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Analysis and Design Of Multistory Apartment Building Using ETABS

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ABSTRACT

Practical knowledge is an important and essential skill required by every engineer. For obtaining this skill, an apartment building is analysed and designed, Located in Latur, Maharastra with (B+G+10) storeys having a car parking facility provided at basement floor. The building has a shear wall around the lift pit. The modelling and analysis of the structure is done by using ETABS and the designing was done. Design of slab, stair case and an isolated footing are done manually. The design methods involves load calculations manually and analysing the whole structure by ETABS. The design methods used in ETABS are limit state design confirming to IS code of practice**.**Along with analysing and designing of this building, construction sites were also visited.

Keywords: *Analysis and design, Apartment Building, Lift pit, Shear wall.*

1. Introduction

Practical knowledge is an essential skill required by an engineer. By industrial training, the practical knowledge can be super imposed to technical knowledge. Industrial training is an essential component in the development of the practical and professional skills required by an engineer. For understanding the engineering practice in general and sense of frequent and possible problems that may arise during construction and also necessary solution for these problems can be experienced and understood during industrial training. This exposure to the practical world is the main objective of industrial training.

2. Training Information

The industrial training was done in STRUCTURAL ONE consultancy; Latur under the guidance of Mr. Faiz Sagri.

An Apartment building is modelled and analysed using AUTOCAD 2016 and ETABS 2015 respectively. Design of slab, stair case and an isolated footing are done manually, for obtaining precise results. The building is a B+G+10 storey structure, the basement floor facilitated for car parking. Shear wall is provided around the lift pit, staircase is provided.

The objectives of industrial training are:

- To get exposure to engineering experience and knowledge, which are required in the industry and not taught in the lecture rooms.
- To apply the engineering knowledge taught in the lecture rooms in real industrial situations.
- To share the experience gained from the "industrial training" in the discussion held in the lecture rooms.
- To get a feel of the work environment.
- To gain exposure on engineering procedural work flow management and implementation.
- To get responsibilities and ethics of engineers.

3. A BRIEF DESCRIPTION OF SOFTWARE'S USED IN TRAINING

ETABS 2015:

ETABS is an engineering software product that caters to multi-story building analysis and design. Modeling tools and templates, code-based load prescriptions, analysis methods and solution techniques, all coordinate with the grid-like geometry unique to this class of structure. Basic or advanced systems under static or dynamic conditions may be evaluated using ETABS. For a sophisticated assessment of seismic performance, modal and direct-integration time-history analyses may couple with P-Delta and Large Displacement effects. Nonlinear links and concentrated PMM or fiber hinges may capture material nonlinearity under monotonic or hysteretic behavior. Intuitive and integrated features make applications of any complexity practical to implement. Interoperability with a series of design and documentation platforms makes ETABS a coordinated and productive tool for designs which range from simple 2D frames to elaborate modern high-rises.

The innovative and revolutionary new ETABS is the ultimate integrated software package for the structural analysis and design of buildings. Incorporating 40 years of continuous research and development, this latest ETABS offers unmatched 3D object based modeling and visualization tools, blazingly fast linear and nonlinear analytical power, sophisticated and comprehensive design capabilities for a wide-range of materials, and insightful graphic displays, reports, and schematic drawings that allow users to quickly and easily decipher and understand analysis and design results.

From the start of design conception through the production of schematic drawings, ETABS integrates every aspect of the engineering design process. Creation of models has never been easier - intuitive drawing commands allow for the rapid generation of floor and elevation framing. CAD drawings can be converted directly into ETABS models or used as templates onto which ETABS objects may be overlaid. The state-of-the-art SAP Fire 64-bit solver allows extremely large and complex models to be rapidly analyzed, and supports nonlinear modeling techniques such as construction sequencing and time effects (e.g., creep and shrinkage).

Design of steel and concrete frames (with automated optimization), composite beams, composite columns, steel joists, and concrete and masonry shear walls is included, as is the capacity check for steel connections and base plates. Models may be realistically rendered, and all results can be shown directly on the structure. Comprehensive and customizable reports are available for all analysis and design output, and schematic construction drawings of framing plans, schedules, details, and cross-sections may be generated for concrete and steel structures.

ETABS provides an unequaled suite of tools for structural engineers designing buildings, whether they are working on one-story industrial structures or the tallest commercial high-rises. Immensely capable, yet easy-to-use, has been the hallmark of ETABS since its introduction decades ago, and this latest release continues that tradition by providing engineers with the technologically-advanced, yet intuitive, software they require to be their most productive.

AUTO-CAD 2016:

All the drawing and detailing works for this training were done by making use of AutoCAD 2007, developed by M/s. AUTODESK, USA. As such, this is the pioneering software in CAD. AutoCAD is a vector graphics drawing program. It uses primitive entities such as lines, poly-lines, circles, arcs and text as the foundation for more complex objects. AutoCAD's native file format, DWG, and to a lesser extent, its interchange file format, DXF has become the standards for interchange of CAD data.

4. MODELLING IN ETABS

Importing of Floor Plan from Auto-cad:

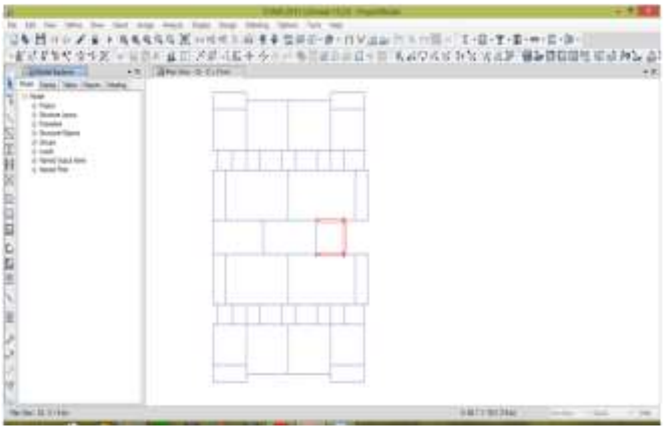

Fig.1 **Centre line plan**

Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

Materials

Table 1 - Material Properties - Summary

Name	Type	E MPa	ν	Unit Weight kN/m³	Design Strengths
HYSD415	Rebar	200000	0	76.9729	Fy=415 MPa, Fu=485 MPa
M25	Concrete	25000	0.2	24.9926	Fc=25 MPa
Mild250	Rebar	200000	0	76.9729	Fy=250 MPa, Fu=410 MPa

Frame Sections

Table 2 - Frame Sections - Summary

Name	Material	Shape
Beam230x380	M25	Concrete Rectangular
Beam230x450	M25	Concrete Rectangular
Beam300x450	M25	Concrete Rectangular
Column300x450	M25	Concrete Rectangular

Shell Sections

Table 3 - Shell Sections - Summary

Name	Design Type	Element Type	Material	Total Thickness mm

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Shearwall	Wall	Shell-Thin	M25	150
Slab125mm	Slab	Shell-Thin	M25	125
Slab175mm	Slab	Shell-Thin	M25	175

Reinforcement Sizes

Table 4 - Reinforcing Bar Sizes

Name	Diameter mm	Area mm²
10	10	79
16	16	201
20	20	314

5. Framing Of Model

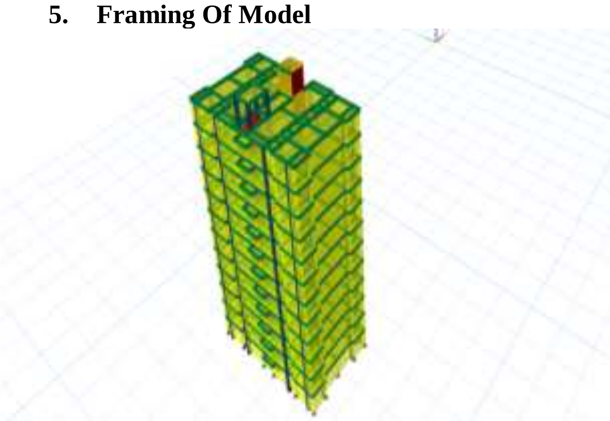

Fig.2 3D Model

6. ANALYSIS IN ETABS

Load Patterns

Table 5 - Load Patterns

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
Superimposed Dead	Superimposed Dead	0	
EQx	Seismic	0	IS1893 2002
EQy	Seismic	0	IS1893 2002

Table 6– Load Cases

Name	Type
Dead	Linear Static
Live	Linear Static
Superimposed Dead	Linear Static
EQx	Linear Static
EQy	Linear Static

Load calculations

Dead loads

The dimensions of the cross section are to be assumed initially which enable to estimate the dead load from the known weights of the structure. The values of the unit weights of the structure and the values of the unit weight of the materials are specified in IS 875:1987(Part-I). As per IS 875: 1987 (part I). The dead load assigned in the ground floor is shown in the figure 3.

- Unit weight of brick = 19.1 kN/m^3
- Unit weight of concrete = 25kN/m^3

Here sample calculation is done:

Wall load

a) Main wall load

Thickness of wall = 150 mm

= unit weight of brick x thickness of wall x(floor height –beam depth)

=19.1 x 0.150 x (3 -0.45)

= 7.305 kN/m

b) Partition wall load

Thickness of wall = 100 mm

= 19.1 x 0.10 x (3 -0.45)

= 4.875 kN/m

c) Parapet wall load

Thickness of wall = 100 mm

= 19.1 x 0.10 x 1.5

= 2.865 kN/m

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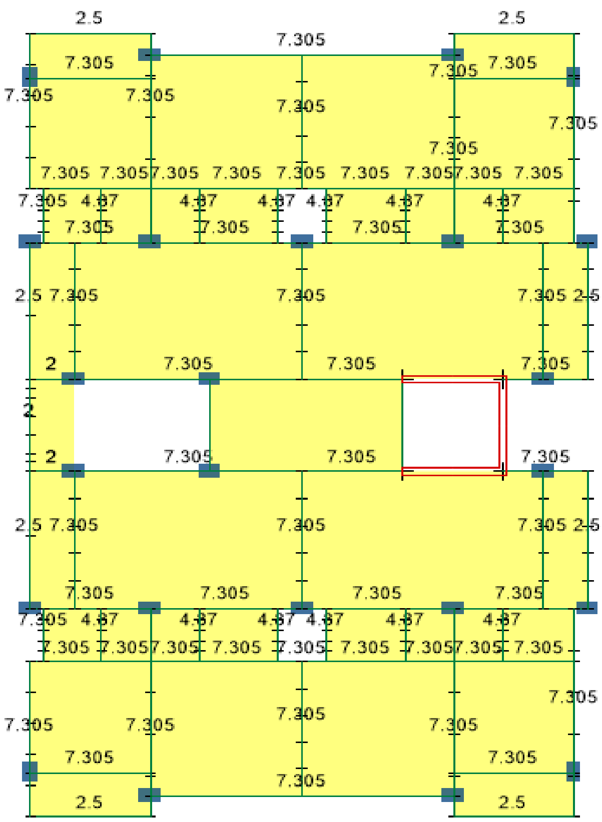

Fig.3- Dead Load

Floor finish = 1.25kN/m^2 (as per IS 875 part 1)

Total floor load = 1.25 kN/m^2

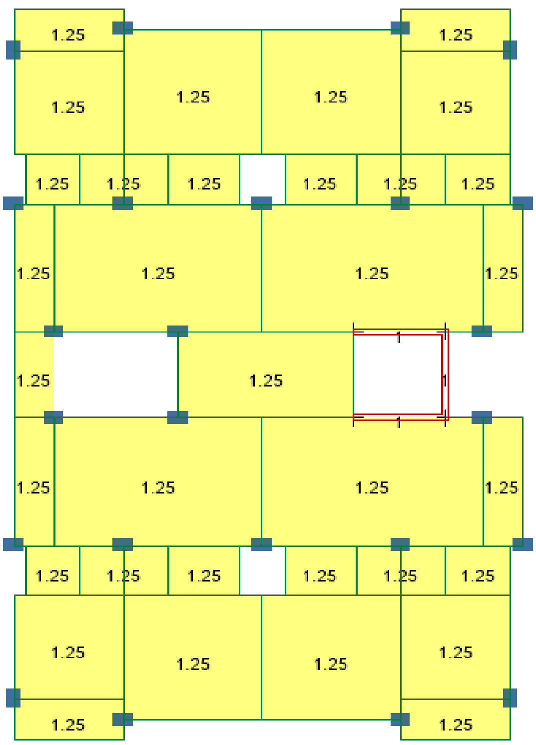

Fig.4- Floor Finish Load (Super Dead)

Live loads

They are also known as imposed loads and consist of all loads other than the dead loads of the structure. The standard values are stipulated in IS875:1987 (part II).The live loads considered are given in table 7. The assigned live load on ground floor in Etabs will be as shown in the figure 5.

Table.7-Live loads

Area	Live load (kN/m^2)
All rooms and kitchens	2
Toilet and bathrooms	2
Corridors, Passages, Staircases	3
Balconies	3
Parking	5
Electrical Room	5
Machine room	5

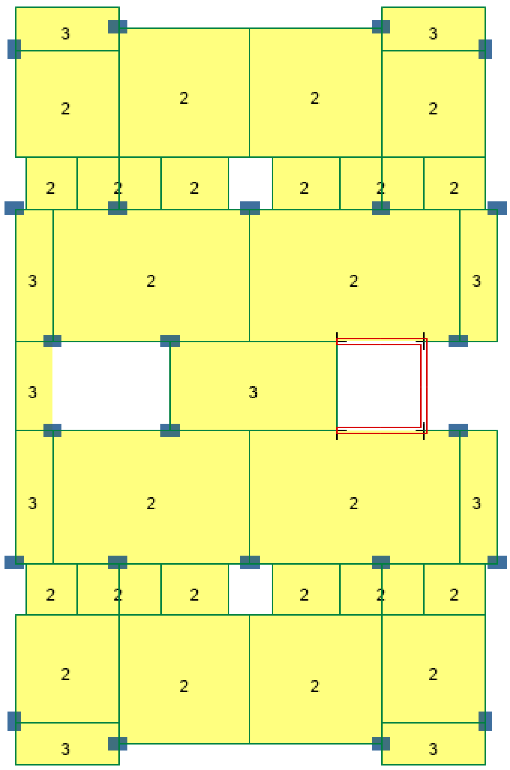

Fig.5-Live Load

Earthquake Forces

Earthquakes generate waves which move from the origin of its location with velocities depending on the intensity and magnitude of the earthquake. The impact of earthquake on the structures depends on the stiffness of the structure, stiffness of the soil media, height and location of the structure, etc. the earthquake forces are prescribed in IS 1893:2002 (part-I).

Since the building is located in Latur, Maharastra, it is included in the zone III. And the seismic base shear calculation and its distribution was done as per IS 1893:2002 (part-I). The

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base shear or total design lateral force along any principle direction shall be determined by the following expression:

$V_B = A_h X W$

Where,

V_B = Design base shear

A_h = Design horizontal seismic coefficient based on fundamental natural period, and type of soil

W = Seismic weight of the building

The design horizontal seismic coefficient,

$$A_h = \frac{ZIS_a}{RS_g}$$

Where,

Z = Zone factor given in table 2, for the maximum considered earthquake (MCE) and service life of the structure in a zone. The factor 2 in the denominator is used so as to reduce the MCE zone factor to the factor for design basic earthquake (DBE) I = Importance factor, depending upon the functional use of structures, characterized by hazardous consequences of failure, post-earthquake functional needs, historical value or economic importance (Table 6 of IS 1893 (Part 1): 2002).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. The value for buildings are given in Table7 of IS 1893 (Part 1): 2002.

Sa/g = Average response acceleration coefficient. Sa/g is determined on the basis of approximate fundamental natural period of vibration on both the directions.

Natural period of vibration,

$$T_a = \frac{0.09 \times h}{\sqrt{d}}$$

Earthquake loading

As per IS 1893:2002 (part-I) earthquake loads are calculated.
Latur belongs to seismic zone 3.
So seismic zone coefficient, Z =0.16
Importance factor, I =1(other buildings)
Response reduction factor, R =3
Height of building =33 m
Dimension of building along X- direction = 12.19 m
Dimension of building along Y- direction =18.288 m

Time period,

Along x direction,

$$T_a = \frac{0.09 \times 33}{\sqrt{12.19}} = 0.850$$

Along y direction,

$$T_a = \frac{0.09 \times 33}{\sqrt{18.288}} = 0.694$$

Auto Seismic Loading

IS1893 2002 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQx according to IS1893 2002, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.850$ sec

Factors and Coefficients

Seismic Zone Factor, Z [IS Table 2] $Z = 0.16$

Response Reduction Factor, R [IS Table 7] $R = 3$

Importance Factor, I [IS Table 6] $I = 1$

Site Type [IS Table 1] = II

Seismic Response

Spectral Acceleration Coefficient, S_a /g [IS 6.4.5] $\frac{S_a}{g} = \frac{1.36}{T}$ $\frac{S_a}{g} = 1.36$

Equivalent Lateral Forces

Seismic Coefficient, A_h [IS 6.4.2] $A_h = \frac{ZI\frac{S_a}{g}}{2R}$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	V_b (kN)
X	0.850	15000.4234	553.9871

Applied Storey Forces

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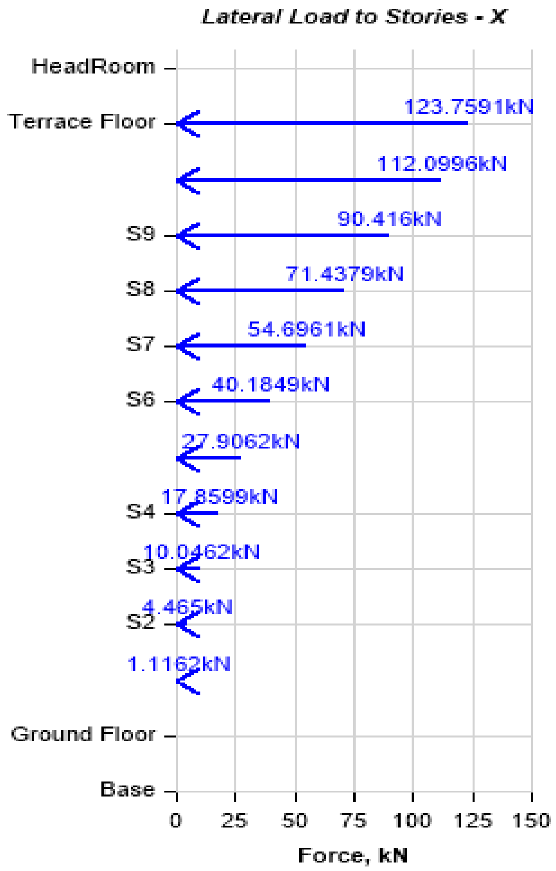


IS1893 2002 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQy according to IS1893 2002, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.694$ sec

Factors and Coefficients

Seismic Zone Factor, Z [IS Table 2] $Z = 0.16$

Response Reduction Factor, R [IS Table 7] $R = 3$

Importance Factor, I [IS Table 6] $I = 1$

Site Type [IS Table 1] = II

Seismic Response

Spectral Acceleration Coefficient, S_a /g [IS 6.4.5] $\frac{S_a}{g} = \frac{1.36}{T}$ $\frac{S_a}{g} = 1.36$

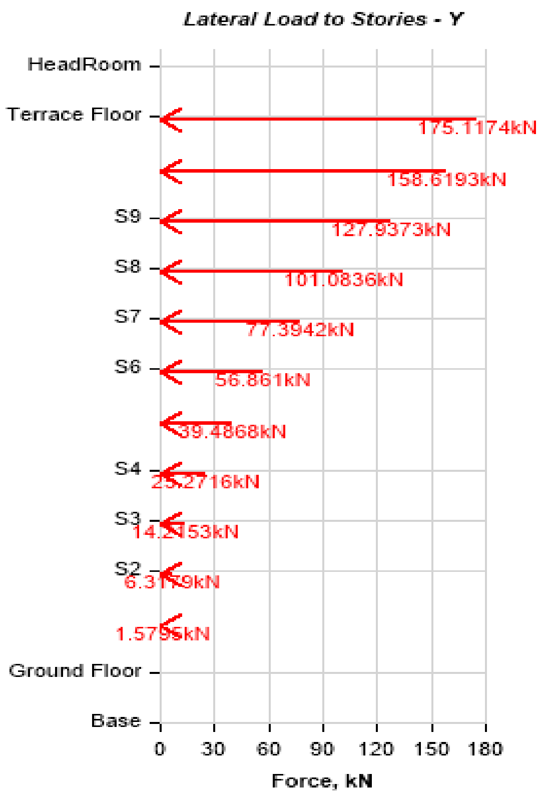
Equivalent Lateral Forces

Seismic Coefficient, A_h [IS 6.4.2] $A_h = \frac{ZI\frac{S_a}{g}}{2R}$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	V_b (kN)
Y	0.694	15000.4234	783.8838

Applied Story Forces



Load Combinations

Design of the structures would have become highly expensive in order to maintain either serviceability and safety if all types of forces would have acted on all structures at all times. Accordingly the concept of characteristics loads has been accepted to ensure at least 95 percent of the cases, the characteristic loads are to be calculated on the basis of average/mean load of some logical combinations of all loads mentioned above.

IS 456:2000, IS 875:1987 (Part-V) and IS 1893(part-I):2002 stipulates the combination of the loads to be considered in the design of the structures. The different combinations used are:

Table.8- Load Combinations

Name	Load Case/Combo	Scale Factor	Type	Auto
UDCon1	Dead	1.5	Linear Add	No
UDCon1	Superimposed Dead	1.5		No
UDCon2	Dead	1.5	Linear Add	No
UDCon2	Live	1.5		No
UDCon2	Superimposed Dead	1.5		No
UDCon3	Dead	1.2	Linear Add	No
UDCon3	Live	1.2		No
UDCon3	Superimposed Dead	1.2		No

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UDCon3	EQx	1.2		No
UDCon4	Dead	1.2	Linear Add	No
UDCon4	Live	1.2		No
UDCon4	Superimposed Dead	1.2		No
UDCon4	EQx	-1.2		No
UDCon5	Dead	1.2	Linear Add	No
UDCon5	Live	1.2		No
UDCon5	Superimposed Dead	1.2		No
UDCon5	EQy	1.2		No
UDCon6	Dead	1.2	Linear Add	No
UDCon6	Live	1.2		No
UDCon6	Superimposed Dead	1.2		No
UDCon6	EQy	-1.2		No
UDCon7	Dead	1.5	Linear Add	No
UDCon7	Superimposed Dead	1.5		No
UDCon7	EQx	1.5		No
UDCon8	Dead	1.5	Linear Add	No
UDCon8	Superimposed Dead	1.5		No
UDCon8	EQx	-1.5		No
UDCon9	Dead	1.5	Linear Add	No
UDCon9	Superimposed Dead	1.5		No
UDCon9	EQy	1.5		No
UDCon10	Dead	1.5	Linear Add	No
UDCon10	Superimposed Dead	1.5		No
UDCon10	EQy	-1.5		No
UDCon11	Dead	0.9	Linear Add	No
UDCon11	Superimposed Dead	0.9		No
UDCon11	EQx	1.5		No
UDCon12	Dead	0.9	Linear Add	No
UDCon12	Superimposed Dead	0.9		No
UDCon12	EQx	-1.5		No
UDCon13	Dead	0.9	Linear Add	No
UDCon13	Superimposed Dead	0.9		No
UDCon13	EQy	1.5		No
UDCon14	Dead	0.9	Linear Add	No
UDCon14	Superimposed Dead	0.9		No
UDCon14	EQy	-1.5		No
UDWal1	Dead	1.5	Linear Add	No
UDWal1	Superimposed Dead	1.5		No
UDWal2	Dead	1.5	Linear Add	No
UDWal2	Live	1.5		No
UDWal2	Superimposed Dead	1.5		No
UDWal3	Dead	1.2	Linear Add	No
UDWal3	Live	1.2		No
UDWal3	Superimposed Dead	1.2		No
UDWal3	EQx	1.2		No
UDWal4	Dead	1.2	Linear Add	No
UDWal4	Live	1.2		No
UDWal4	Superimposed Dead	1.2		No
UDWal4	EQx	-1.2		No
UDWal5	Dead	1.2	Linear Add	No
UDWal5	Live	1.2		No
UDWal5	Superimposed Dead	1.2		No
UDWal5	EQy	1.2		No
UDWal6	Dead	1.2	Linear Add	No
UDWal6	Live	1.2		No
UDWal6	Superimposed Dead	1.2		No
UDWal6	EQy	-1.2		No
UDWal7	Dead	1.5	Linear Add	No
UDWal7	Superimposed Dead	1.5		No
UDWal7	EQx	1.5		No
UDWal8	Dead	1.5	Linear Add	No
UDWal8	Superimposed Dead	1.5		No
UDWal8	EQx	-1.5		No
UDWal9	Dead	1.5	Linear Add	No
UDWal9	Superimposed Dead	1.5		No
UDWal9	EQy	1.5		No
UDWal10	Dead	1.5	Linear Add	No
UDWal10	Superimposed Dead	1.5		No
UDWal10	EQy	-1.5		No
UDWal11	Dead	0.9	Linear Add	No
UDWal11	Superimposed Dead	0.9		No
UDWal11	EQx	1.5		No
UDWal12	Dead	0.9	Linear Add	No
UDWal12	Superimposed Dead	0.9		No
UDWal12	EQx	-1.5		No
UDWal13	Dead	0.9	Linear Add	No
UDWal13	Superimposed Dead	0.9		No
UDWal13	EQy	1.5		No
UDWal14	Dead	0.9	Linear Add	No
UDWal14	Superimposed Dead	0.9		No
UDWal14	EQy	-1.5		No
Envelope combo	UDCon1	1	Envelope	No
Envelope combo	UDCon2	1		No
Envelope combo	UDCon3	1		No
Envelope combo	UDCon4	1		No
Envelope combo	UDCon5	1		No
Envelope combo	UDCon6	1		No
Envelope combo	UDCon7	1		No
Envelope combo	UDCon8	1		No
Envelope combo	UDCon9	1		No
Envelope combo	UDCon10	1		No
Envelope combo	UDCon11	1		No
Envelope combo	UDCon12	1		No
Envelope combo	UDCon13	1		No
Envelope combo	UDCon14	1		No

All these combinations are built in the Etabs 2015. analysis results from the critical combinations are used for the design of structural member.

Note:

DL - Dead load

LL - Live load

EL_x - Earthquake load in x direction

EL_z - Earthquake load in z direction

Analysis Results

The structure was analysed as ordinary moment resisting space frames in the versatile software Etabs 2015. Joint co-ordinate command allows specifying and generating the co-ordinates of the joints of the structure, initiating the specifications of the structure. Member incidence command is used to specify the members by defining connectivity between joints. The columns and beams are modelled using beam elements. Member properties have to be specified for each member. From the analysis, maximum design loads, moments and shear on each member was obtained. From these values, we design the structure.

Axial Force

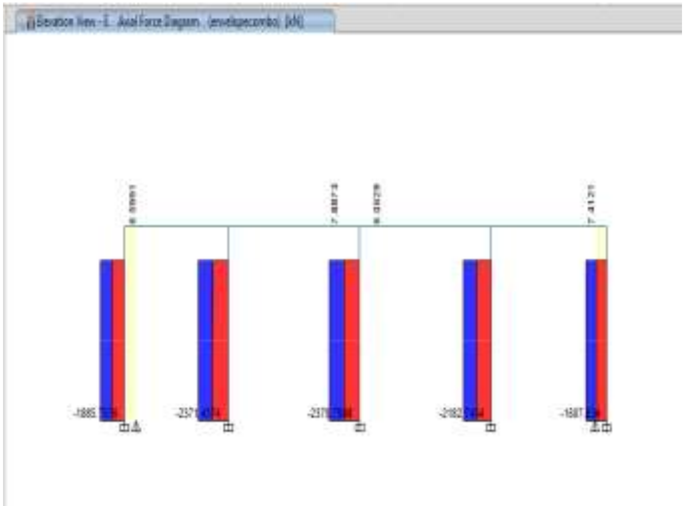

Fig.6 Axial Force Diagram

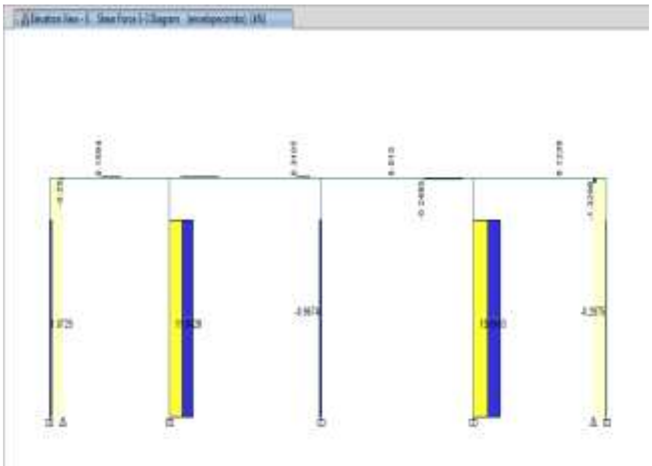

Fig.7 Bending Moment Diagram

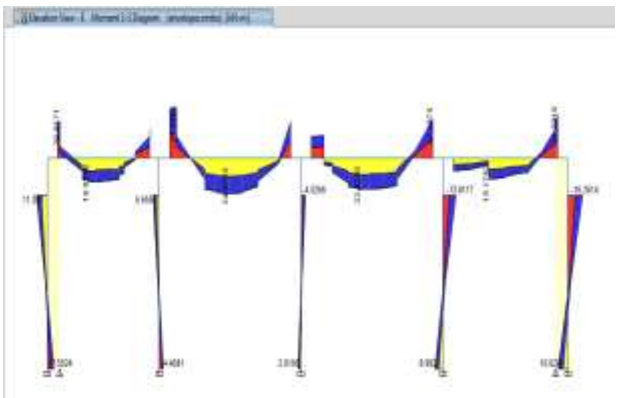

Fig.8 Torsion Force Diagram

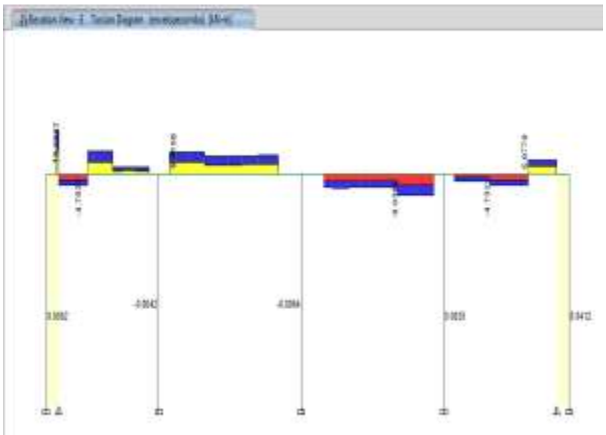

Fig.9 Torsional moment diagram

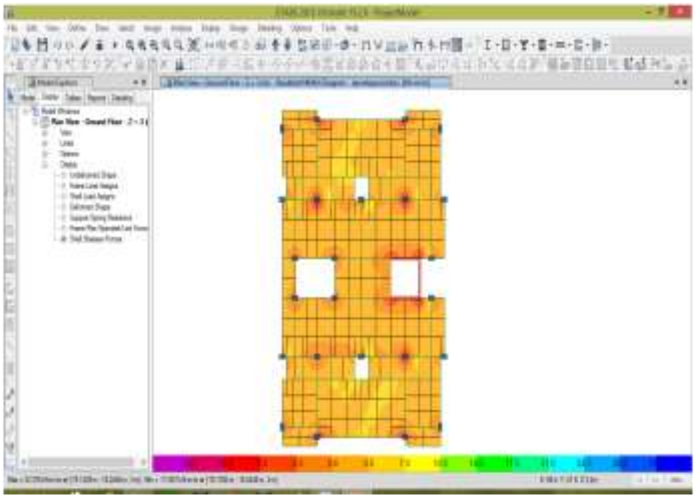

Fig.10 Slab bending moment diagram

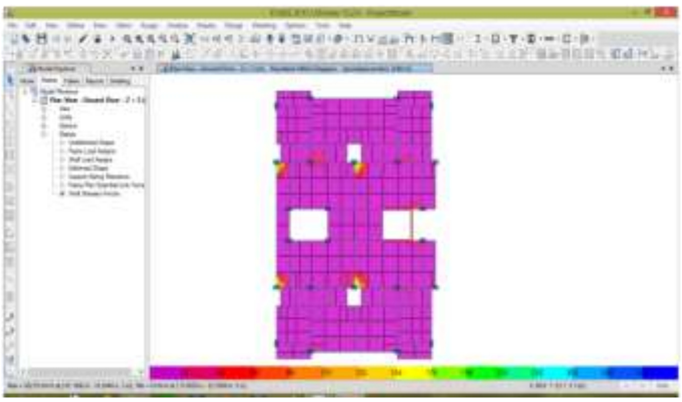

Fig.11 Slab shear force diagram

7. Design of RC Building

General

The aim of structural design is to achieve an acceptable probability that the structure being designed will perform the function for which it is created and will safely withstand the influence that will act on it throughout its useful life. These influences are primarily the loads and the other forces to which it will be subjected. The effects of temperature fluctuations, foundation settlements etc. should be also considered.

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The design methods used for the design of reinforced concrete structures are working stress method, ultimate load method and limit state method. Here we have adopted the limit state method of design for slabs, beams, columns and stairs.

In the limit state method, the structure is designed to withstand safely all loads liable to act on it through its life and also to satisfy the serviceability requirements, such as limitation to deflection and cracking. The acceptable limit of safety and serviceability requirements before failure is called limit state. All the relevant limit states should be considered in the design to ensure adequate degrees of safety and serviceability. The structure should be designed on the basis of most critical state and then checked for other limit states.

The design of a structure must satisfy three basic requirements:

- Stability - To prevent overturning, sliding or buckling of the structure, or part of it, under the action of loads.
- Strength - To resist safely the stresses induced by the loads in the various structural members.
- Serviceability - To ensure satisfactory performance under service load conditions which implies providing adequate stiffness and reinforcement to contain deflections, crack widths and vibrations within acceptable limits, and also providing impermeability and durability.

Concrete Frame Design in ETABS

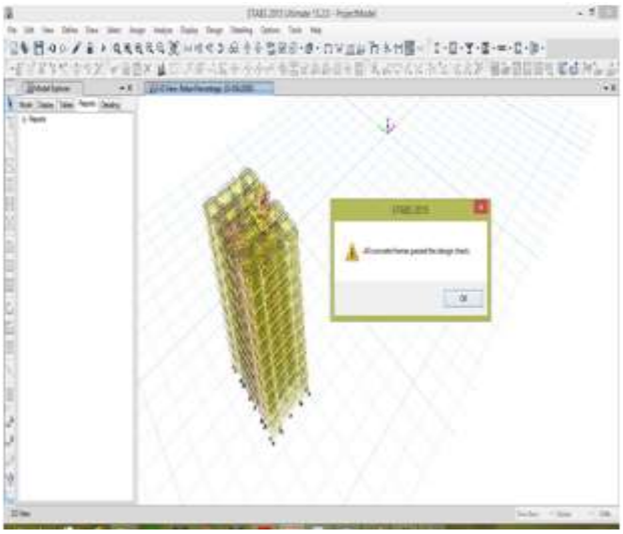

Fig.12

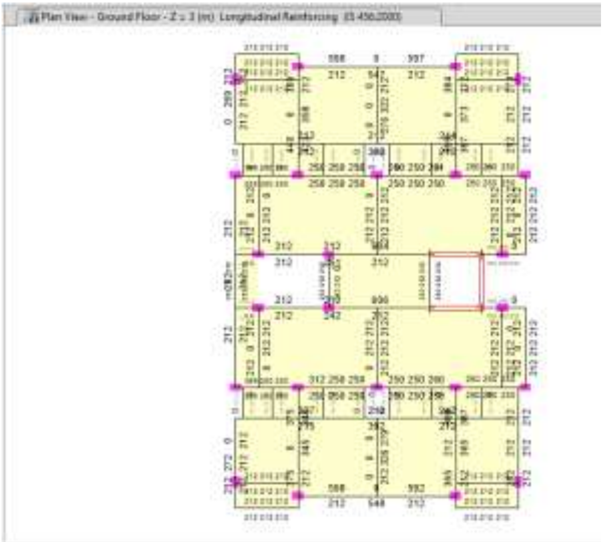

Fig.13

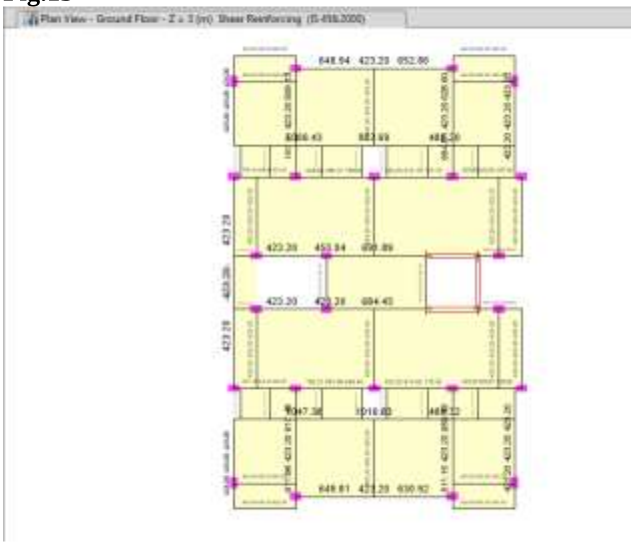

Fig.14

Beam section design (ETABS)

Beam Element Details

Level	Element	Unique Name	Section ID	Length (mm)	LLRF
Ground Floor	B9	10D	Beam230x450	2440	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
230	450	230	0	30	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	415	250

Design Code Parameters

γ_C	γ_S
1.5	1.15

Flexural Reinforcement for Major Axis Moment, M_{u3}

	End-I Rebar Area mm²	End-I Rebar %	Middle Rebar Area mm²	Middle Rebar %	End-J Rebar Area mm²	End-J Rebar %

	End-I Rebar Area mm²	End-I Rebar %	Middle Rebar Area mm²	Middle Rebar %	End-J Rebar Area mm²	End-J Rebar %
Top (+2 Axis)	212	0.2	212	0.2	212	0.2
Bot (-2 Axis)	212	0.2	212	0.2	212	0.2

Flexural Design Moment, M_{u3}

	End-I Design M_u kN-m	End-I Station Loc mm	Middle Design M_u kN-m	Middle Station Loc mm	End-J Design M_u kN-m	End-J Station Loc mm
Top (+2 Axis)	-7.4992	406.7	-2.7195	1626.7	-4.0314	2440
Combo	envelopecombo		envelopecombo		envelopecombo	
Bot (-2 Axis)	2.9202	406.7	8.4427	1626.7	5.8811	2440
Combo	envelopecombo		envelopecombo		envelopecombo	

Shear Reinforcement for Major Shear, V_{u2}

End-I Rebar A_{sv} /s mm²/m	Middle Rebar A_{sv} /s mm²/m	End-J Rebar A_{sv} /s mm²/m
423.2	423.2	423.2

Design Shear Force for Major Shear, V_{u2}

End-I Design V_u kN	End-I Station Loc mm	Middle Design V_u kN	Middle Station Loc mm	End-J Design V_u kN	End-J Station Loc mm
10.5504	406.7	0.0004	1626.7	6.2524	2440
envelopecombo		envelopecombo		envelopecombo	

Torsion Reinforcement

Shear Rebar A_{svt} /s mm²/m
0

Design Torsion Force

Design T_u kN-m	Station Loc mm	Design T_u kN-m	Station Loc mm
1.3507	1220	1.3507	1220
envelopecombo		envelopecombo	

Column Section Design (ETABS)

Column Element Details

Level	Element	Unique Name	Section ID	Length (mm)	LLRF
Ground Floor	C29	189	Column300x450	3000	0.556

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
300	450	58	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	415	250

Design Code Parameters

γ_C	γ_S
1.5	1.15

Longitudinal Reinforcement Design for P_u - M_{u2} - M_{u3} Interaction

Column End	Rebar Area mm²	Rebar %
Top	1080	0.8
Bottom	1080	0.8

Design Axial Force & Biaxial Moment for P_u - M_{u2} - M_{u3} Interaction

Column End	Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Station Loc mm	Controlling Combo
	kN	kN-m	kN-m	mm	
Top	852.19	0.1859	-16.2614	2470	envelopecombo
Bottom	859.6904	-0.0938	10.6284	0	envelopecombo

Shear Reinforcement for Major Shear, V_{u2}

Column End	Rebar A_{sv} /s mm²/m	Design V_{u2} kN	Station Loc mm	Controlling Combo
Top	552	10.8866	2470	envelopecombo
Bottom	552	10.8866	0	envelopecombo

Shear Reinforcement for Minor Shear, V_{u3}

Column End	Rebar A_{sv} /s mm²/m	Design V_{u3} kN	Station Loc mm	Controlling Combo
Top	828	0.2975	2470	envelopecombo
Bottom	828	0.2975	0	envelopecombo

SHEAR WALL DESIGN (ETABS)

Shear Wall Preferences - IS 456-2000

Item	Value
Rebar Material	HYSD415
Rebar Shear Material	Mild250
Phi (Steel)	1.15
Phi (Concrete)	1.5
PMax factor	0.8
# Interaction Curves	24
# Interaction Points	11
Min Eccentricity Major?	No
Min Eccentricity Minor?	No
Edge Design PT-Max	0.06
Edge Design PC-Max	0.04
Section Design IP-Max	0.04
Section Design IP-Min	0.0025
D/C Ratio Limit	0.95

Shear Wall Pier Overwrites - IS 456-2000

Story	Pier	Design	LL RF	Seismic	PierSec Type	End Bar	Edge Bar	EdgeBarSpc mm	Cover mm	Material	Design/Check
Ground Floor	P1	Yes	1	Yes	Uniform Reinforcing Section	3	3	250	31.3	M25	Design
Ground Floor	P2	Yes	1	Yes	Uniform Reinforcing Section	2	2	250	31.3	M25	Design

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Shear Wall Pier Summary - IS 456-2000 (Part 1 of 2)

Story	Pier Label	Station	Design Type	Edge Rebar	End Rebar	Rebar Spacing mm	Required Reinf %	Current Reinf %	Pier Leg	Leg X1 mm	Leg Y1 mm	Leg X2 mm
Ground Floor	P1	Top	Uniform	10	10	250	0.25	0.46	Top Leg 1	-73600.2	-24448.3	-71558.6
Ground Floor	P1	Top	Uniform	10	10	250	0.25	0.46	Top Leg 2	-73600.2	-22356.8	-71558.6
Ground Floor	P1	Bottom	Uniform	10	10	250	0.25	0.46	Bottom Leg 1	-73600.2	-24448.3	-71558.6
Ground Floor	P1	Bottom	Uniform	10	10	250	0.25	0.46	Bottom Leg 2	-73600.2	-22356.8	-71558.6
Ground Floor	P2	Top	Uniform	8	8	250	0.25	0.29	Top Leg 1	-71558.6	-24448.3	-71558.6
Ground Floor	P2	Bottom	Uniform	8	8	250	0.25	0.29	Bottom Leg 1	-71558.6	-24448.3	-71558.6

Shear Wall Pier Summary - IS 456-2000 (Part 2 of 2)

Story	Pier Label	Station	Leg Y2 mm	Shear Rebar mm²/m	Boundary Zone Left mm	Boundary Zone Right mm	Warnings	Errors
Ground Floor	P1	Top	-22356.8	375			No Message	No Message
Ground Floor	P1	Bottom	-24448.3	375			No Message	No Message
Ground Floor	P1	Bottom	-22356.8	375			No Message	No Message
Ground Floor	P2	Top	-22356.8	375			No Message	No Message
Ground Floor	P2	Bottom	-22356.8	375			No Message	No Message

IS 456:2000 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Ground Floor	P2	-71558.6	-23402.5	2091.4	150	1

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	415	250

Design Code Parameters

Γ_S	Γ_C	IP_{MAX}	IP_{MIN}	P_{MAX}	MinEcc Major	MinEcc Minor
1.15	1.5	0.04	0.0025	0.8	No	No

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	-71558.6	-24448.3	-71558.6	-22356.8	2091.4	150
Bottom	Leg 1	-71558.6	-24448.3	-71558.6	-22356.8	2091.4	150

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	784	0.0025	0.0029	UDWal14	-13.4082	0	0	313714
Bottom	784	0.0025	0.0029	UDWal14	13.4082	0.4022	0	313714

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	V_c kN	$V_c + V_s$ kN
Top	Leg 1	375	UDWal14	-13.4082	0	0	72.7817	209.1792
Bottom	Leg 1	375	UDWal14	13.4082	0	0	73.155	209.5525

Boundary Element Check

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa
Top–Left	Leg 1	0	UDWal1	0	0	0	0
Top–Right	Leg 1	0	UDWal1	0	0	0	0
Bottom–Left	Leg 1	0	UDWal1	22.3469	0	0.07	5
Botttom–Right	Leg 1	0	UDWal1	0	0	0	0

IS 456:2000 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
S9	P1	-72579.4	-23402.5	083.3	150	1

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	415	250

Design Code Parameters

Γ_S	Γ_C	IP_{MAX}	IP_{MIN}	P_{MAX}	MinEcc Major	MinEcc Minor
1.15	1.5	0.04	0.0025	0.8	No	No

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	-73600.2	-24448.3	-71558.6	-24448.3	2041.6	150
Top	Leg 2	-73600.2	-22356.8	-71558.6	-22356.8	2041.6	150
Bottom	Leg 1	-73600.2	-24448.3	-71558.6	-24448.3	2041.6	150
Bottom	Leg 2	-73600.2	-22356.8	-71558.6	-22356.8	2041.6	150

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	1531	0.0025	0.0046	UDWal14	-26.1778	0	0	612490
Bottom	1531	0.0025	0.0046	UDWal14	26.1778	0	0	612490

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	V_c kN	$V_c + V_s$ kN
Top	Leg 1	375	UDWal14	-13.0889	0	0	71.0488	204.1988
Top	Leg 2	375	UDWal14	-13.0889	0	0	71.0488	204.1988
Bottom	Leg 1	375	UDWal14	13.0889	0	0	71.4132	204.5632
Bottom	Leg 2	375	UDWal14	13.0889	0	0	71.4132	204.5632

Boundary Element Check

Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa
Top–Left	Leg 1	0	UDWal1	0	0	0	0
Top–Right	Leg 1	0	UDWal1	0	0	0	0
Top–Left	Leg 2	0	UDWal1	0	0	0	0
Top–Right	Leg 2	0	UDWal1	0	0	0	0
Bottom–Left	Leg 1	0	UDWal1	21.8149	0	0.07	5
Botttom–Right	Leg 1	0	UDWal1	0	0	0	0
Bottom–Left	Leg 2	0	UDWal1	0	0	0	0
Botttom–Right	Leg 2	0	UDWal1	21.8149	0	0.07	5

The detailing of concrete frame and shear wall was done using ETABS and various drawings and scheduling tables were obtained.

Design of Two-Way Slab

Slabs are plate elements having their depth much smaller than other two dimensions. They usually carry a uniformly distributed load from the floors and roof of the building. Design of reinforced concrete slab was done using IS 456:2000. Slabs of thickness 125 mm is used in the building and designed as two-way slab. Grade of concrete M25 is assumed for slab design. The slab to be designed is shown in Figure 15.

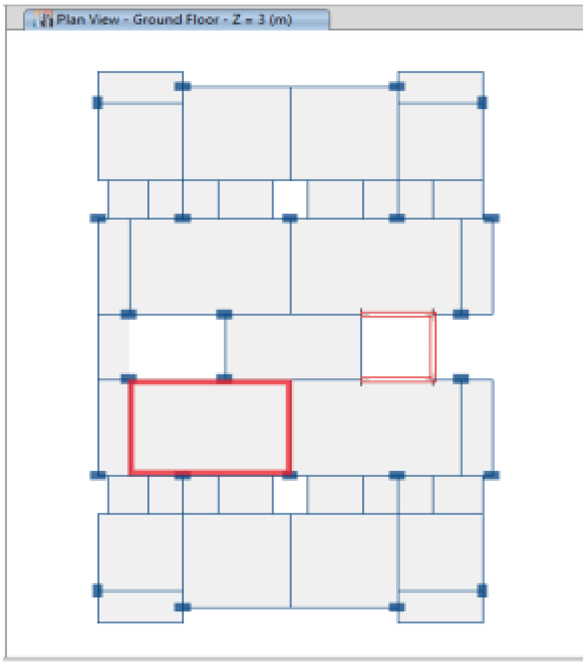

Fig.15 Two Way Slab

Material constants

Concrete, f_{ck} = 25N/mm²

Steel, f_y = 415 N/mm²

Dimensioning

Clear span distance in shorter direction, l_x = 3.11 m

Clear span distance in longer direction, l_y = 4.6 m

As per IS 456:2000, Clause 24.1,
Assuming thickness of slab 125mm
Assume 20mm cover and 8mm diameter bars

Effective depth, d = 125 – 20 – 8/2 = 101mm

Effective span

As per IS 456: 2000 clause 22.2

Eff. Span along short and long spans is computed as:

L_{ex1} = centre to centre of support = 3.11m

L_{ex2} = clear span + eff. depth = 3.11 + 0.101= 3.211m

L_{ex1} = centre to centre of support = 4.6m

L_{ex2} = clear span + eff. depth= 4.6 + 0.101 = 4.701m

Eff. span along short span, L_{ex} = 3.211m

Eff. span along long span, L_{ey} = 4.701m

Load calculation

Dead Load on Slab = 0.125×25 = 3.125kN/m²
Live Load on Slab = 2kN/m²
Floor Finish = 1kN/m²
Total load = 6.125kN/m²
Factored load = 1.5×6.125 = 9.187kN/m

Type of slab

Eff. span along short span, L_{ex} = 3.211m

Eff. span along long span, L_{ey} = 4.701m

= 4.701/3.211 = 1.46< 2.

Hence, design as two-way slab.

Ultimate design moment coefficients

As per IS 456:2000 table 26, take the moment coefficients for = 1.46 interior panels.

Short span moment coefficients:

Negative moment coefficient, αx = 0.052
Positive moment coefficient, αx = 0.040

Long span moment coefficients:

Negative moment coefficient, αy = 0.032
Positive moment coefficient, αy = 0.024

Design moments

$M_x(-ve) = \alpha_x W l_x^2$ = 0.052× 9,187 × 3.211² =4.925kNm

$M_x(+ve) = \alpha_x W l_x^2$ = 0.040 × 9.187 × 3.211² = 3.788kNm

$M_y(-ve) = \alpha y\, W l_x^2$ = 0.032× 9.187 ×3.211² = 3.031kNm

$M_y(+ve) = \alpha y\, W l_x^2$ = 0.024 × 9.187 × 3.211² = 2.598kNm

Check for depth

$M_u = 0.133 f_{ck} bd^2$

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$4.925 \times 10^6 = 0.133 \times 25 \times 1000 \times d^2$

= 38.48 mm < 101mm

Hence the effective depth selected is sufficient to resist the design ultimate moment.

Reinforcements along Short and long span directions

As per IS: 456 Annex G Clause. G.1

x_u /d =0.47 is less than limiting value (0.48)

The area of reinforcement is calculated using the relation:

$$M_u = 0.87 f_y A_{st}\, d\{1 - \left(\frac{A_{st} f_y}{bd\, f_{ck}}\right)\}$$

			Area (mm^2)
short span	+ve moment(kNm)	3.788kNm	**105.71**
	-ve moment(kNm)	4.925kNm	**138.19**
long span	+ve moment(kNm)	2.598kNm	**72.098**
	-ve moment(kNm)	3.031kNm	**84.28**

Check for area of steel

As per IS 456 clause 26.5.2.1

A= 0.12 % of bD = 0.0012x1000x125 = 150 mm^2

Check for spacing

As per IS 456:2000 Clause. 26.3.3(b)

Maximum spacing = 3d or 300mm, whichever is less

= 3 × 101 = 303mm (or) 300mm (take lesser value)

= 300 mm

Reinforcement provided

Short span: Provide 8mm diameter bars @ 275mm c/c

($A_{st\ prov}$ = 182.78 mm²)

Long span: Provide 8mm diameter bars @ 275mm c/c

($A_{st\ prov}$ = 182.78 mm²)

$Spacing_{prov} < spacing_{max}$

Check for shear

As per IS 456:2000, Table13

Shear force, *Vu* = $1wl_x/2$

= 1x9.187x3.211/2= 14.75kN

As per IS 456:2000 Clause 40.1

Nominal shear stress, $\tau_v = \frac{v_u}{bd}$

= 14.75× 10³/ (1000 ×101)

= 0.146N/mm²

Percentage of steel, $p_t = 100\, A_s$ / bd

= (100 x201)/ (1000x 101) = 0.20

Permissible shear stress, τ_c = 0.33N/mm² (IS 456:2000, Table 19)

Design shear strength of concrete = k τ_c

= 1.3x 0.33 = 0.429 N/mm² (IS 456:2000 Clause 40.2)

Maximum shear stress,

τ_{cmax}= 3.1 N /mm² (IS 456:2000 Table 20)

$\tau_v < k\tau_c < \tau_{cmax}$, so shear reinforcement is not required

Check for deflection

$A_{st\ prov}$ = 182.78 mm² (From 6.2.11)

A_{streq} =150 mm²

$f_s = 0.58 x f_y A_{streq} / A_{st\ prov}$ = 197.53 N/mm² =

$p_t = 100\, A_s / bd$ = (100 ×182.78) / (1000 × 101) =0.18

Modification factor = 2 (IS 456:2000, fig. 4)

Permissible *l/d* ratio = 32 × 2= 64

Actual *l/d* = (4701/101) = 46.54< 64

Therefore, deflection is safe with provided depth.

Check for cracking

(As per IS 456:2000, Clause 43.1)

1. Steel provided is more than 0.12%
2. Spacing of main steel < 3d = 3 ×125 = 375mm
3. Diameter of reinforcement <D/8 = 125/8 = 15.62mm

Hence it is safe against cracking.

Reinforcement detailing

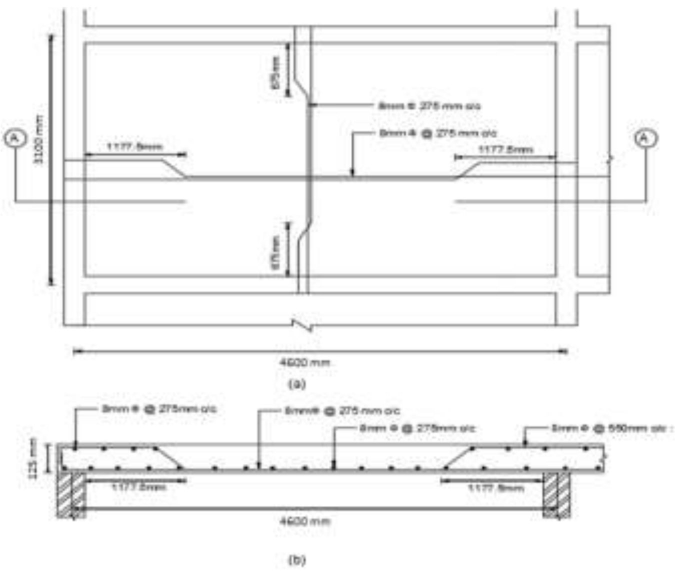

Fig.16 Reinforcement detailing of two way slab

Design of Staircase

Material Constants

Concrete, f_{ck} = 25 N/mm²

Steel, f_y = 415 N/mm²

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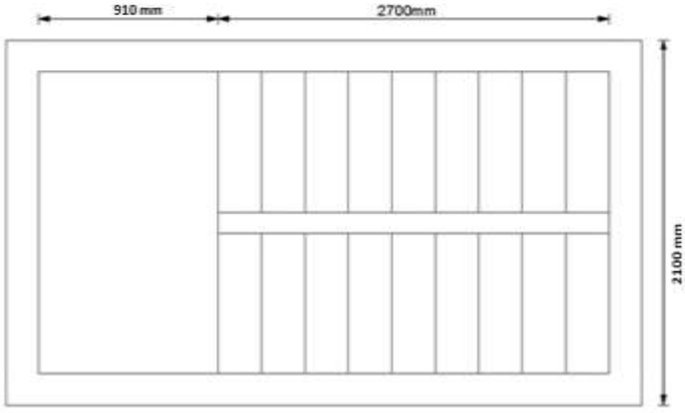

Fig.17

Preliminary dimensioning

Rise of stair, R = 150mm

Tread of stair, T = 300mm

Effective span = 3.65+0.175

=3.825 m

(As per IS 456:2000, Clause 33.1)

Let thickness of waist slab = 175mm

Use 12mm dia. bars and clear cover 25mm

Load calculation

Self-weight of landing slab = 0.125 x 25 = 3.125kN/m^2

Live load on loading slab = 3kN/m^2

Finishes = 1 kN/m^2

Total load on the landing slab = 7.125kN/m^2

Factored load = 1.5x 7.125= 10.68kN/m^2

Dead load of waist slab

= Thickness of waist slab×25×$\frac{\sqrt{r^2+t^2}}{t}$

= 0.175 ×25× = 3.439kN/m

The self-weight of the steps is calculated by treating the step to be equivalent horizontal slab of thickness equal to half the rise.

Self-weight of step = 0.5×0.15×25 = 1.875kN/m²

Floor finish = 1kN/m²

Live load = 3kN/m²

Total service load= 9.314kN/m²

Consider 1m width of waist slab

Total service load /m run = 9.314×1.0 = 9.314kN/m

Total ultimate load = W_u = 1.5×9.314 = 13.971kN/m

Ultimate design moment

Maximum bending moment at the centre of the span is given by,

Mu =$\frac{Wx l_{eu}^2}{8}$ =$\frac{13.971X3.825X3.825}{8}$ = 25.55kNm

Check for the depth of waist slab

d=$\sqrt{\frac{\text{Mu}}{b\times 0.134\times f_{ck}}}$ = $\sqrt{\frac{25.55\times 10^6}{1000x25x0.134}}$ = 87.33mm

$d_{provided} > d_{required}$

Hence the effective depth selected is sufficient to resist the ultimate moment

Reinforcements

$\frac{\text{Mu}}{b\times d^2}$= $\frac{25.55\times 10^6}{1000\times 175\times 175}$= 0.834

From table 3 of SP 16: 1980,

P_t = 0.241

$\frac{100Ast}{bd}$ = P

A_{st} = $\frac{0.241X1000X175}{100}$ = 421.75mm

Maximum spacing for 12 mm Ø bars

Spacing = $\frac{1000ast}{Ast}$ =$\frac{1000\times\frac{\pi}{4}\times 12^2}{421.75}$ = 268mm

Provide 12 mm Ø bars @ 250 mm c/c spacing

Check for spacing of main steel

As per IS 456:2000 Cl. 26.3.3 (b)

Max spacing = (300 or 3d)whichever is less

=, whichever is less

= 300 mm

Spacing provided < spacing maximum ∴ Safe

Check for area of steel

As per IS 456:2000, Cl. 26.5.2.1,

A_{st} min = 0.12% cross sectional area

=$\frac{1000\times 144\times 0.12}{100}$ = 172.8 mm^2

$Ast_{provided} > Ast_{minimum}$

Hence ok.

Distribution reinforcement

0.12% cross sectional area =$\frac{1000\times 144\times 0.12}{100}$

= 172.8 mm^2

Use 8mm Ø bars

Spacing =$\frac{1000ast}{Ast}$ =$\frac{1000\times\frac{\pi}{4}\times 8^2}{172.8}$ = 290.88 mm

Spacing = 250mm

Provide 8 mm Ø bars at 250 mm c/c

Check for spacing of distribution steel

As per IS 456:2000 Cl: 26.3.3 (b)

Max spacing = (5d or 450mm) whichever is less,

= whichever is less

= 450 mm

$\text{Spacing}_{provided} < \text{spacing}_{maximum}$

∴ Safe

Check for shear (As per IS 456:2000, Clause 40)

Shear, V_u = $\frac{W_u \times L_e}{2}$ = $\frac{13.971\times 3.825}{2}$ = $26.72\ kN$

= 41.28kN

As per IS 456:2000, Clause 40.1

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Nominal shear stress, $\tau = \frac{V_u}{bd} = \frac{26.72\times10^3}{1000\times144}$ = 0.185N/mm²

$P_t = \frac{100\,Ast}{bd}$ = 0.241
= = 0.167
As per IS 456: 2000, Table 19, τ_c = 0.355N/ mm²
As per IS 456: 2000, Cl: 40.2
Design shear strength of concrete, ($k\times\tau_c$)
= 1.25 × 0.355= 0.443 N/ mm²
As per IS 456: 2000, Table 20
Max. value of shear stress, τ_{cmax} = 3.1 N/ mm²
$\tau_v < \tau_c < \tau_{cmax}$

So shear reinforcement is not required.

Reinforcement detailing of staircase

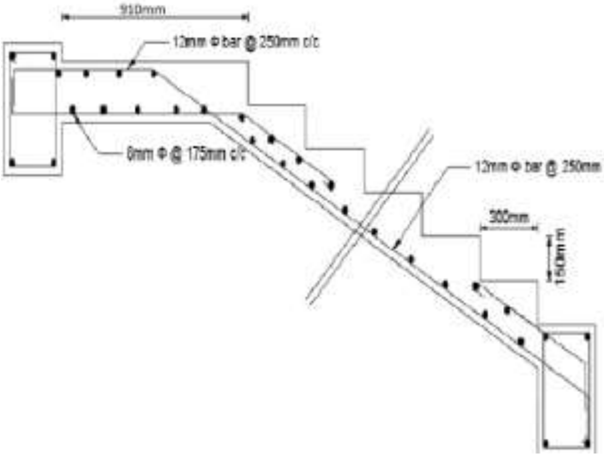

Fig.18 Reinforcement detailing of staircase

Design of Isolated Footing

Foundation is that part of the structure which is in direct contact with soil. The R.C. structures consist of various structural components which act together to resist the applied loads and transfer them safely to soil. In general the loads applied on slabs in buildings are transferred to soil through beams, columns and footings. Footings are that part of the structure which are generally located below ground Level. They are also referred as foundations. Footings transfer the vertical loads, Horizontal loads, Moments, and other forces to the soil.

Material constants

Use M_{25} grade concrete and HYSD steel bars of grade Fe_{500}.
Concrete, f_{ck} = 25 N/mm²
Steel, f_y = 415 N/mm²
Column size =230 mm x 450 mm
Depth of column, a = 450 mm
Breadth of column, b = 230 mm
Factored axial Load, P_u =2505 kN
Safe Bearing Capacity of soil = 200 kN/m^2

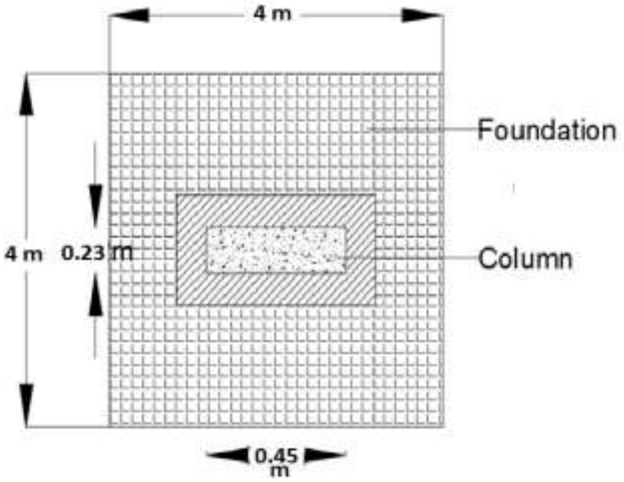

Fig.19 Size of footing

Size of footing

Factored axial Load, P_u = 2505 kN
Safe Bearing Capacity of soil = 200 kN/m^2

Area of footing $=\frac{2505}{200}$ = 12.525 m^2

Provide a square footing of 4× 4 m
Net upward pressure, $P_u=\frac{2505}{4\times4}$ = 156.56 kN/m^2 < 200kN/m^2
Hence safe.

Two way shear

Assume a uniform overall thickness of footing,
D = 600mm.

Assuming 25 mm diameter bars for main steel, effective thickness of footing,'d' is
d = 600 – 50 –12.5= 537.5 mm
The critical section for the two way shear or punching shear occurs at a distance of d/2 from the face of the column, where a and b are the sides of the column.
Hence, punching area of footing = (a + d)^2 = (0.5375 + 0.45)^2 = 0.975 m^2
Punching shear force = Factored load – (Factored upward pressure × punching area of footing)
= 2505 – (156.56×0.975)
= 2352.354kN
Perimeter of the critical section = 4 (a+d) = 4 (450+ 537.5) = 3950 mm
Therefore, from clause 31.6.3 of IS 456-2000
Nominal shear stress in punching or punching shear stress $_v$ is computed as,

$$\tau_v=\frac{2352.35\times1000}{3950\times537.5}$$
= 1.107 N/mm^2

Allowable shear stress = $k_s \times \tau_c$
Where, $k_s = (0.5 + \beta_c)$;
$\beta_c = \frac{0.23}{0.45}$ =0 .511
k_s = 0.5 + 0.511 = 1.011 so take k_s = 1
$\tau_c = 0.25 \times \sqrt{fck}$ = 1.25
Allowable shear stress = $k_s \times \tau_c$

$= 1 \times 1.25 = 1.25 N/mm^2$

Since the punching shear stress (1.107 N/mm^2) < allowable shear stress (1.25 N/mm^2),

Hence safe.

The check for assumed thickness is done and it is safe.

Hence, the assumed thickness of footing D = 600 mm is sufficient.

The effective depth for the lower layer of reinforcement, d_l =600-50-12.5=537.5mm, Effective depth for the upper layer of reinforcement, d_u =600-50-25-12.5=512.5 mm

Design for flexure

The critical section for flexure occurs at the face of the column.

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

Factored upward pressure of soil, P_u = 156.56kN/m^2

Projection of footing beyond the column face, l = (4000-450/2) = 1775 mm

Hence, bending moment at the critical section in the footing is

$M_u = \frac{P \times l \times l}{2} = \frac{156.56 \times 1.755 \times 1.755}{2}$ =241.104kN/m – m width of footing

The area of steel Ast can be determined using the following moment of resistance relation for under reinforced condition given in Annex G – 1.1 b of IS 456:2000.

The area of reinforcement is calculated using the relation:

$$M_u = 0.87 f_y A_{st} d\{1 - \left(\frac{A_{st} f_y}{bd f_{ck}}\right)\}$$

$$241.104 \times 10^6 = 0.87 \times 415 \times A_{st} \times 512.5\{1 - \left(\frac{A_{st} \times 415}{1000 \times 512.5 \times 25}\right)\}$$

Ast=1363.18 mm^2

The corresponding value of P_t = 0.267%

Hence from flexure criterion, P_t = 0.265 %

One way shear

The critical section for one way shear occurs at a distance 'd' from the face of the column

For the cantilever slab, total Shear Force along critical section considering the entire width B is

$V_u = P_u B (l - d)$

$= 156.56 \times 4 (1.775 - 0.5125) = 778.103kN$

The nominal shear stress is given by,

$$\tau_v = \frac{V_u}{bd} = \frac{778.103 \times 10^3}{4000 \times 512.12} = 0.37 N/mm^2$$

From Table 61 of SP 16, find the P_c required to have a minimum design shear strength $\tau_c = 0.36 N/mm^2$,

$\tau_v = 0.37 N/mm^2$ $f_{ck} = 30 N/mm^2$.

For P_c = 0.26% , the design shear strength , $\tau_c = .36 N/mm^2$ < $\tau_v = 0.37 N/mm^2$

hence from one way shear criterion provide P_{ct} = 0.4 %, with , τ_c =0.45 N/mm^2

Comparing P_t from flexure and one way shear criterion, provide P_t = 0.4 % (larger of the two values)

Hence,

Ast =Pt$\times b \times d/100$ =0.4$\times 1000 \times \frac{600}{100}$ =2400 mm^2

Provide 25mm dia. bars at 200mm c/c.

Therefore, A_{st} provided = 1938.95 mm^2 > A_{st} required (1363.18 mm^2). Hence O.K.

Check for development length

Sufficient development length should be available for the reinforcement from the critical section.

Here, the critical section considered for L_d is that of flexure.

The development length for 25 mm dia. bars is given by

Ld = 47Ø = 47 ×25 = 1175 mm.

Providing 60 mm side cover, the total length available from the critical section is

0.5(4000-450) = 1775mm > 1175mm.

Hence O.K

Check for bearing stress

From IS 456-2000, clause 34.4

The load is assumed to disperse from the base of column to the base of footing at rate of 2H: 1V.

Hence, the side of the area of dispersion at the bottom of footing = 450 + 2 (2 ×600) = 2850 mm.

Since this is lesser than the side of the footing (i.e., 4000 mm)

$A_1 = 4 \times 4 = 16 m^2$

The dimension of the column is 230 mm x 450mm. Hence,

$A_2 = 0.230 \times 0.45 = 0.103 m^2$

$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{16}{0.103}}$ =12.46 > 2

Hence, Limit the value of $\sqrt{\frac{A_1}{A_2}} = 2$

∴Permissible bearing stress = 0.45 ×f_{ck} $\times \sqrt{\frac{A_1}{A_2}}$ = 0.45 x 250 x 2 = 22.5 N/mm^2

Actual bearing stress $= \frac{Factored\ load}{Area\ of\ column\ at\ base} = \frac{2505 \times 10^3}{230 \times 450}$ = 24.20 N/mm^2

Since, The Actual bearing stress (22.20N/mm^2) <The Permissible bearing stress (22.50N/mm^2) according to IS clause34.4 , the design for bearing stress is satisfactory.

Reinforcement detail of footing

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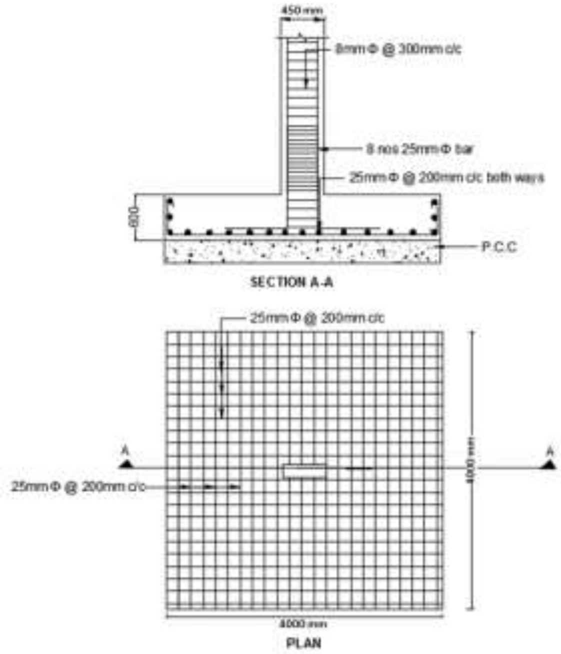

Fig. 20 Reinforcement detail of footing

8. RESULT AND CONCLUSION:

Analysis and design of an apartment building having G+10 storeys is done. Analysis is done by using the software ETABS V15.2, which proved to be premium of great potential in analysis and design of various sections. The structural elements like RCC frame, shear wall and retaining walls are also provided. As per the soil investigation report, an isolated footing is provided. The design of RCC frame members like beam and column was done using ETABS. The analysis and design was done according to standard specifications to the possible extend. The various difficulties encountered in the design process and the various constraints faced by the structural engineer in designing up to the architectural drawing were also understood.

FUTURE SCOPE:

- Dynamic analysis can also be done using ETABS.
- Slab and footing can be designed using SAFE.
- In ETABS 2016 V16.2 different types of slabs can be designed.
- The sections designed in ETABS can also be designed by conventional methods or STAAD-PRO and result can be compared.
- The irregular structures subjected to different load cases can also be analyzed and designed in ETABS.

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