE AS 300mm x 600mm
       AXIAL LOAD IS THE SHEAR FORCE AT THE COLUMN
       INTEGER DIA,SPACING,NOBARS
       REAL MuX,MuY,Nu,OMEGA
       WRITE(*,*)'ENTER AXIAL LOAD in N'
       READ(*,*)AXLLOD
       WRITE(*,*)'ENTER MX,MY in N-mm'
       READ(*,*)MX,MY
       WRITE(*,*)'ENTER THE DIA in mm'
       READ(*,*)DIA
       WRITE(*,*)'ENTER THE VALUE OF Fck'
       READ(*,*)Fck
       WRITE(*,*)'ENTER THE VALUE OF Fy'
       READ(*,*)Fy
       OPEN(2,FILE='C1.OUT')
       H=600.
       B=300.
       C=60.
       GAMMAC=1.5
       GAMMAS=1.15
       N=AXLLOD
       HBYB=H/B
       CBYH=C/H
       MuX=(GAMMAC/1.5)*(MX/(FCK*B*H*H))
       MuY=(GAMMAC/1.5)*(MY/(FCK*B*B*H))
       Nu=(GAMMAC/1.5)*(N/(FCK*B*H))
       WRITE(*,10001)MuX,MuY,Nu
0001   FORMAT('INPUT OMEGA FOR:µx=',F4.2,'µy=',F4.2,'Nu='
       WRITE(*,*)'ENTER OMEGA VALUE'
       READ(*,*)OMEGA
       ATOT=OMEGA/(GAMMAC/1.5*FY/B/H/FCK)
       ASTI=ATOT/4.
       WRITE(2,10002)
10002  FORMAT(75('*'))
       WRITE(2,*)'COLUMN No:  1 '
       WRITE(2,*)'                SUMMERY OF DESIGN OF COLUMN'
       WRITE(2,10003)
10003  FORMAT(75('*'))
       WRITE(2,*)'DIAMETER OF THE BAR:',DIA,'mm'
       WRITE(2,*)'THE TOTAL REINFORCEMENT IS:',ATOT,'mm2'
       WRITE(2,*)'THE REINFORCEMENT IS:',ASTI,'mm2 IN EACH CORNER'
       NOBARS=ASTI/((3.141592654*(DIA/2)**2))
       NB=NINT((NOBARS+0.5))
       NB10=NB*4
       WRITE(2,*)'PROVIDE',NB,'BAR(S) OF',DIA,'mm DIA IN EACH CORNER
     1 AND MIDDLE OF THE LONGER SIDE'
       WRITE(2,*)'TOTALLY PROVIDED',NB10,'BAR(S) OF',DIA,'mmDIA
       WRITE(2,10004)
10004  FORMAT(75('*'))
       STOP
       END
```

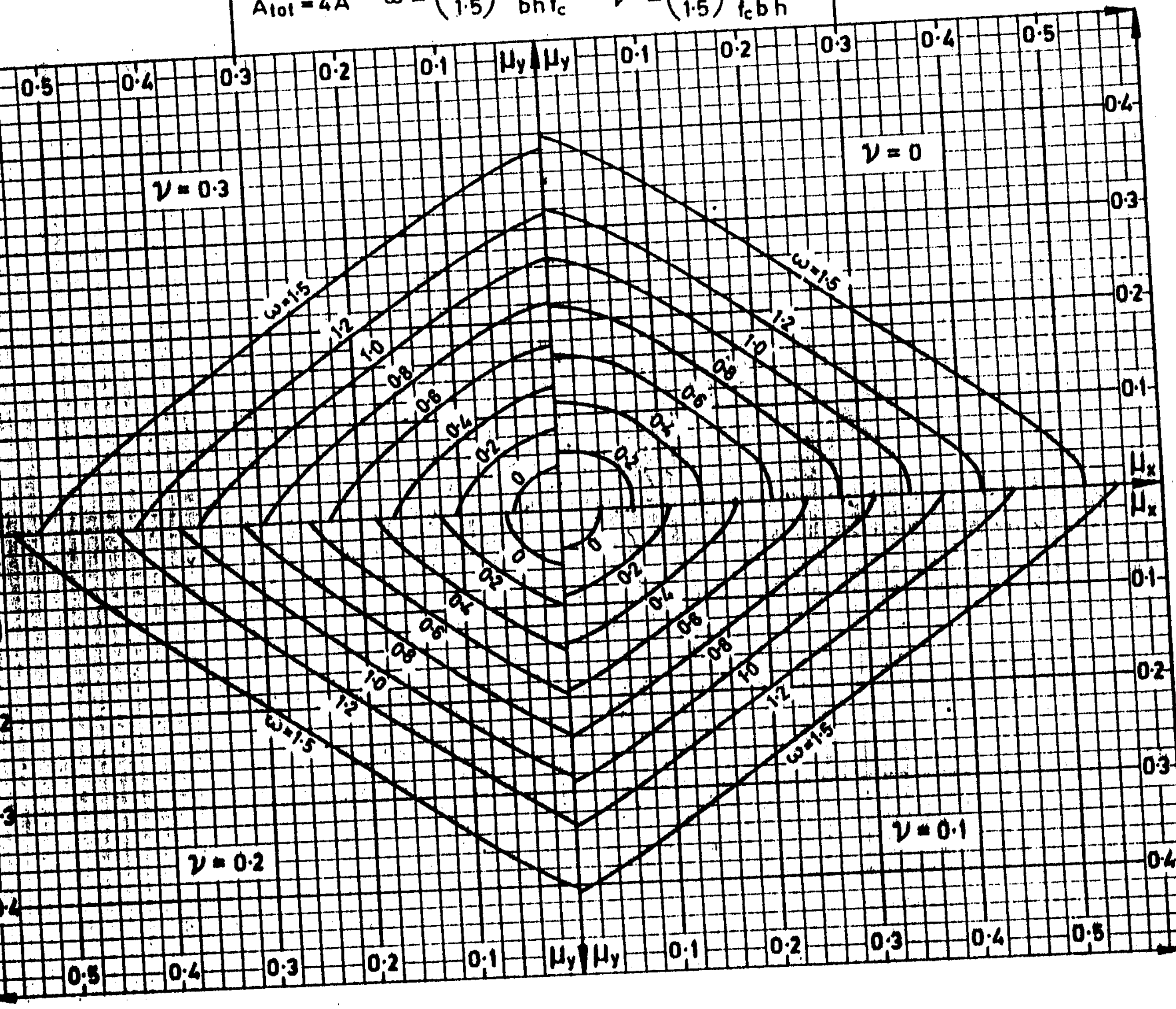
$f_y = 415\ N/mm^2$	
$\frac{h}{b} = 2.0$	$\frac{c}{h} = 0.1$

$$\mu_x = \left(\frac{\gamma_c}{1.5}\right)\frac{M_x}{f_c b h^2}$$

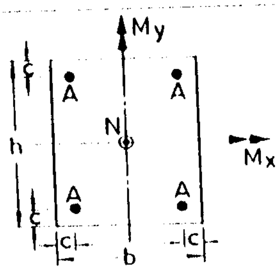
$$\mu_y = \left(\frac{\gamma_c}{1.5}\right)\frac{M_y}{f_c b^2 h}$$

$A_{tot} = 4A \quad \omega = \left(\frac{\gamma_c}{1.5}\right)\frac{A_{tot} f_y}{b h f_c} \quad \nu = \left(\frac{\gamma_c}{1.5}\right)\frac{N}{f_c b h}$





Columns with biaxial bending



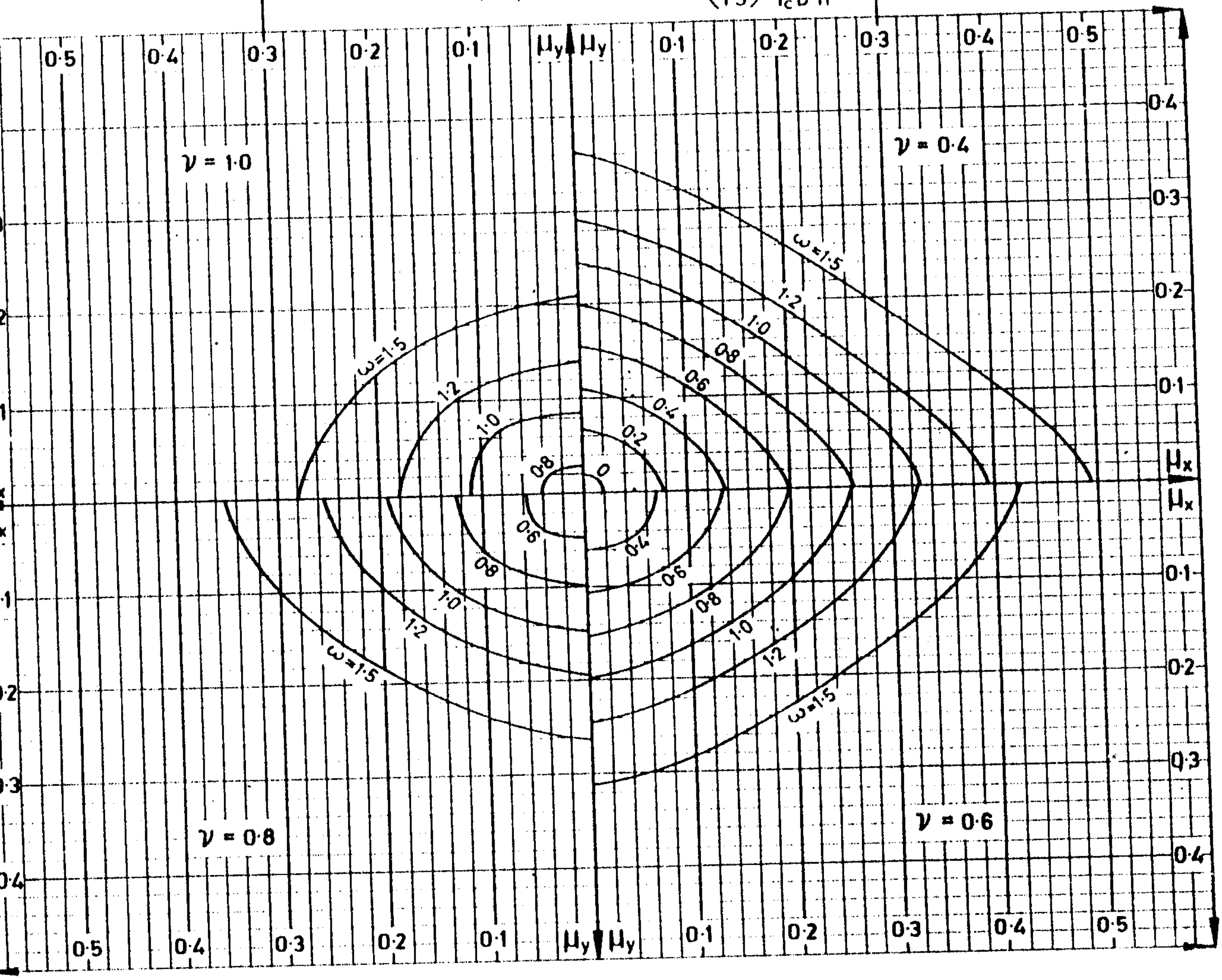
$f_y = 415\ \mathrm{N/mm^2}$

$\frac{h}{b} = 2{\cdot}0 \qquad \frac{c}{h} = 0{\cdot}1$

$$\mu_x = \left(\frac{\gamma_c}{1{\cdot}5}\right)\frac{M_x}{f_c b h^2}$$

$$\mu_y = \left(\frac{\gamma_c}{1{\cdot}5}\right)\frac{M_y}{f_c b^2 h}$$

$A_{tot} = 4A \qquad \omega = \left(\frac{\gamma_c}{1{\cdot}5}\right)\frac{A_{tot} f_y}{b h f_c} \qquad \nu = \left(\frac{\gamma_c}{1{\cdot}5}\right)\frac{N}{f_c b h}$



I N P U T   D A T A S   F O R   C O L U M N   D E S I G N

.	Total Axial force(Pu)x10^3N	(Mux x 10^6)	(Muy x 10^6)
	203.57	18.72	12.66
	302.89	23.72	23.13
	386.05	23.72	-13.52
	236.57	18.72	4.57
	248.53	18.72	10.50
	240.48	19.89	12.66
	358.72	24.89	23.13
	439.2	24.89	-13.52
	249.64	19.89	10.50
	315.54	23.56	12.66

o.	Total Axial force(Pu)x10^{3}N	(Mux x 10^6)	(Muy x 10^6)
	474.62	23.56	18.13
	388.22	23.56	-8.52
	262.44	23.56	4.57
	256.61	23.56	10.50
	294.96	-44.70	12.66
	550.7	-44.70	18.13
	464.33	-44.70	-8.52
.	184.44	-44.70	4.57
.	184.44	-44.70	4.57
).	177.48	-44.70	10.50

DATA
****

MN NO:1
****

R AXIAL LOAD in N
70
ER MX,MY in N-mm
20000
50000
ER THE DIA in mm

ER THE VALUE OF Fck

ER THE VALUE OF Fy

UT OMEGA FOR:μx=0.01μy=0.02Nu=0.1
ER OMEGA VALUE
2

TPUT DATA
****

OLUMN No: 1
****

****

SUMMARY OF DESIGN OF COLUMN

****

DIAMETER OF THE BAR:	20mm
THE TOTAL REINFORCEMENT IS:	1301.2050000mm²
THE REINFORCEMENT IS:	325.3012000mm² IN EACH CORNER

PROVIDE 2BAR(S) OF 20mm DIA IN EACH CORNER AND MIDDLE OF
THE LONGER SIDE
TOTALLY PROVIDED 8BAR(S) OF 20mm DIA

****

OUTPUT COLUMN DETAILS

Column No.	Size of the Column (mm)	Diameter of the bar (mm^2)	Total Rein forcement (mm^2)	Reinforcement in each corner & middle of the longerside	No. of bars each corner & middle of the longer side	Total No. of bars
1.	300x600	20	1301.205	216.867	1	6
2.	300X600	20	2602.41	433.74	2	12
3.	300X600	20	2602.41	433.74	2	12
4.	300X600	20	1301.205	216.87	1	6
5.	300X600	20	1301.205	216.87	1	6
6.	300x600	20	1301.205	216.87	1	6
7.	300X600	20	2602.41	433.74	2	12
8.	300X600	20	1951.81	325.31	2	12
9.	300X600	20	1301.205	216.87	1	6
10.	300X600	20	1301.205	216.87	1	6
11.	300X600	20	1301.205	216.87	1	6
12.	300X600	20	2602.41	433.74	2	12
13.	300X600	20	1301.21	216.87	1	6

Column No.	Size of the Column (mm)	Diameter of the bar (mm^2)	Total Rein forcement (mm^2)	Reinforcement in each corner & middle of the longerside	No. of bars each corner & middle of the longer	Total No. of bars
14.	300X600	20	1301.21	216.87	1	6
15.	300X600	20	1301.21	216.87	1	6
16.	300X600	20	2602.41	433.74	2	12
17.	300X600	20	1301.21	216.87	1	6
18.	300X600	20	1301.21	216.87	1	6
19.	300X600	20	1301.21	216.87	1	6
20.	300X600	20	1951.81	325.31	2	12

[6] DESIGN OF FOOTINGS

The substructure or foundation is that past of structure which is usually placed below the surface of the ground and which transmits the loads to the under lying soil or rock. The two essential requirements in the design of foundation are that the total settlement of the structure shall be limited to a tolerably small amount and that differential settlement of various parts of structure shall be eliminated as nearly possible.

Reinforced concrete footings are provided to transit the load of the structure supported by column posts to the soil. The pressure on the soil the safe bearing capacity of the soil is 200 KN/m^2

The footing is designed as isolated footing by limit state Design method. Concrete grade M15 and steel grade Fe415 are used. The design is computerised and the program is written in FORTRAN 77.

```
*********************************************************************
      ISOLATED FOOTING DESIGN
*********************************************************************
      REAL  SIZE(100),SIZEFOOT(100)
      REAL LOADFACTOR ,DIA ,NOBARS
      OPEN(2,FILE='FT.FOR')
      OPEN(3,FILE='FT20.OUT')
      WRITE(*,*) 'ENTER AXIAL LOAD IN N'
      WRITE(*,*) 'ENTER COLUMN SIZE B,D IN mm'
      WRITE(*,*) 'ENTER SAFE BEARING CAPACITY OF SOIL IN N/mm2'
      WRITE(*,*) 'ENTER THE DEPTH OF FOUNDATION BELOW GL IN mm'
      WRITE(*,*) 'ENTER UNIT WEIGHT OF EARTH IN N/mm3'
      WRITE(*,*) 'ENTER THE CHARACTERISTIC STRENGTH OF MIX IN N/mm2'
      WRITE(*,*) 'ENTER THE YIELD STRENGTH OF STEEL IN N/mm2'
      WRITE(*,*) 'ENTER YIELD STRENGTH OF STEEL IN N/mm2'
      READ(2,*)ALOAD ,(SIZE(I),I=1,2),SBC,DF,UNTWT,Fck,Fy
      AFOOT=ALOAD/SBC
      WFOOT=UNTWT*DF*AFOOT
      TOTWHT=ALOAD+WFOOT
      ACTAREA=TOTWHT/SBC
      IF(SIZE(1).GT.SIZE(2))THEN
      X=SIZE(1)
      SIZE(1)=SIZE(2)
      SIZE(2)=X
      ENDIF
      RATIO=SIZE(1)/SIZE(2)
      SIZEFOOT(1)=SQRT(ACTAREA*RATIO)
      SIZEFOOT(2)=ACTAREA/SIZEFOOT(1)
      SIZEFOOT(1)=(NINT((SIZEFOOT(1)/100)+0.5))*100
      SIZEFOOT(2)=(NINT((SIZEFOOT(2)/100)+0.5))*100
C     ASSUME THE LOAD FACTOR PARTIAL SAFETY FACTOR)AS 1.5
      LOADFACTOR=1.5
      UPWRPR=ALOAD*LOADFACTOR/SIZEFOOT(1)/SIZEFOOT(2)
C     BENDING MOMENT CALCULATION
      BM2=UPWRPR*SIZEFOOT(2)*(((SIZEFOOT(2)-SIZE(2))/2)**2)*0.5
      BM1=UPWRPR*SIZEFOOT(1)*(((SIZEFOOT(1)-SIZE(1))/2)**2)*0.5
      MAXBM=BM1
      IF(MAXBM.LT.BM2)MAXBM=BM2
      IF(BM1.GE.BM2)BR=SIZEFOOT(1)
      IF(BM2.GT.BM1)BR=SIZEFOOT(2)
      DEPTH=SQRT(MAXBM/(0.138*FCK*BR))
C     ASSUME DEPTH AS 600 mm
      DE=600
      IF(DEPTH.LT.DE)DEPTH=560
 20   OVRDEP=DEPTH + 40
      XYZ=0.87*FY
      IF(MAXBM.EQ.BM1)BFCK=SIZEFOOT(1)*FCK
      IF(MAXBM.EQ.BM2)BFCK=SIZEFOOT(2)*FCK
      YZ=XYZ*FY/BFCK
      XY=SQRT((XYZ*DEPTH)**2-4*YZ*MAXBM)
      XY1=XYZ*DEPTH
      AST1=(-XY1+XY)/(-2)/YZ
      AST2=(-XY1-XY)/(-2)/YZ
      AST=AST1*5
      IF(AST.GT.AST2)AST=AST2*5
C     WRITE(*,*)'ENTER THE DIA'
      READ(2,*)DIA
      NOBR=AST/(3.141592654*(DIA/2)**2)
```

```
      VU2=UPWRPR*SIZEFOOT(2)*((SIZEFOOT(2)-SIZE(2))/2)-DEPTH
      VU=VU1
      IF(VU.LT.VU2)VU=VU2
      TOWVEE=VU/BR/DEPTH
      PERSTEEL=AST*100/BR/DEPTH
C     PERCENTAGE OF STEEL FOR M15 MIX
      IF(FCK.EQ.15)THEN
      IF(PERSTEEL.LE.0.25)P=0.35
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     *P=0.35+(0.11/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     *P=0.46+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     *P=0.54+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     *P=0.6+(0.04/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     *P=0.64+(0.04/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LT.1.75))
     *P=0.68+(0.03/0.25)*(PERSTEEL-1.5)
      IF(PERSTEEL.GE.1.75)P=0.71
      ENDIF
C     PERCENTAGE OF STEEL FOR M20 MIX
      IF(FCK.EQ.20)THEN
      IF(PERSTEEL.LE.0.25)P=0.36
      IF((PERSTEEL.GT.0.25).AND.(PERSTEEL.LE.0.5))
     *P=0.36+(0.12/0.25)*(PERSTEEL-0.25)
      IF((PERSTEEL.GT.0.5).AND.(PERSTEEL.LE.0.75))
     *P=0.48+(0.08/0.25)*(PERSTEEL-0.5)
      IF((PERSTEEL.GT.0.75).AND.(PERSTEEL.LE.1.0))
     *P=0.56+(0.06/0.25)*(PERSTEEL-0.75)
      IF((PERSTEEL.GT.1.0).AND.(PERSTEEL.LE.1.25))
     *P=0.62+(0.05/0.25)*(PERSTEEL-1.0)
      IF((PERSTEEL.GT.1.25).AND.(PERSTEEL.LE.1.5))
     *P=0.67+(0.05/0.25)*(PERSTEEL-1.25)
      IF((PERSTEEL.GT.1.5).AND.(PERSTEEL.LE.1.75))
     *P=0.72+(0.03/0.25)*(PERSTEEL-1.5)
      IF((PERSTEEL.GT.1.75).AND.(PERSTEEL.LE.2.0))
     *P=0.75+(0.04/0.25)*(PERSTEEL-1.75)
      IF((PERSTEEL.GT.2.0).AND.(PERSTEEL.LE.2.25))
     *P=0.79+(0.02/0.25)*(PERSTEEL-2.0)
      IF((PERSTEEL.GT.2.25).AND.(PERSTEEL.LT.2.5))
     *P=0.81+(0.01/0.25)*(PERSTEEL-2.25)
      IF(PERSTEEL.GE.2.50)P=0.82
      ENDIF
      IF(P.LE.TOWVEE)THEN
                DEPTH =DEPTH+50
                GO TO 20

      ENDIF
      WRITE(3,*)'FOOTING No:
      WRITE(3,1001)
1001  FORMAT(75('*'))            SUMMARY OF FOOTING DESIGN
      WRITE(3,*)'
      WRITE(3,1002)
1002  FORMAT(75('*'))
      WRITE(3,30)SIZE(1),SIZE(2)
30    FORMAT(3X,'THE SIZE OF THE COLUMN IS:',F7.0,'mm',F7.0,'mm'
      ...SIZEFOOT(1),SIZEFOOT(2)
```

```
      WRITE(3,50)AST
      FORMAT(3X,'AREA OF STEEL:',F6.2,'mm²')
      WRITE(3,60)NOBARS,DIA
      FORMAT(3X,'PROVIDE', F7.0,'BARS OF',F7.0,'mm DIA')
      WRITE(3,*)'THE SECTION IS SAFE AGAINST SHEAR'
      WRITE(3,1003)
3     FORMAT(75('*'))
      STOP
      END
```

INPUT DATA TO THE ISOLATED FOOTING DESIGN

1. Enter the axial size B,D in mm, Depth of foundation unit weight of earth, Fck and Fy.

2. Enter the diameter.

UT DATA
****
TING NO:1
****

570 300 600 0.2 1500 0.00002 15 415

TPUT DATA
****

OTING No: 1
****

```
************************************************************************
         SUMMARY OF FOOTING DESIGN
************************************************************************
 THE SIZE OF THE COLUMN IS:      300.mm,    600.mm
 THE SIZE OF THE FOOTING IS:     800.mm,   1600.mm
 DEPTH:                          560.0000000mm
 OVER ALL DEPTH   =              600.0000000mm
 AREA OF STEEL    =              1188.61mm²
 PROVIDE 4 BARS OF 20 mm  DIA
 THE SECTION IS SAFE AGAINST SHEAR
************************************************************************
```

UT DATA
****

TING NO: 2
****

390 300 600 0.2 1500 0.00002 15 415

TPUT DATA
****

OTING No:2
****

```
**************************************************************************
          SUMMARY OF FOOTING DESIGN
**************************************************************************
 THE SIZE OF THE COLUMN IS:   300.mm,   600.mm
 THE SIZE OF THE FOOTING IS:  1000.mm,  1900.mm
 DEPTH:                       560.0000000mm
 OVER ALL DEPTH  =            600.0000000mm
 AREA OF STEEL   =            2403.53mm²
 PROVIDE 8 BARS OF 20 mm DIA
 THE SECTION IS SAFE AGAINST SHEAR
**************************************************************************
```

OUTPUT DETAILS OF FOOTINGS

Footing No.	Size of Column (mmxmm)	Size of Column (mmxmm) side	Overall footing Depth	Area of reinforcement (mm^2)	No: of bars	Whether the section is safe or not against shear
1.	300x600	800x1600	600.00	1188.61	4 Nos-20mm	safe
2.	300x600	1000x1900	600.00	2403.53	8 Nos-20mm	safe
3.	300x600	1100x2200	600.00	4246.88	14 Nos-20mm	safe
4.	300x600	900x1700	600.00	1487.61	5 Nos-20mm	safe
5.	300x600	900x1700	600.00	1512.42	5 Nos-20mm	safe
6.	300x600	900x1700	600.00	1512.42	5 Nos-20mm	safe
7.	300x600	1100x2100	600.00	3458.48	10 Nos-22mm	safe
8.	300x600	1200x2300	600.00	5012.44	14 Nos-22mm	safe
9.	300x600	900x1700	600.00	1570.56	5 Nos-20mm	safe
10.	300x600	1000x2000	600.00	2909.47	8 Nos-22mm	safe
11.	300x600	1200x2400	600.00	6094.84	17 Nos-22mm	safe
12.	300x600	1100x2200	600.00	4271.23	12 Nos-22mm	safe

Footing No.	Size of Column (mmxmm)	Size of Column (mmxmm)	Overall footing Depth	Area of reinforcement (mm^2)	No: of bars	Whether the section is safe or not against shear
13.	300x600	900x1800	600.00	1968.26	7 Nos-22mm	safe
14.	300x600	900x1800	600.00	1924.07	7 Nos-20mm	safe
15.	300x600	1000x1900	600.00	2339.81	8 Nos-20mm	safe
16.	300x600	1300x2600	600.00	8107.64	22 Nos-22mm	safe
17.	300x600	1200x2400	600.00	5959.28	16 Nos-22mm	safe
18.	300x600	800x1500	600.00	870.89	3 Nos-20mm	safe
19.	300x600	800x1500	600.00	870.89	3 Nos-20mm	safe
20.	300x600	800x1500	600.00	837.84	5 Nos-16mm	safe

[7] DESIGN OF THREE QUARTER TURN STAIRCASE

The arrangement of staircase as known in drawings.

Width of stair case = 1.2m

The height between floors = 3.5m

Provide 3 flights.

Height of each flights = 3500/3

= 1166.67mm

Assume to provide 150mm risers.

No. of risers required $= \dfrac{1166.67}{150}$

= 7.77

Provided 8 risers

Actual height of each riser $= \dfrac{1166.67}{8}$

= 145.83 mm

Number of treads in each flight = No. of risers -1

= 8-1

= 7

Let the tread be 300mm

DESIGN OF FLIGHT AB AND CD

The bearing of flight is 230mm effective horizontal span.

= 1.8 + 1.2 + 0.23 + 23/2

= 3.345m

Let the thickness of waist be 250mm

LOADS

Dead loads of 250mm waist = $0.25 \times 0.25 = 6.25$

Ceiling finish (12.5mm) $0.0125 \times 251 = 0.313$

Total load = $6.563\ KN/m^2$

Corresponding load/sq.m = $\dfrac{(R^2 + T^2)^{1/2}}{T} \times w$

$$= \frac{(0.14583^2 + 0.3^2)^{1/2}}{0.3} \times 6.653$$

$$= 7.23\ KN$$

Hence the actual load/sq.m of plan area will consist of waist and ceiling finish = 7.23 KN

Dead load of steps $= \dfrac{7 \times 145.83 \times 300}{2 \times 1000 \times 1000} \times 25$

$= 3.828$

Top finish $= 0.313$

Live load $= 3.500$

Total load $= 14.87KN$

Bending moment $= \dfrac{wt^2}{8}$

$$= \frac{14.87 \times 3\cancel{3}.345^2}{8}$$

$$= 20.79\ KNm$$

Required effective depth $= \sqrt{\dfrac{M}{Qb}}$

$$= \sqrt{\frac{20.79 \times 10^6}{0.658 \times 1000}}$$

Required effective depth = 177.75mm

Hence safe.

d provided = 250-15-12/2

= 229 mm (using 12mm bars)

Area of steel (Ast) $= \dfrac{M}{stjd} = \dfrac{20.79 \times 10^6}{230 \times 0.9 \times 229}$

$= 438.58mm^2$

Provide 12mm φbars @ 250 mm c/c.

DISTRIBUTION OF STEEL

Ast required = 0.15% of bD

$= \dfrac{0.15}{100} \times 1000 \times 250$

$= 375mm^2$

Provide 8mm φbars @ 130mm c/c.

DESIGN OF FLIGHT BC

Effective horizontal span = 1.2 + 1.2 + 0.23 + 1.5 + 0.23 +0.23/2

= 4.59m

Bending moment $= \dfrac{Wl^2}{8} = \dfrac{14.87 \times 4.59^2}{8}$

= 39.16 KN-m

Required effective depth $= \dfrac{M}{\delta stjd}^{1/2}$

$= \dfrac{39.16 \times 10^6}{230 \times 0.9 \times 229}$

$= 826.11mm^2$

Provided 12 mm @ 130 mm c/c.

DISTRIBUTION STEEL

Area of steel required $= 0.15\%$ of bD

$$= \frac{0.15}{100} \times 1000 \times 250 = 375 \text{ mm}^2$$

Provided 8 mm ϕbars @ 130 mm c/c.

[8] SEPTIC TANK DESIGN

No. of flats = 20

No. of persons/flat = 10

No. of persons (Total) = 20 x 10

= 200

Quantity of sewage per capita per day = 120 lit

De-sledging period = 1year

Quantity of sewage produced per day = 120 x 200

= 24,000 liters/day

Assuming the detention period to be 24hrs

i.e capacity of tank = 24,000 x 24/24

= 24,000 liters

Now assuming the rate of sludge deposit as 30 liters/capita/year and with the given 1 year period of cleaning, we have,

The quantity of sludge deposited = 30 x 200 x 1

= 600 liters

Total capacity of tank required = 24,000 + 6,000

= 30,000 lit

= $30m^3$

Assuming the depth of tank as 2.5m, the cross-sectional area of tank

$$= \frac{30}{2.5} = 12m^2$$

Assume L:B ratio as 4:1, then we have,

$4B^2$ = 12

B = 1.75 m

L = 7.0m

The dimension of the tank will be 7.Ø x 1.75m x(2.5 + Ø.3) as overall depth with Ø.3 m free Board.

Hence use a tank of size.

7.Øm x 1.75m x 2.8m

DESIGN OF SOAK-WELL

The soak-well or soak pit can be designed by assuming media, say as 125Ø liters/cum/day.

$$\text{Volume required for the soak-well} = \frac{30 \times 1000}{1250}$$

$$= 24 m^3.$$

$$\text{Area of soak-well required} = 24/3$$

$$= 8 m^2$$

Diameter of soak-well required

$$\pi/4 \times d^2 = 8$$

$$d^2 = \frac{8 \times 4}{\pi}$$

$$d = \sqrt{\frac{8 \times 4}{\pi}}$$

$$d = 3.19 m$$

$$\text{Say diameter of soak-well} = 3.5 m$$

[9] WATER TANK DESIGN - (RECTANGULAR TANK)

DESIGN:

No. of people = 200

Per capita demand = 200 lit/head/day

Capacity of Tank = 40,000 lit/day

We have,

length of tank (l) = 4.27 m

breadth of tank (b) = 2.44m

volume = lxbxh

40000×10^{6} mm3 = 4270mm x 2440 mm x h

h = 3840 mm

Adopt h = 3850 mm

Provide height of 3.85m for tank with free board of 150mm.

$$\frac{L}{B} = \frac{4.27}{2.44} = 1.75 < 2$$

The walls of tanks are to be designed as continuous slabs.

$$\frac{H}{4} = \frac{3.85}{4} = 0.963\text{m}$$

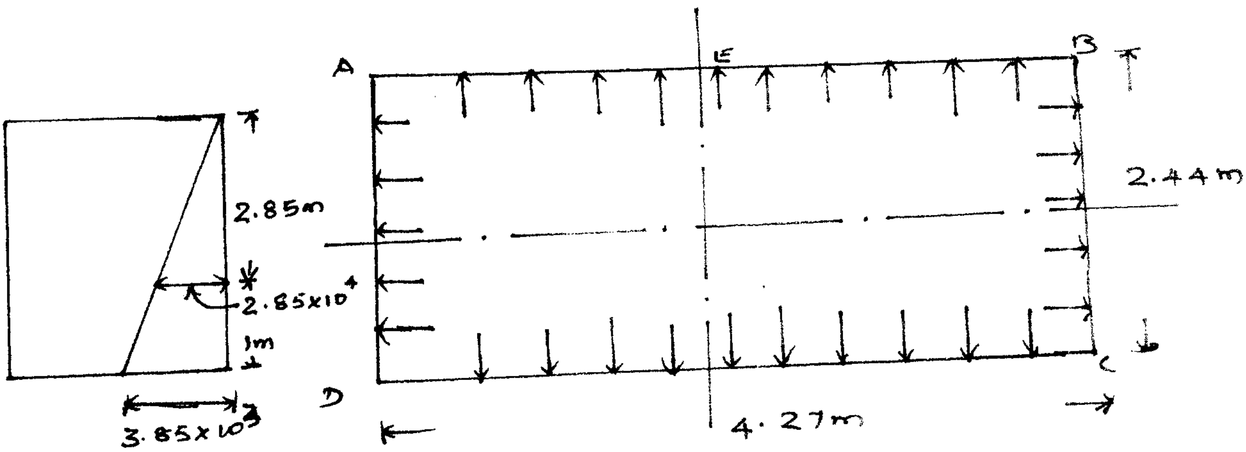
Bottom 1m of tank will be designed as cantilever Pressure at depth of 2.85m.

$P = wh = 2.85 \times 10{,}000$

$= 28500$ N/m2

Moments in the walls are found by moment distribution, As the Frame is symmetrical about both axes moment distribution in alone. For 1/4th of the tank only

Joint	A	
Members	AB	AD
Distribution factor	0.244	0.427
Fixed moment	-2.135p	+1.143p
Balancing	0.242p	0.424p
Final	-1.893p	+1.893p



Moment at support $= 1.893p$

$= 1.893 \times 28500 = 53950.5\text{Nm}$

BM at centre of long span $= \dfrac{28500 \times 4.27^2}{8} - 53950.5$

$= 11004.21 \text{ Nm}$

BM at centre of shorter span $= \dfrac{28500 \times 2.44^2}{8} - 53950.5$

$= -32740.8 \text{ Nm}$

Maximum BM $= 53950.5 \text{ Nm}$

Direct tension in long wall $= \dfrac{28500 \times 2.44}{2}$

$= \dfrac{Wb}{2} = 34770$ N

Direct tension in short wall $= \dfrac{28500 \times 4.27}{2} = \dfrac{Wa}{2}$

$= 60847.5$ N

Let C = 7 N/mm2, m = B, t = 100 N/mm2 on the water tank.

$$k = \frac{1}{1 + t/cm} = 0.48$$

$$j = 1 - k/3 = 0.84$$

$$Q = 1/2\ ckj = 1.411$$

Effective depth $= \sqrt{\dfrac{M}{Qb}}$

$$= \sqrt{\frac{53950.5 \times 10^3}{1.411 \times 1000}}$$

$= 195.54$ mm

Provide overall depth of 250mm width effective depth of 215 mm.

Net moment $= M - Tx$

Area of steel $= \dfrac{M - Tx}{0.84 \times 100 \times 215} + \dfrac{T}{100}$

$$= \frac{53950.5 \times 10^3 - 34770\ (215 - 125)}{0.84 \times 215 \times 100}$$

$$+ \frac{34770}{100}$$

$$= 2814 + 347.7$$

$$= 3161.7 \text{ mm2}$$

Provide 20mm ϕbars @ 95 mmc/c

Area of steel provided $=$ 3307 mm2

Steel @ centre of span

$$\text{Area of steel} = \frac{11004.21 \times 103 - 34770 (215 - 125)}{0.86 \times 215 \times 152}$$

$$+\frac{34770}{100}$$

$$= 688 \text{ mm2}$$

Half the bars from inner face at support are bent into centre face providing area of

$$\frac{3307}{2} = 1653.5\text{mm2}$$

Additional RF of 16 mm ϕ bars @ 160mm c/c. At the centre of shoot span BM is of -ve sign. however nominal RF is provided on that face.

Cantilever moment:

$$\text{cantilever Moment} = 1/2 \times wh \times 1/3$$

$$= 1/2 \times 100,00 \times 3.85 \times 1/3$$

$$= 6416.67 \text{ Nm}$$

$$\text{Area of steel required} = \frac{6416.67 \times 10^{3}}{0.84 \times 215 \times 100}$$

$$= 355.3 \text{ mm2}$$

Provide 8 mm ϕbars @ 150mm c/c

Distribution Steel:

Distribution steel $= 0.3 - \dfrac{0.1 \times (250 - 100)}{(450-100)}$

$= 0.257\ \%$

Area of steel $= \dfrac{0.257}{100} \times 250 \times 1000$

$= 642.5\ mm2$

Area on each face $= 321.2mm2$

Provide 8mm @150 mm c/c.

BASE SLAB:

Provide 150mm thick slab with 8mm bar @ 200 mm c/c both ways @ top and bottom.

Size of rectangular tank

4.27 x 2.44 x 4.0 (3.85 + 0.15)

[5] APPROXIMATE ESTIMATION

AREA:

Total plinth area = 444 Sq.m

RATES:

The rates are assumed as follows:

For ground floor	= Rs. 2000.00/m2
For first floor	= Rs. 2100.00/m2
For second floor	= Rs. 2200.00/m2
For third floor	= Rs. 2300.00/m2
For fourth floor	= Rs. 2400.00/m2

APPROXIMATE COST:

For ground floor	= Rs. 2002000.00
For first floor	= Rs. 2102100.00
For second floor	= Rs. 2202200.00
For third floor	= Rs. 2302300.00
For fourth floor	= Rs. 2402400.00
Total approximate cost	= Rs. 11011000.00

CONCLUSION

In this project, we have used the high level language FORTRAN - 77. We have developed the programme for the analysis of frames, design of beams, columns and footings.

Bearing in mind the cost of construction, maximum utilisation of the area is obtained for minimum space and cost. Using the programme any number of frames can be analysed, evenif the number of storyes is made to increase.

[6] B I B L I O G R A P H Y

Sl. No.	Author's Name	Name of the book	Year of the publication	Publisher
Ø1.	Dutta B.N	Estimation & costing in Civil Engineering	1993	VBS Publications Distributors, Madras.
Ø2.	Jain A.K	Reinforced Concrete-Limit State Design	199Ø	IVem Chand & Bros., Roorke
Ø3.	MacCinley	Reinforced Concrete Design theory and examples	1978	E & F.N. Spon Ltd, London
Ø4.	Natarajan A.A	Design of short axially loaded reinforced column and RC Isolated footing according to IS 456-1978	1982	--NIL--
Ø5.	Ramamrutham S.	Design of R.C. structures	1978	J.C. Kapur for Dhanpat Raj & Sons, Delhi-Jullandur
Ø6.	Reynold. E.	R.C. Designers hand book		Concrete Publication Limited London SW_2

Sl. No.	Author's Name	Name of the book	Year of the publication	Publisher
Ø7.	Vazirani V.N Ratwani	Analysis,. Design & Details of concrete		Khama publishers
Ø8.	V.K. Ghanekar J.P. Jain	Hand book for limit state Design of Reinforced concrete members. (Structural Engineering Research Centre, Roorkee).	1981	Tata MC graw - Hill Publishing Company Ltd., New Delhi.
Ø9.	SHAH S.J	Reinforced "Concrete"	1985	Charotar Publishing House Anand
1Ø.		IS 456-1978 Code of practice for plain and Reinforced concrete	1987	Indian Standards Institution
11.		IS: 875-1964 code of practice for structural Safety of buildings Loading Standards (Revised)	--	--do--
12.		Design Aids - SP - 16		--do--