SEISMIC ANALYSIS AND DESIGN OF HOSPITAL BUILDING

Submitted by

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A final report submitted in partial fulfillment of the requirements for the degree of Bachelor of Civil Engineering

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ACKNOWLEDGEMENT

The trend of getting structures analyzed scientifically for structural safety and economic reasons is getting more and more popular nowadays. Nearing graduation, after which we would qualify as full engineers, the desire to learn about such an analysis and design has motivated us for this project, entitled "**Seismic Analysis and Design of Hospital Building**".

At the outset we will like to forward our sincere thanks and gratitude to our Respected Supervisor, **Asst. Prof. Dibyashree Lohani** for providing immense guidance and support for our project. We benefited a lot in a great deal from her logical thoughts, experience, and incisive comments. We extend our heartfelt appreciation to our respected teachers of civil department associated with Nepal Engineering College for their valuable suggestions.

We also acknowledge our gratitude towards each other for such a united coordination amongst the group members during the project as well some of the other friend who helps throughout the project.

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ABSTRACT

This report has been prepared as part of project work to fulfill the requirement of course syllabus prescribed to Civil Engineering final year course. Our project group has chosen to do analysis and design of Reinforced Concrete framed building under the guidance of our dedicated supervisor and department of Civil Engineering.

Among the two category of building structure, via, load bearing and framed structures, here, in the project work the frame structure is practiced. Building frame is the three dimensional structure as space which consist of rigidly interconnected beams, slab and columns. It produces greater number of the redundancy thus reduces the moments and facilitates the even distribution of the load.

This project enabled us to acquire knowledge on proper analysis and design of building for earthquake safety including the capability of solving and tackling the field problem to somewhat. It has taught us to work in team which will surely help us in the future to come. The results of calculation are presented in tabular form and sample calculations are provided in details to reduce the bulkiness of the report. Sufficient figure and sketches have been introduced to illustrate the theories. Reference to the appropriate clauses of standard codes of practices has been made wherever necessary. It is clear that for understanding the process physically and realizing the structure behavior, manual steps by steps procedure is necessary. However due to the time constraint and to be familiar to the modern technology, the structural analysis and design part is performed using computer software "ETABS". The burden of repeated calculations in analysis has been reduced due to use of computer software. Report is focused on the design of slab, beam (primary), column (square), staircase (dog legged) and foundation (mat foundation).

List of Symbols and Abbreviations

Symbol	Description
ϕ	Diameter of Bar
τ_c	Shear Stress
γ_m	Partial Safety Factor
A_b	Area of Each Bar
A_g	Gross Area of Concrete
A_h	Horizontal Seismic Coefficient
A_{sc}	Area of Steel in Compression
A_{st}	Area of Steel
A_{sv}	Area of Stirrups
B	Width
d	Effective Depth
d′	Effective Cover
D	Overall Depth
e	Structure Eccentricity
E	Young’s Modulus of Rigidity
E_s	Modulus of Elasticity of Steel
f_{ck}	Characteristics Strength of Concrete
f_y	Characteristics Strength of Steel
f_s	Steel Stress of Service Load
h	Height of building
I	Importance Factor (For Base Shear Calculation)
I	Moment of Inertia
I_p	Polar Moment of Stiffness
k	Lateral Stiffness
L	Length of Member
L_d	Development Length
M	Bending Moment
P_c	Percentage of Compression Reinforcement
P_t	Percentage of Tension Reinforcement
Q	Design Lateral Force

R	**Response Reduction Factor**
S_a/g	**Average Response Acceleration Coefficient**
S_v	**Spacing of Each Bar**
T	**Torsional Moment due to Lateral Force**
T_a	**Fundamental Natural Period of Vibrations**
V'	**Additional Shear**
V_B	**Design Seismic Base Shear**
W	**Seismic Weight of Floor**
X_u	**Actual Depth of Neutral Axis**
X_{ul}	**Ultimate Depth of Neutral Axis**
Z	**Zone Factor**

Contents

1 INTRODUCTION

1.1 Background

With the advancement in technology of the country, the construction work has been carried out rapidly. Earthquake is one of the dominant constraints while designing the frame building in the earthquake prone zone like Nepal. Earthquake is considered to be greatest and unpredictable among all other natural disasters. Loss of lives and properties are huge and unpleasant because of worst scenario it can create. So that the designer‘s emphasis on seismic analysis of engineering structure to prevent from large intensity earthquake.

One being a designer has to deal with various structures ranging from simple ones like the curtain rods, and electric poles to more complex ones like multi-storied frame buildings, shell roofs, bridges etc. These structures are subjected to various loads like concentrated loads, uniformly distributed loads, uniformly varying loads, internal or earthquake load and dynamic forces. The structure transfers its load to the supports and ultimately to the ground. While transferring the loads, the members of the structure are subjected to internal forces like axial forces, shearing forces, bending and torsion moments. So considering all these the project of structures is designed as follows. First of all the most appropriate structural system and initial proportioning of members is done. Generally the initial drawing of architecture is referred. After that the loads are estimated based on functionality and purpose of building based on codes. Then the process of structural analysis and design evolves.

Structural Analysis deals with the prediction of performance of a given structure under stipulated loads and other external effects. The performance characteristics of interest are stresses and stress resultants such as axial forces, shear forces, bending moments, deflections and support reactions. Structural Design deals with sizing various members of the structure of actual proportion (member sizes, reinforcement details) and grades of materials required for safety and serviceability under the calculated member forces. The Indian Standard Code of Practice is thoroughly implemented for proper analysis, design and detailing with respect to safety, economy, stability, strength here in our project.

This project work has been undertaken as a partial requirement for B.E. degree in Civil Engineering. All the theoretical knowledge on analysis and design acquired on the course work are utilized with practical application. The main objective of the project is to acquaint in the practical aspects of Civil Engineering. We, being the budding engineers of tomorrow, are interested in such analysis and design of structures which will, we hope, help us in similar jobs that we might have in our hands in the future.

1.2 Objective of the project work

1. Learn the concept of lateral and vertical loadings on the building.
2. Learn the analysis for earthquake loading on the building.
3. Identification of the structural arrangement of the plan.
4. Modeling of the building for the structural analysis.
5. Detail structural analysis using ETABS 2017.
6. Sectional design of the structural components.
7. Structural detailing of the members and the system.

1.3 Scope of the project

1. The structural analysis of the building is done by ETABS 2017, for different cases of loads.
2. Design of RCC framed members, walls, mat foundation, staircase, and other by limit state method of design.
3. The project is not concerned with existing soil condition of the locality and the bearing capacity of the soil is adopted.
4. In the time of analysis and design all beams are considered as a rectangular beams instead of T-beam or L- beam.

For the design purpose IS codes are followed.

1.4 Brief description of the proposed project

Name of the Project	: Hospital Building
Location	: Gokarneshwor municipality - 8 Kathmandu.
Structure system	: RCC framed structure
No. of blocks	: 1
No. of storey	: **8** storey
Type of Slab	: Two way Slab
Type of beam	: Rectangular (600*300 mm)
Type of column	: Rectangular (600*450 mm)
Type of foundation	: Mat foundation
Type of staircase	: Dog Legged Staircase
Method of analysis	: ETABS 2017.01
Design concept	: Limit State Design
Concrete grade	: M20,M25
Reinforcement grade	: Fe500 (column) Fe415 for slab and beam
Dead load	: As per materials usage in building
Live load	: As per usage and as Indian Standard Code
Floor to Floor height	: 3 m
Plinth Area	: 561.52 m^2
Occupancy of the building	: Hospital
Topography	: Mountainous

1.5 Units

SI (i.e. metric) units are used in this report. Whenever dimensions are not mentioned in figure and drawing should be taken as mm.

1.6 Interpretation

Whenever reference to the clause of an Indian standard is made, it will be written as IS 456:2000 for structural design. Also some of clauses are written from the IS 1893:2016, SP-16 and other important factors from our book other reference books.

1.7 Detailing

Detailing are done by using code IS 13920:2016, Handbook on concrete reinforcement and detailing and reference books stated in project are extensively used.

2 METHODOLOGY

2.1 Load calculation

Load calculations are done using the IS 875:1987 as reference. The exact value of unit weights of the materials from the code is used in the calculation. The thickness of materials is taken as per design requirement.

2.2 Gravity load calculation

There are three types of loads for which are considered in this analysis.

1. Dead load
2. Live load
3. Lateral load

2.2.1 Dead load

Dead load consists of the self-weight of the column, beam, slab and wall. Dimensions of column, beam, and slab are taken from preliminary design. For wall load, thickness of wall is taken from plan and deduction of opening is done according to the average size of opening in the outer and inner walls and deduction is not made in solid wall. In the case of the partition wall in the middle of slab, the total weight of the wall is calculated as uniformly distributed load and assign over the nearest beam.

2.2.2 Live load

Live load is taken from IS 875: 1987(Part II). The magnitude of the live load depends upon the occupancy of the building. In the case of different live load in one panel of slab, highest value of the live load is taken for the panel.

2.2.3 Lateral load

Lateral load acting in the building is earthquake/seismic load and wind load. Only earthquake load analysis is carried out in the project. For the analysis of earthquake load, following methods is generally carried out:

1. Seismic Coefficient method (Static)
2. Response Spectrum method (Dynamic)

In this project we use **seismic coefficient method** (linear static method).

2.3 Loading pattern

The loading is applied to the slab elements directly. The total load (DL and LL) on staircase is equally distributed on both supporting beam and the load is converted to UDL. The load on slab is taken as per the requirement stated in IS875:1987(Part I & II).

Dead loads are computed from the dimensions of the structural member such as walls, beams, slabs, etc. and their material densities confirming to IS 875 (Part I). Similarly, live loads are to be chosen from IS 875 (part II) for various occupancies where required. The uniformly distributed dead and live load acting on the slab are transferred to the beams holding the slab. The slab load is distributed on the floor beams as shown in figure below. The smaller beam holds the triangular load and the longer beams hold the trapezoidal load as shown in figure. The beam element also resists the self-weight and the wall load including all the finish loads on wall such as external and internal plaster.

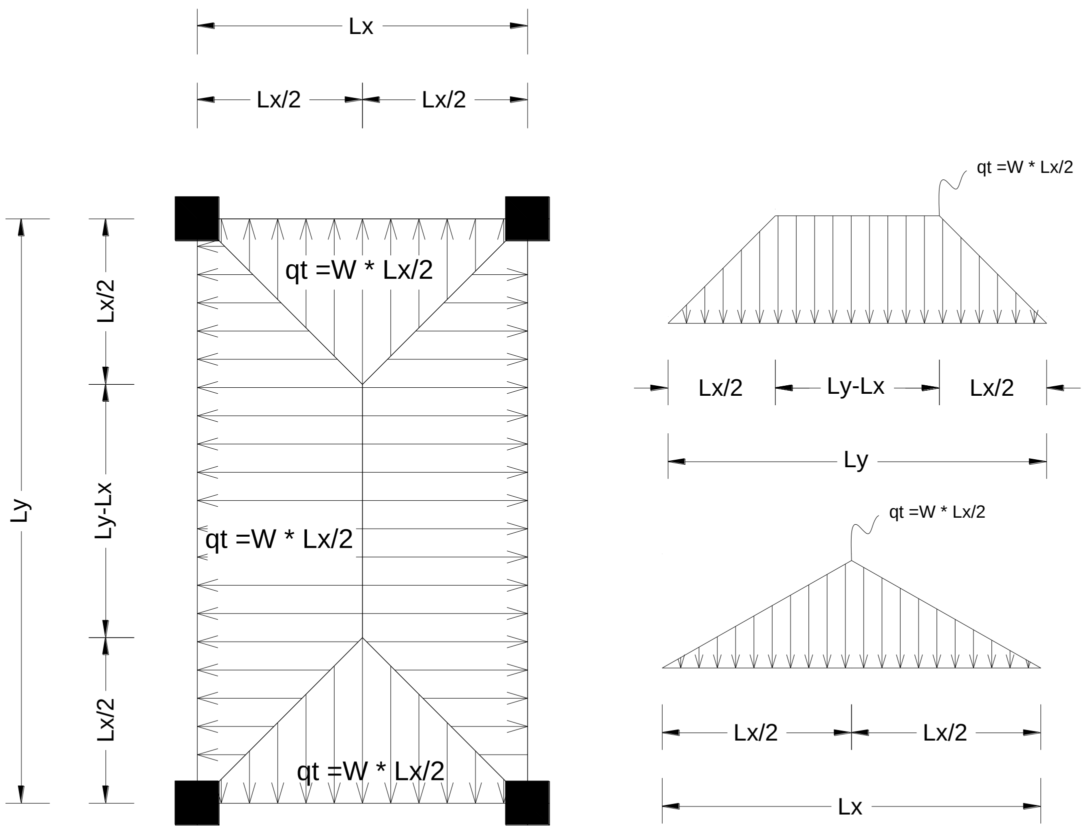


Fig. Loading pattern in beam

2.4 Methods and tools for analysis

1. Creating grid/ model
2. Defining
   a) Material
   b) Section (beam, column, slab)
   c) Load cases
   d) Load pattern
   e) Load combination
3. Assigning
   a) Section
   b) Load
4. Analyze the structure

   Earthquake loads are calculated using seismic coefficient method.
5. Design of structural elements
6. Limit state design using above mentioned codes.

2.5 Design method

We adopt limit state design method for design.

2.5.1 Limit state method

It uses the concept of the probability and based on the application of method of statistic to the variation that occurs in the practice in the loads acting in the structures or in the strength of material.

The structures may reach a condition at which it becomes unfit for use for one of many reasons e.g. collapse, excessive deflection, cracking, etc. and each of these conditions is referred to a limit state condition. The aim of limit state design is to achieve an acceptable probability that the structure will not become unserviceable in its life time for the use of which it has been intended i.e. it will not reach a limit state. It means the structure should be able to withstand safely all loads that are liable to act on it throughout its life and it would satisfy the limitations of deflection and cracking.

2.5.2 Assumptions for flexural member

1. Plane sections normal to the axis of the member remain plane after bending.
2. The maximum strain in concrete at the outermost compression fiber is 0.0035.
3. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoidal, parabola or any other shape which results in prediction of strength in substantial agreement with the result of test. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γm = 1.5 shall be applied in addition to this.
4. The tensile strength of concrete is ignored.
5. The design stresses in reinforcement are derived from representative stress-strain curve for the type of steel used. For the design purposes the partial safety factor γm = 1.15 shall be applied.
6. The maximum strain in the tension reinforcement in the section at failure shall not be less than: $\frac{0.87fy}{Es} + 0.002$

Where,

fy= characteristic strength of steel

Es = modulus of elasticity of steel

2.5.3 Limit state of collapse for compression

Assumption:

In addition to the assumptions given above from i) to v), the following shall be assumed:

1. The maximum compressive strain in concrete in axial compression is taken as 0.002.

2. The maximum compressive strain at highly compressed extreme fibre in concrete subjected to axial compressive and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fibre.
3. The limiting values of the depth of neutral axis for different grades of steel based on the assumptions are as follows:

Table 1: Limiting Value of Neutral Axis

F_y	$X_{u,max}$
250	0.53
415	0.48
500	0.46

2.5.4 Limit state of serviceability

This state corresponds to development of excessive deformation and is used for checking members in which magnitude of deformation may limit the use of the structure or its component. This limit may corresponds to

1. Deflection
2. Cracking
3. Vibration

The choice of the degree of reliability should be taken into account the possible consequences of exceeding the limit state of collapse which may be classified according to

1. Risk to life negligible and economic consequences small or negligible.
2. Risk to life exists and / or economic consequences considerable and
3. Risk to life great and / or economic consequences also great.

2.5.5 Structural system

The system has been analysed as 3D space frame. Element stresses in beams and columns are calculated by ETABS with provision of special joint frame system. Raft foundation has been constructed looking towards its effectiveness in the construction of building. Due to complexity of the structure, variation of load is high thus the mat foundation is best foundation for the structural system in comparison of other on context of safety, economy and stability.

2.5.6 Analysis of building

For the static analysis of our building we use the structural analysis program ETABS that has special option for modelling horizontal rigid floor diaphragm system. We have adopted seismic coefficient method in ETABS analysis.

2.5.7 Load cases

Load cases are independent loading for which the structure is explicitly analysed. Earthquake forces occur in random fashion in all directions. For building whose lateral load resisting elements are oriented in two principal directions. It is usually sufficient to analyze in these two principal directions (X and Y direction) separately one at a time. Thus the load cases adopted are as follows.

1. **Dead load(DL)**
2. **Live load(LL)**
3. **Earthquake load in X direction (EQX)**
4. **Earthquake load in Y direction (EQY)**

Following load combination are adopted for design

a) 1.5(DL+LL)

b) 1.2(DL+LL±EQX)

c) 1.2(DL+LL±EQY)

d) 1.5(DL±EQX)

e) 1.5(DL±EQY)

f) 0.9(DL±EQX)

g) 0.9(DL±EQY)

2.6 Earthquake resistant design philosophy

The primary objective of earthquake resistant design is to prevent building collapse during earthquakes thus minimizing the risk of death or injury to people in or around those buildings.

Engineers do not attempt to make earthquake proof buildings that will not get damaged even during the rare but strong earthquake; such buildings will be too robust and also too expensive. Instead the engineering intention is to make buildings earthquake-resistant; such buildings resist the effects of ground shaking, although they may get damaged severely but would not collapse during the strong earthquake. Thus, safety of people and contents is assured in earthquake-resistant buildings, and thereby a disaster is avoided. This is a major objective of seismic design codes throughout the world.

Design Philosophy

1. Under minor but frequent shaking, the main members of the buildings that carry vertical and horizontal forces should not be damaged; however buildings parts that do not carry load may sustain repairable damage.
2. Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts that do not carry load may sustain repairable damage.
3. Under strong but rare shaking, the main members may sustain severe damage, but the building should not collapse.

The earthquake resistant design process involves various factors to be considered Some of them are sort listed below:

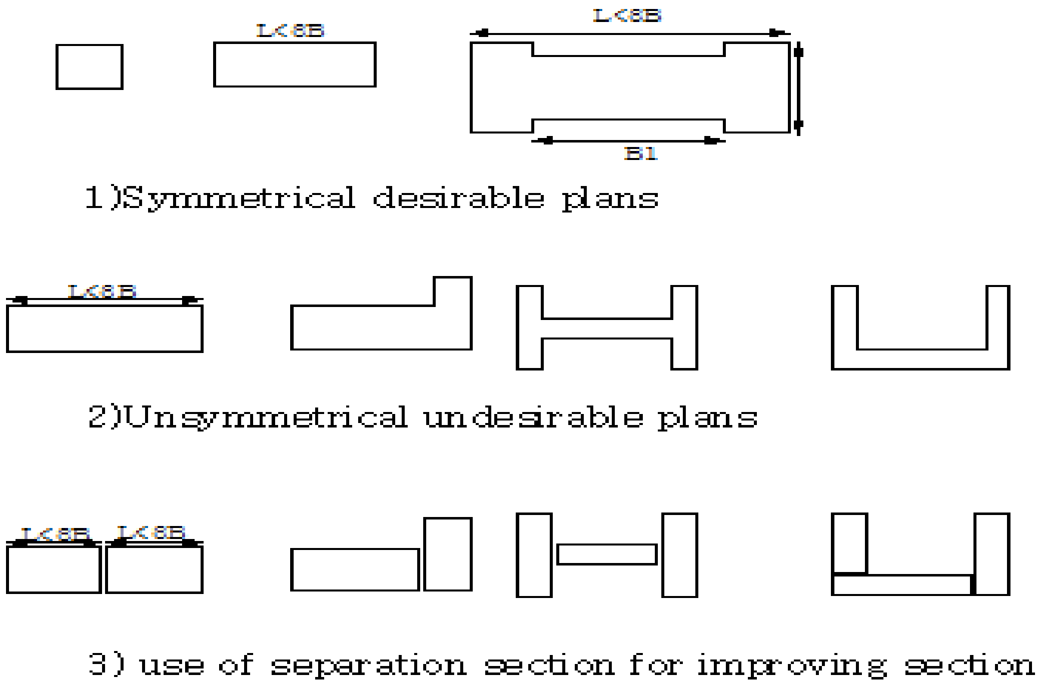
Configuration

1. **Symmetry:**

   The building as a whole or its various blocks should be kept symmetrical about both the axes. Asymmetry leads to torsion during earthquakes and is dangerous.
2. **Regularity:**

Simple rectangular shapes behave better in an earthquake than shapes with many projections. Torsional effects of ground motion are pronounced in long narrow rectangular blocks. Therefore, it is desirable to restrict the length of a block to three times its width. If longer lengths are required two separate blocks with sufficient separation in between should be provided.

3. Separate Buildings for Different Functions



2.6.1 Connection

Proper selection of the material in proper ratio is needed for the construction of earthquake resistant design. Material properties should match the requirement of earthquake resistant concept. This involves various material properties.

Ductility is the major parameter need to be considered in the building performance during earthquake. Earthquake resistant buildings, particularly their main elements, need

to be built with ductility in them. Such buildings have the ability to sway back-and-forth during an earthquake, and to withstand the earthquake effects with some damage, but without collapse.

Thus, a necessary requirement for good earthquake-resistant design is to have sufficient ductile materials at points of tensile stresses.

Again base isolation of the structure from the ground motions which actually impose the forces on the structure can also be done. For reduction of the coefficient of friction between the structure and its foundation, one suggested technique is to place two layers of good quality plastic or flexible connection between the structure and its foundation.

2.6.2 Construction quality

Though the connection and configuration of the structure is properly planned, but if the execution of the work is not accomplished using the planned format or specification then it may be difficult to achieve earthquake resistant structure. Proper supervision of the work by expert is required.

3 PRELIMINARY DESIGN

Preliminary design is carried out to estimate approximate size of the structural members before analysis of structure. Grid diagram is the basis factor for analysis in both approximate, extract method, and is presented below. presented below.

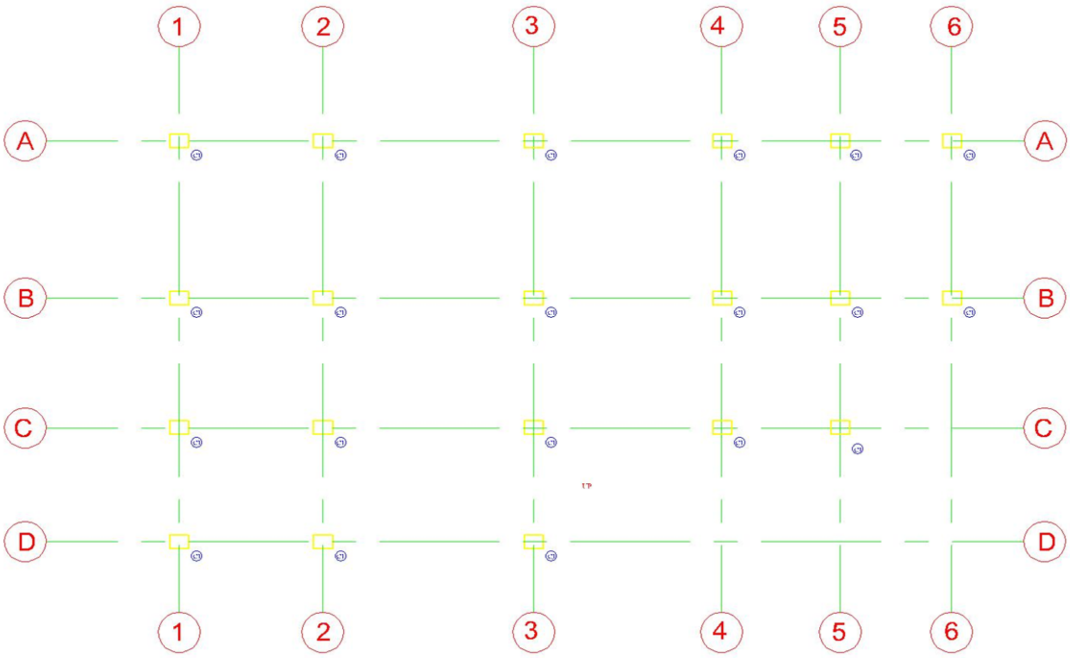


Fig. 4.1: Grid Plan

3.1 Preliminary design of slab

Material used:

Steel Fe415 and Concrete M20

% of steel (0.1% to 0.4%) = 0.3%

From clause 23.2.1 (IS456-2000)

Span /depth $\leq$ $\alpha\beta\gamma\delta\lambda$

α = 26

β = 1

δ =

1

Now for the modification factor 'γ' for tension reinforcement

F_s $= 0.58 * fy * \frac{\text{Ast provided}}{\text{Ast required}}$

$= 0.58 * 415\ N/mm^2 * 1$

$=240.7$

Now from the graph of Modification factor for tension reinforcement

Γ =1.65

For critical **slab S2** (6.6m * 5.175m)

Shortest span = 5.175 m = 5175mm

Effective depth = span/42.9

=5175/42.9

= 120.62 mm

Providing 10 mm dia bar and 15 mm clear cover

Overall depth D = 120.62+ 5 + 15

= 140.62 mm

Hence adopt overall depth (**D**) of slab = **150 mm**

Effective depth d = D-20 = 130 mm

Depth calculation for all slab.

S.N.	Slab	Short Span (lx) Mm	effective depth (d) mm	Overall depth (D) mm	adopt depth (D) mm
1	S1,S2	3700	86.77	106.77	150
2	S3,S5	5175	121.36	141.36	150
3	S4, S6, S8	4276	100.28	120.28	150
4	S7	4500	105.53	125.53	150
5	S9	3750	87.94	107.94	150

So the final overall depth (D) of the slab is taken as **150mm.**

3.2 Preliminary design of beam

i) For Numeric grid:

For beam span/depth =12 to 15

F_{ck} = M20 = 20N/mm^2

F_y = Fe500 = 500N/mm^2

MLargest span = 6600 mm

Effective Depth= span/13

= 6600/13

= 507.69 mm

Assume diameter of main bar Φ = 20mm and clear cover = 25mm

For total depth of beam (D) = d + dia/2+ c/c

=507.69+(20/2)+25

= 542.69 mm

hence Adopt 600 mm

Again,

Width of beam = (1/2 to 2/3 of depth)

Taking B= (1/2)*d

=(600/2)

=300 mm

Size of beam along Numeric grid **= b * D**

= 300 * 600

ii) For Alphabetic Grid A-B

F_{ck} = M20 = 20N/mm^2

F_y = Fe500 = 500N/mm^2

largest span = 5175 mm

Efective depth = L/12 = 5175/15 = 431.33 mm

Assume diameter of main bar Φ =

20mm

And clear cover = 25mm

For total depth of beam (D) = d + dia/2+ c/c

=431.25+(20/2)+25

= 466.25

Let us Adopt 600 mm

Again,

Width of beam = (1/2 to 2/3 of depth)

Taking B= (1/2)*d

=(600/2)

=300 mm

Size of beam along alphabetic grid **= b * D**

= 300 * 600 mm

3.3 Preliminary design of column

Material used

Grade of concrete (f_{ck}) = M25 =25 N/mm^2

Grade of steel (f_y) = Fe500 =500 N/mm^2

Beam size = 600*300 mm

No of floor = 8

Calculation of Column: B3

Most critical due to surrounded by long span floor slab.

Loading Area = (3.3+2.95) + (2.5875+2.1375)

= 29.53125 mm^2

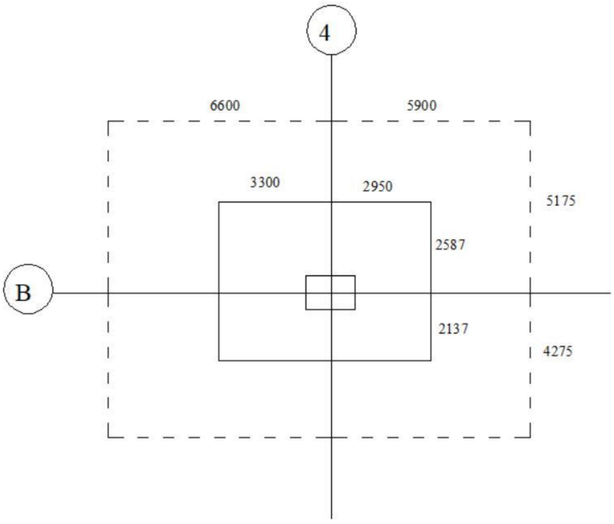


Fig.: Critical Column (B-3)

Load Calculation:

Dead Load:

Beam = 25*0.45*0.3*{3.3+2.95+2.5875+2.1375}

= 37.040625 KN.

Slab = 25*D*Area

= 25*0.15*{(3.3+2.95) + (2.1375+2.5875)}

= 110.74 KN.

Column = 25*b*D*h

= 25*0.6*0.45*3
= 20.25 KN

Floor finish = Intensity * Area = 29.531 KN

Total dead load = 37.041+110.74+20.25+29.53+29.53

=227.10 KN.

Live Load:

As IS 875, part 2 Total live load for each floor = 3*Area =3*29.533 = 88.594KN

Total live load at roof =1.5*29.531= 29.531 KN

S.No.	Floor	Total Dead Load (KN)	Total Live Load (KN)	Total Load (DL+LL) (KN)
1.	First	227.10	88.594	315.694
2.	Second	227.10	88.594	315.694
3.	Third	227.10	88.594	315.694
4.	Fourth	227.10	88.594	315.694
5.	Fifth	227.10	88.594	315.694
6.	Sixth	227.10	88.594	315.694
7.	Seventh	227.10	88.594	315.694
8.	Roof	227.10	44.296	271.396

Total Load = 2481.11 KN.

Total load after Earthquake consideration = 1.3*2481.254

= 3225.630 KN.

Factored load = 1.5*3225.630

= 4838.445 KN.

Total factored load = 4838.445 KN**.**

Pu=0.45*fck*Ac+0.67*fy*Asc

4838.445*1000=0.45*25*0.97*Ag+0.67*500*0.03*Ag

On solving we get

Ag=230814.313 mm^2

Taking one side of column as 600mm

B=(Ag/b)=(230814.313/600)

= 394.69 mm

Adopt 450 mm i.e Column Size =600mm*450mm

4 LATERAL ANALYSIS

4.1 Introduction of load lateral

Seismic weight is the total dead load plus appropriate amount of specified imposed load. While computing the seismic load weight of each floor, the weight of columns and walls in any story shall be equally distributed to the floors above and below the storey. The seismic weight of the whole building is the sum of the seismic weights of all the floors. It has been calculated according to IS: 1893(Part I) – 2002.

IS 1893(Part I) – 2002 states that for the calculation of the design seismic forces of the structure the imposed load on roof need not be considered.

4.2 Calculation of self- weights of elements

Self weight of Elements

a) Self weight of beam

Cross section of beam=300mm*600mm

Slab thickness=150mm

Self weight of beam excluding slab thickness=0.3*(0.6-0.15)*25

=3.37KN/m

b) Self weight of floor slab =0.15*25

=3.75KN/m^2

c) Column

Size of column=600mm*450mm

Self weight=0.6*0.45*25

=6.75KN/m

d) Main outside brick wall (230 thick)

i) Without opening

Height of wall (H)=3-0.6

=2.4m

Dead load of wall=2.4*0.23*18.85

=10.405KN/m

Using 12mm cement plaster on both side =2*24*0.012*20.4

=1.175KN/m

Total dead load of wall with cement plaster =10.405+1.175

=11.580KN/m

ii) With opening

Assuming 35% wall is covered by spacing then,

Total dead load of wall with opening =0.65*11.580

=7.527KN/m

e) Partition wall(110mm thick)

iii) Without opening

Height of wall(H)=

Dead load of wall =2.4*0.11*18.85

=4.976KN/m

Using 12mm cement plaster on both side =2*24*0.012*20.4

=1.175KN/m

Total dead load of wall with cement plaster =4.976+1.175

=6.151KN/m

iv) With opening

Assuming 35% of wall is covering by spacing then,

Total dead load of partition wall with opening=0.65*6.151

=3.998KN/m

f) Terrace parapet wall (1m)

Dead load of brick wall =0.23*(0.9-0.075)*18.85

=3.576KN/m

12mm thick two side paster=2*0.012*(0.9-0.075)*20.4

=0.403KN/M

RCC coping at top of wall=0.350*0.075*25

=0.656KN/M

Total dead load of parapet wall=3.576+0.403+0.656

=4.535KN/M

g) Landing Beam

Cross section=230*300mm

Self weight=0.23*0.3*25

=1.1752KN/M

4.3 Wall length calculation

1) Basement floor

   230mm wall without opening =30.10m

   230mm with opening =50.050m

   110mm wall without opening =15.825m

   110mm wall with opening =8.550m

2) Ground Floor

   230mm wall without opening =(4500-600+6600-600+4275+5175)

   =19.35m

   230mm wall with opening=(3300+3825+4725+4500-600+6600-600+5900-600+3700-600+3500-600+4725+4725+3825+3700-600+5900-600+3300)

   =60.925mm

   110mm wall without opening =12.525m

   =(4500-600+4500-600+5175-450)

   110mm wall with opening =4.550mm

   =(5175-450+6600-600+4275-450)

3) Remaining 1st ,2nd ,3rd, 4th ,5th ,6th

   230mm thick wall without opening =13.725m

   =(4500-600+6600-600+4275-450)

   230mm thick wall with opening =60.925m

   110mm wall without opening =(4275-400+4500-600)

   = 7.775m

   110m wall with opening =(5175-400)*3+(6600-600+5900-600+3700-600)*2(4725-400)*2+(6600-600)

   =57.775m

   Parapet wall length =4*4.565

   =18.26m

4) Roof Floor

230mm wall without opening =17.25m

=(4500-600)*2+(5175-400)*2

230mm wall with opening =12.4m

Parapet wall =54.375m

4.4 Lumped Mass Calculation

Beam length = 121.675m

Landing beam length =6.2m

Floor slab area=238.297 m^2

Area of cantileaver slab = 2.422 m^2

Length of column = 3-0.15
=2.85m

Stair 1

Volume of stair=1/2*0.25*0.15*1.5*18+(2.70*1.5*0.18*)*2+(1.5*3.5*0.18)+(1.425*3.5*0.18)
=3.807 m^3

Dead load on stair-1=3.807*25
=95.175KN

Live load =0.5*16.98 *4
=33.975 KN

Stair 2

Volume of stair=
½*0.25*0.15*1.4*18+2.70*1.4*0.180*2+(1.4*3.730*0.180)+(1.4*0.850*0.180)
=2.987m^3

Dead load on stair-2=2.987m^3*25
=74.68KN

Live load =0.5*13.88m^2 4kn/m^2
=27.76KN

Total load=102.44KN

Ground Floor

1. Beam=121*3.375
=456.28KN
2. Landing beam = 6.2*1.725
=10.695KN

3. Slab=238.297*3.75
   =893.613KN
4. Column=20*2.85*6.75
   =384.75KN
5. Wall=(30..10*11.85)+(50.50*7.527)+(15.825*6.151)+(8.55*3.998)}/2+(19.35*11.850)+(6 0.925*7.527)+(12.252*6.151)+(14.550*3.998)/2
   =425.71+411.54
   =837.256KN
6. Floor finish=238.2997m^2*1KN/m^2
   =238.297KN
7. Live load=0.25*3*260.782
   =195.586KN
8. Stair case-1=129.15KN
9. Staircase-2=102.44KN
   TOTAL MASS OF GROUND FLOOR=3248.067KN

First Floor

1. Beam=121*3.375
   =456.28KN
2. Landing= 6.2*1.725
   =10.695KN
3. Slab=238.297*3.75
   =893.613KN
4. Cantilever slab=9.082KN
5. Column=20*2.85*6.75
   =384.75KN
6. Wall=(19.35*11.85)+(60.925*7.527)+(12.525*6.151)+(57.775*3.998)}/2+((13.725*11.850)+(60.325*7.527)+(7.775*6.151)+(57.775*3.998)}/2+18.26*4.535
   =411.54+450.016+82.809
   =944.365KN
7. Floor finish=238.2997m^2*1KN/m^2
   =238.297KN
8. Live load=0.25*3*260.782
   =195.586KN
9. Staircase-1=129.15KN
10. Staircase-2=102.44KN
    Total mass of first floor=3364.258

Second,Third,Fourth,Fifth,Sixth Floor

1. Beam=121*3.375
   =456.28KN
2. Landing= 6.2*1.725
   =10.695KN
3. Slab=238.297*3.75

=893.613KN

4. Cantilever slab=9.082KN
5. Column=20*2.85*6.75
   =384.75KN
6. Wall=(13.725*11.85)+(60.925*7.527)+(7.775*6.151)+(57.775*3.998) +18.26*4.535
   =900.02+82.809
   =982.841KN
7. Floor finish=238.2997m^2*1KN/m^2
   =238.297KN
8. Live load=0.25*3*260.782
   =195.586KN
9. Staircase-1=129.15KN
10. Staircase-2=102.44KN

Total mass of Second,third,fourth,fifth,sixth floor=3402.735KN

Roof Floor

1. Beam=121*3.375
   =456.28KN
2. Landing beam = (6.2*1.725)/2
   =5.347KN
3. Slab=238.297*3.75
   =893.613KN
4. Cantilever slab=9.082KN
5. Column=(20*2.85*6.75)/2+(8*2.85*6.75)/2
   =269.325KN
6. Wall=(13.725*11.85)+(60.925*7.527)+(7.775*6.151)+(57.775*3.998)/2+ {(17.25*11.85)+(12.4*7.527)}/2+54.375*4.535
   =450.016+148.87+246.59
   =982.841KN
7. Floor finish=238.2997m^2*1KN/m^2
   =238.297KN
8. Live load=0
9. Staircase-1=129.15KN/2
   =64.57KN
10. Staircase-2=102.44KN/2
   =51.22KN

Total mass of first floor=2833.21KN

Staircase cover

1. Beam=29.65*3.375

   =100.068KN
2. Column=(8*2.85*6.75)/2

=76.95KN

3. Slab=51.84*3.75

=194.4KN

4. Live load=0

Total mass of first floor=371.418KN

FLOOR	LUMPED MASS
Staircase cover	371.418KN
Roof floor	2833.21KN
Sixth floor	3402.735KN
Fifth floor	3402.735KN
Fourth floor	3402.735KN
Third floor	3402.735KN
Second floor	3402.735KN
First floor	3364.258KN
Ground floor	3248.067KN
Total	26830.628KN

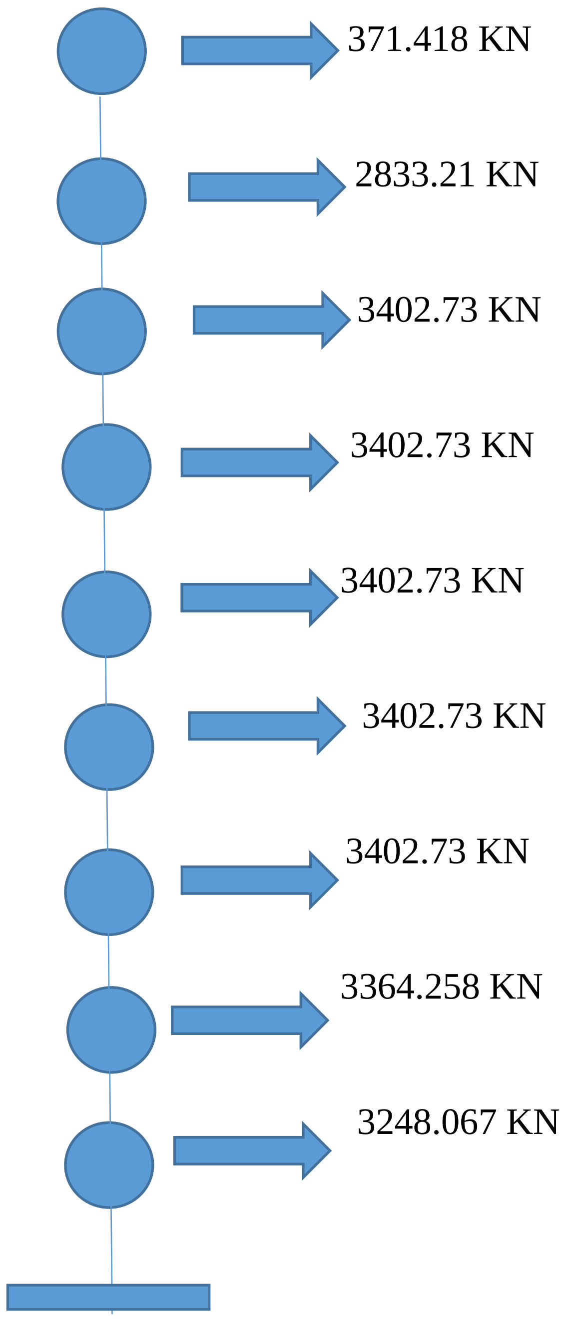


FIG: LUMPED MASS IN EACH FLOOR

4.5 Calculation of base shear

According to IS 1893 (Part I): 2002 Cl. No. 7.7.1 the design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$ (For relatively flexible structure) ………… eq. 2.1

Where,

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor I measured from base

n = No. of storey in the building

According to IS 1893 (Part I) : 2002 Cl. No. 7.5.3 the total design lateral force or design seismic base shear (V_B) along any principle direction is given by

$V_B = A_h \text{ x } W$………… eq. 2.2

Where,

W = Seismic weight of the building

A_h = design horizontal acceleration spectrum value.

According to IS 1893 (Part I): 2002 Cl. No. 6.4.2 the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\, I\, S_a}{2\, R\, g}$$ ……… eq. 2.3

Where,

Z = Zone factor given by IS 1893 (Part I): 2002 Table 2, Here for Zone V, Z = 0.36

I = Importance Factor, I = 1.5 for hospital buildings

R = Response reduction factor given by IS 1893 (Part I): 2002 Table 7, R = 5.0

S_a/g = Average response acceleration coefficient which depends on Fundamental natural period of vibration (T_a).

According to IS 1893 (Part I): 2002 Cl. No. 7.4.2

$T_a = \frac{0.09h}{\sqrt{d}}$ Seconds ………… eq. 2.4

Where,

h = height of building in m, h = 27 m

d_x = 24.8m

d_y = 13.2m

$$T_{ax} = \frac{0.09h}{\sqrt{dx}} = 0.48\text{Secs}$$

$$T_{ay} = \frac{0.09h}{\sqrt{dy}} = 0.66\text{Secs}$$

For T_{ax} = 0.48Secs and T_{ay} =0.66Secs and medium soil type Sa/g = 2.

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

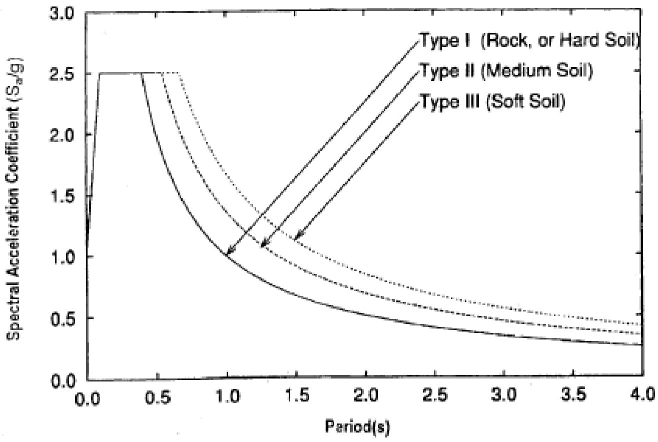


Fig : Chart showing Spectral acceleration coef. Vs Time period

Now,

$$A_{hx} = \frac{0.36x1.5x2.5}{2x5} \quad = 0.135$$

$$A_{hy} = \frac{0.36x1.5x2.06}{2x5} \quad = 0.111$$

Therefore,

Base Shear along X axis= A_{hx} x W= 0.135*26830.628=3622.13KN

Base shear along Y axis= A_{hy} x W=0.111*26830.628=2978.19KKN

$Q_i = V_B * \frac{W_i H^2}{\sum_{j=1}^{n} W_j H^2}$

S.N	**Storey**	**H_i(M)**	**W_i(KN)**	**Hi^2 Wi**	**Q_i**
1	Stair cover	27	371.418	270763.72	165.76
2	Roof floor	24	2833.21	1631928.96	999.06
3	6^{th}	21	3402.735	1500606.13	918.66
4	5^{th}	18	3402.735	1102486.14	674.94
5	4^{th}	15	3402.735	765615.37	468.71
6	3^{rd}	12	3402.735	489993.84	299.97
7	2^{nd}	9	3402.735	275621.53	168.73
8	1^{st}	6	3364.258	121113.288	74.14
9	GF	3	3248.067	29232.60	17.89
10	Basement	0	0	0	0
Total				**5916597.86**	**3622.1**

5 STRUCTURAL ANALYSIS

For the structural analysis of the structure, there considered four load cases which are mentioned below:

1. Dead load(DL)
2. Live load(LL)
3. Earthquake load in X direction (EQX)
4. Earthquake load in Y direction (EQY)

5.1 Salient features of ETAB17 which is used for the analysis.

ETABS is a programme for linear, nonlinear, static and dynamic analysis, and the design of building systems. From an analytical standpoint, multistorey buildings constitute a very special class of structures and therefore deserve special treatment. The concept of special programmes for building type structures was introduced over 40 years ago and resulted in the development of the TABS series of computer programmes.

5.2 Features and Benefits of ETABS

- The input, output and numerical solution techniques of ETABS are specifically designed to take advantage of the unique physical and numerical characteristics associated with building type structures. As a result, this analysis and design tool expedites data preparation, output interpretation and execution throughput.
- The need for special purpose programmes has never been more evident as Structural Engineers put non-linear dynamic analysis into practice and use the greater computer power available today to create larger analytical models.
- Over the past two decades, ETABS has numerous mega-projects to its credit and has established itself as the standard of the industry. ETABS software is clearly recognised as the most practical and efficient tool for the static and dynamic analysis of multistorey frame and shear wall buildings.

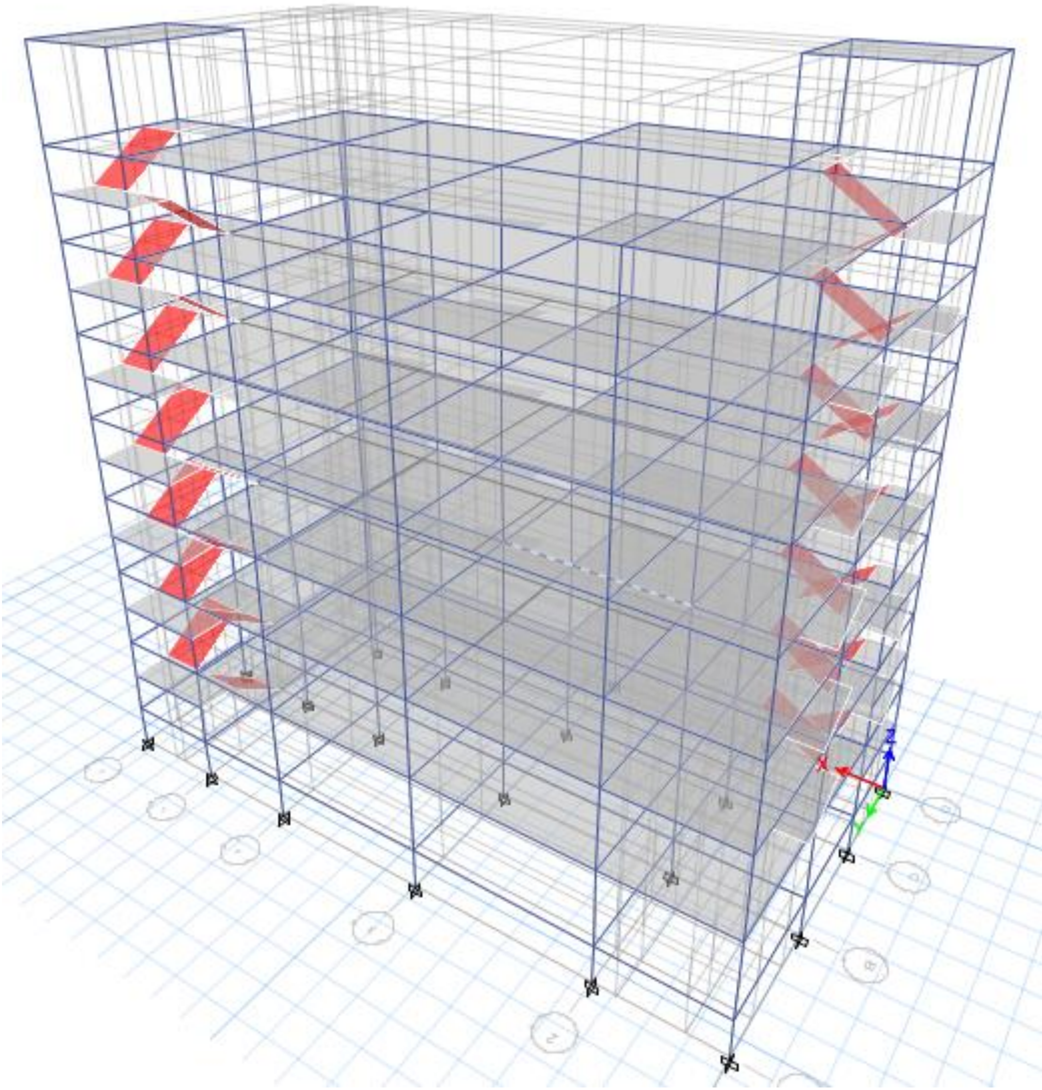


FIG: 3D MODEL

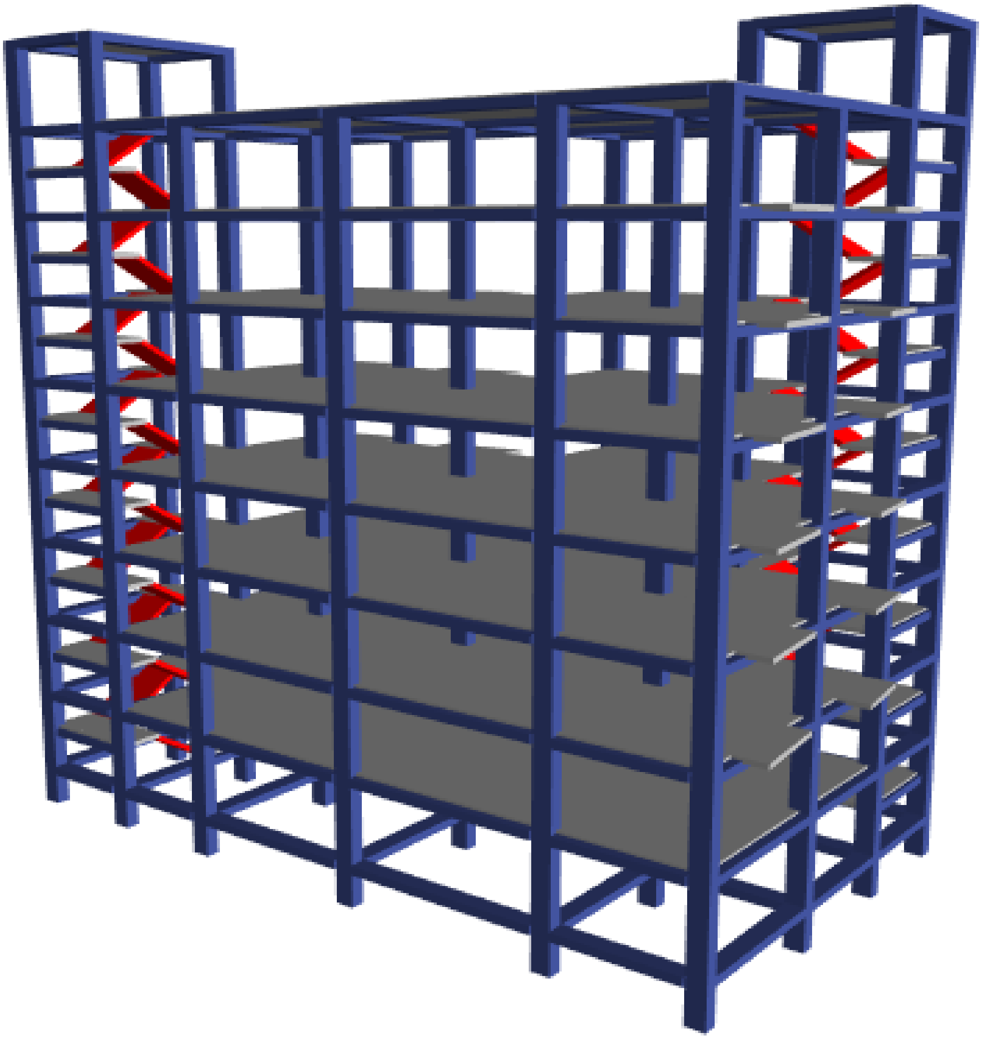


FIG: RANDER VIEW

STRUCTURAL DESIGN

Design of structural elements

The design section is the most important part. The design of the structural elements should be done for durability, construction and use in entire service life of the structure. The realization of design objectives requires compliance with clearly defined standards for materials, production, workmanship, and also maintenance and use of structure in service. This chapter includes all the design process of sample calculation for a single element as slab, beam, column, staircase, basement wall, lift wall and mat foundation.

1. Design of slab
2. Design of beam
3. Design of column
4. Design of staircase
5. Design of mat foundation
6. Design of lift wall

5.3 Design of slab

Slabs are the plate elements forming floors and roofs ofbuilding and carrying distributed loads primarily by flexure, which may be supported by beams or walls and may be used as the flange of T or L-beam. Slab may be simply supported or continuous over one or more supports and is classified according to the manner of support as: one way slabs spanning in one direction, two-way slab spanning in directions, circular slabs, and grid floor slabs resting directly on columns with no beams and grid floor and ribbed slabs.

Slabs are designed by using the same theories of bending and shear as are used for beams. The following methods of analysis are available:

1. Elastic analysis-idealization into strips or beams.
2. Semi empirical coefficients as given in the code, and
3. Yield line theory

Slabs are analyzed and designed as having a unit width that is 1 m wide strips. Compression reinforcement is used only in exceptional cases in a slab. Shear stresses are usually very low and shear reinforcements is never provided in slabs .It is preferred to increase the depth of a slab and hence reduce the shear stress rather than provide shear reinforcement. Temperature reinforcement is invariably provided at right angles to the main longitudinal reinforcement in a slab. There are two types of slabs described as follow:

1. One way slab

One –way slabs are those in which the length is more than twice the breadth. Aone-way slab may be simply supported or continuous, or can be analyzed in a manner similar to that for continuous beam.

2. Two way slab

When slabs are supported on the four sides, two-way spanning actions occur. Such slabs may be simply supported or continuous or any other sides. The deflection and bending moments in a two-way slab are considerably reduced as compared to those in one- way slab .In a square slab, the two-way action is equal in each direction.

A slab may have its few or all edges restrained. The degree of restraints may vary depending whether it is continuous over its supports or cast monolithically with its supporting beams .A hogging or negative moment will develop in the top face of the slab at the supported sides. In these slabs the corners are prevented from lifting and provision is made for torsion. The maximum moments M_x and M_y at mid span on strips of unit width for spans l_x,$l_y$are given by: $M_x = \beta_x\, w\, l_x^2$

$M_y = \beta_y\, w\, l_x^2$

Where,

βx,β_y= moment coefficients that can be obtained from table 26 of IS 456:2000 for different arrangements of slabs.

Slab panel	Descriptions
S1, S8,	One short edge discontinuous
S2, S7, S9	Two adjacent Edge discontinuous
S3, S4, S5	One long edge discontinuous
S6	Interior panels

Materials used for slab

Steel Fe415

Concrete M20

Calculation of L_y/L_x for all slabs

S.N.	SLAB	Lx	Ly	Ly/Lx	Remarks
1	S1	3700	5175	1.3986	Ly/Lx <2
2	S2	3700	4275	1.1554	
3	S3	5175	5900	1.1401	
4	S4	4275	5900	1.3801	
5	S5	5175	6600	1.2754	
6	S6	4275	6600	1.5439	
7	S7	4500	5175	1.1500	
8	S8	4275	4500	1.0526	
9	S9	3750	6600	1.7600	

Since L_y/L_x<2 for all panels. Hence the slab is two-way slab and it is design according to as two-way slab

a) Load calculation

Dead Load

1) 150 mm thick RCC slab = 25*0.15

$= 3.75\ KN/m^2$

2) 25 mm thick cement plaster and screeding= 20.4*0.025

$= 0.50\ KN/m^2$

3) 25 mm thick marble= 26.7*0.025

$= 0.6675\ KN/m^2$

Total Dead Load= 4.867 KN/m^2

For Panel S1 with washroom, Live load= 2 KN/m^2

Total load = 6.867 KN/m^2

Total factored load =10.30 KN/m^2

For panel S6 with storage, Live load = 5 KN/m^2

Total Load = 8.67 KN/m^2

Total factored load =13.30 KN/m^2

For remaining panel, Live load = 3 KN/m^2

Total Load=7.867 3 KN/m^2

Total factored load =11.801 KN/m^2

b) **Bending moment coefficient and bending moments for all slabs**

S.N.	Types of panel and moment Considered	A_x	α_y	W KN/m2	M_x KN/m2	M_y KN/m2
1	S1			10.300		
	Negative moments at continuous edge	0.0549	0.037		8.2515	5.5611
	positive moments at mid span	0.0410	0.028		6.1623	4.2084
2	S2			11.801		
	Negative moments at continuous edge	0.0568	0.047		9.1764	7.5931
	positive moments at mid span	0.0427	0.035		6.8984	5.6544
3	S3			11.801		
	Negative moments at continuous edge	0.0472	0.037		14.9170	11.6934
	positive moments at mid span	0.0354	0.028		11.1878	8.8491
4	S4			11.801		
	Negative moments at continuous edge	0.0618	0.037		13.3285	7.9798
	positive moments at mid span	0.0464	0.028		10.0071	6.0388
5	S5			11.801		
	Negative moments at continuous edge	0.0557	0.037		17.6033	11.6934
	positive moments at mid span	0.0427	0.028		13.4948	8.8491
6	S6			13.301		
	Negative moments at continuous edge	0.0540	0.032		13.1265	7.7787
	positive moments at mid span	0.0416	0.024		10.1123	5.8340
7	S7			11.801		
	Negative moments at continuous edge	0.0565	0.047		13.5018	11.2316
	positive moments at mid span	0.0425	0.035		10.1562	8.3640
8	S8			11.801		
	Negative moments at continuous edge	0.0401	0.037		8.6484	7.9798
	- positive moments at mid span	0.0301	0.028		6.4874	6.0388
9	S9			11.801		
	Negative moments at continuous edge	0.0844	0.047		17.5669	9.7825
	positive moments at mid span	0.0632	0.035		13.1627	7.2849

Choosing the maximum bending moment for calculating effective depth The maximum bending moment = 14.917 KN-m

M_u = BM = $0.138 * f_{ck} * b * d^2$ (for **Fe415)**

$17.6033*10^6 = 0.138 * 20 * 1000 * d^2$

d = 79.86 mm

Provide effective depth = 100mm

D = 100 + c/c + Φ/2

= 100 +15 + 5

= 120

Provide overall depth **D = 150mm**

Effective depth (d) = 150-20

=130mm

c) **Calculation of steel**

$BM = 0.87 f_y A_{st} (d - \frac{f_{y*Ast}}{f_{ck*b}})$

Where, fy = 415 N/mm^2

Calculation of steel along short span

S.N.	DescriptioN	Mx KN-M	Ast Required (mm2)	Φ of bar (mm)	spacing required (mm)	spacing provided	Ast provided (mm2)
1	PannelS1	8.8768	195.20	10	402.29	200	393.9
2	PannelS2	9.1763	202.02	10	388.72	200	393.9
3	PannelS3	14.917	335.81	10	233.85	200	393.9
4	PannelS4	13.328	298.15	10	263.38	200	393.9
5	PannelS5	17.603	390.17	10	201.10	200	393.9
6	PannelS6	13.126	293.40	10	267.65	200	393.9
7	PannelS7	13.501	302.24	10	259.82	200	393.9
8	PannelS8	8.6483	190.02	10	413.27	200	393.9
9	PannelS9	17.566	399.78	10	196.43	200	393.9

Calculation of steel along long span

S.N.	DescriptioN	Mx KN-M	Ast Required (mm2)	Φ of bar (mm)	Spacing Required (mm)	spacing provided	Ast provided (mm2)
1	PannelS1	5.9775	130.05	10	603.83	300	261.933
2	PannelS2	7.5931	166.18	10	472.55	300	261.933
3	PannelS3	11.693	259.91	10	302.14	300	261.933
4	PannelS4	7.9798	174.89	10	449.01	300	261.933
5	PannelS5	11.693	259.91	10	302.14	300	261.933
6	PannelS6	7.7786	170.36	10	460.96	300	261.933
7	PannelS7	11.231	249.20	10	315.12	300	261.933
8	PannelS8	7.9798	174.89	10	449.01	300	261.933
9	PannelS9	9.7825	215.85	10	363.80	300	261.933

d) Check for minimum reinforcement

According to clause 26.5.2.1. of IS-456:2000

A_{st} = 0.12 % of total X- section area

= 0.12* 150 *1000/ 100

= 180 mm^2

e) Check for spacing

As per of IS-456:2000 clause 26.3.3 (b) 1

Spacing < 300 mm

< 3*150 or 435mm (**Hence OK)**

f) Check for shear (for critical panel S5)

Along short span at support edge

Maximum shear force at support (V_u) = $wl_x/2$

= 11.801*5.295/2

= 31.2431 KN

Nominal shear stress

τ_V = V_u/bd

$=(31.2431*10^3)/(1000*130)$

= 0.2348 N/mm^2

Percentage of steel = 100 Ast_provided /bd *2

= 100*393.9/(1000*130 *2)

= 0.17 %

T_c = 0.295N/mm^2

τ_c' = k * τ_c

=1.3 * 0.295

= 0.383N/mm^2 $\tau_c > \tau_v$

Hence Shear reinforcement is not required.

g) Check for deflection

From clause 23.2.1, IS- 456:2000

L/d =αβγδγλ

α = 26

Now for the modification factor 'γ' for tension reinforcement

$$f_s = 0.58 * fy * \frac{\text{Ast required}}{\text{Ast provided}}$$

= 0.58 * 415 N/mm^2 * 390.17/393.9

= 214.92 N/mm^2

Now from the graph of Modification factor for tension reinforcement

γ = 1.8

$(l/d)_{max}$ = 26*1.8 = 46.8

$(l/d)_{provided}$ = (5175/130) = 39.80 < $(l/d)_{max}$ Hence ok

h) Check for development length

M1 = 0.87 * 415 *393.9/2 (130-415*Ast/(20*1000))

= 0.87 * 415 *393.9/2 (130-415*393.9/(2*20*1000))

= 9.553 KN-m

According to IS- 456: 2000 clause 26.2.1.1

For M20 concrete bond stress τ_{bd} = 1.2 *1.6 * (60% increased for deformed bar)

= 1.92 N/mm^2

Anchorage length (Lo) = 16Φ for U bent

Ld $\leq$ 1.3*(M1/Vu) + Lo

$(f_s * \Phi)/4*\tau_{bd}$ $\leq 1.3*(M_1/V_u) + 16\Phi$

(0.87*415*Φ)/4*1.92 $\leq 1.3*(9.533*10^6/25.895*10^3) + 16\Phi$

Φ ≤ 31.62 mm > 10mm (**Hence OK)**

Torsional Reinforcement:

According to IS: 456-2000 ANNED D (1.8 &1.9)

Corner reinforcement is provided at top and bottom of corner for 1/5 of shorter span

5175/5 = 1035 mm

At corner one edge discontinuous

Area of Torsional reinforcement=3/8*Ast short

= 0.375*448.74

= 168.27 mm^2

Spacing = $1000*\pi/4*10^2/168.27$

=466.74 mm^2

Hence provide 10mm- Φ bars @300 mm up to 1053 mm from corner.

5.4 DESIGN OF COLUMN

SAMPLE DESIGN OF CRITICAL COLUMN(C3)

1) Load combination (1.2*Dead+1.2 Live)

Design Parameters

Design constants

Fck = 25 N/mm^2

Fy = 500 N/mm^2

Column size = 450 mm× 650 mm

Check for short or long column

As per *Clause 25.1.2, IS:456-2000*

$\frac{Lex}{D}$ and $\frac{Ley}{b}$ < 12, Short column

$\frac{Lex}{D}$ and $\frac{Ley}{b}$ > 12, Long column

Unsupported length L =3000 − 450 = 2550 mm

Effective length Le =0.80 × 2.55 = 2.040 m

Le/D = 2040/600 = 3.4 <12

Le/b = 2040/450 = 4.53 < 12

Hence column is designed as short column.

From C*lause 25.4,* Minimum eccentricity e_{min} is given by;

$$e_{min_x} \geq \frac{L}{500} + \frac{D}{30} \geq 20 \text{ mm}$$

$$= \frac{2040}{500} + \frac{600}{30}$$

$$= 24.08 \text{ mm} > 20 \text{ mm}$$

$\frac{e_{min_x}}{d}$ = 0.040 < 0.05 Hence ok

$$e_{min_y} \geq \frac{L}{500} + \frac{b}{30} \geq 20 \text{ mm}$$

$$= \frac{2040}{500} + \frac{450}{30}$$

$$= 19.08 \text{ mm} < 20 \text{ mm}$$

Hence minimum eccentricity is taken as 20 mm

$\frac{e_{min_y}}{b}$= 0.04< 0.05 ,Hence ok

DESIGN DATA(2)

Pu	=	**3091.468**	**KN**
Mux	=	**5..045**	**Knm**
Muy	=	**16.4224**	**Knm**

Let us assume percentage of reinforcement (pt)

$$\frac{Pt}{Fck} = \frac{2.60}{25} = 0.10$$

$$\frac{Pu}{fck \times bD} = \frac{3091.468 \times 1000}{25 \times 450 \times 650} = 0.422$$

Assuming 25 mm dia bars with 40 mm clear cover and distributing reinforceme
side. d' = 40 + 25 / 2 = 52.5 Mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{650} = 0.08$$

Chart for
d'/D = 0.10 will be used.
Referring to *chart 48 ,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.12$$

$$Mux1 = 0.12 \times 25 \times 450 \times 600^2$$
$$= 570.375 \text{ KNm}$$

$$Muy1 = 0.15 \times 25 \times 600 \times 450^2$$
$$= 394.875 \text{ KNm}$$

Calculation of P_{Uz}

P_{uZ} $=0.45 \times fck \times Ac + 0.75 \times fy \times Asc$

$= 0.45* 25* 0.974*450*600+ 0.75*500*0.026*450*600$

$=5591.02$ KN

Now,

$$\frac{Pu}{Puz} = \frac{3091.468}{5591.02} = 0.553$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in table

Pu/Puz	α_n
≤0.2	1.0
≥0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \alpha_n = 1+ \left(\frac{0.553 - 0.2}{0.80 - 0.2} \right) = 1.588$$

$$\frac{Mux}{Mux1} = \frac{5.045}{570.375} = 0.0088$$

$$\frac{Muy}{Muy1} = \frac{16.5224}{394.875} = 0.041$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n}$$

$=0.0088^{1.588}+0.041^{1.588}$

$=0.0068 < 1$

The column is safe. Hence OK.

Area of reinforcement of required column (A_{sc}) = 2.60%*(450*600)

$=7020\ mm^2$

Let us provides 12-28 mm Ø bars giving area of steel $=12*\pi/4*28^2$

$=7389.025\ mm^2$

$\therefore$Total Area of reinforcement provided in column (Asc) $=7389.025\ mm^2$

DESIGN OF TRANSVERSE REINFORCEMENT

As per *IS: 456-2000*, *Clause 26.5.3.2*

1. Diameter of lateral ties $\varnothing_T$

$\varnothing_T \geq \varnothing_L/4 = 25 / 4 =$

$\varnothing_T \geq 6$ Mm

Hence, provide 8 mm ∅ lateral ties.

2. Spacing of lateral ties i.e. pitch distance

Pitch distance ≤ Least lateral dimension (450mm)

≤ 16×smallest dia. of longitudinal reinforcement(16*25=400)

≤ 48×Lateral ties (384)

≤ 300 Mm

Provide 8mm dia. Lateral ties at spacing of 300 mm,

But from ductility requirement provide ties @ 100 mm C/C up to 1/6 of clear height (500mm) or Largest column size(600mm)

or 450mm

Therefore Provide lateral ties **8 mm Ø @ 100 mm C/C up 600 mm** below and above Connection and **150 mm C/C** in remaining portion.

DESIGN DATA(5)

Pu = 2409.453 KN
Mux = 4.2149 Knm
Muy = 439.3229 Knm

Let us assume percentage of reinforcement (pt)

$$\frac{pt}{fck} = \frac{2.60}{25} = 0.10$$

$$\frac{Pu}{fck \times bD} = \frac{2409.453 \times 1000}{25 \times 450 \times 650} = 0.329$$

Assuming 25 mm dia bars with 40 mm clear cover and distributing reinforcement equally on side. d' = 40 + 25 / 2 = 52.5 mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{650} = 0.08$$

Chart for d'/D = 0.10 will be used.

Referring to *chart* *48* *,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.15$$

$$Mux1 = 0.15 \times 25 \times 450 \times 600^2$$
$$= 607.50 \text{ KNm}$$

$$Muy1 = 0.15 \times 25 \times 600 \times 450^2$$
$$= 455.625 \text{ KNm}$$

Calculation of P_{Uz}

P_{uZ} $=0.45 \times fck \times Ac + 0.75 \times fy \times Asc$

= 0.45* 25* 0.974*450*600+ 0.75*500*0.026*450*600

=5591.02 KN

Now,

$$\frac{Pu}{Puz} = \frac{2409.453}{5591.02} = 0.4308$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in table

Pu/Puz	α_n
≤0.2	1.0
≥0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \alpha_n = 1+ \left(\frac{0 - 0.2}{0.80 - 0.2} \right) = 1.3846$$

$$\frac{Mux}{Mux1} = \frac{4.2149}{607.50} = 0.00693$$

$$\frac{Muy}{Muy1} = \frac{439.229}{455.625} = 0.964$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(Mux \right)^{\alpha_n} + \left(Muy \right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n}$$

$=0.00693^{1.384}+0.964^{1.384}$

$=0.951 < 1$

The column is safe. Hence OK.

DESIGN DATA

Pu	=	**2546.614**	**KN**
Mux	=	**11.9316**	**Knm**
Muy	=	**433.3147**	**Knm**

Let us assume percentage of reinforcement (pt) 2.6

$$\frac{pt}{fck} = \frac{2.60}{25} = 0.10$$

$$\frac{Pu}{fck \times bD} = \frac{2546.614 \times 1000}{25 \times 450 \times 600} = 0.377$$

Assuming 25 mm dia bars with 40 mm clear cover

d' = 40 + 25 / 2 = 52.5 mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{650} = 0.08$$

Chart for d'/D = 0.10 will be used.

Referring to *chart 48 ,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.15$$

$Mux1 = 0.15 \times 25 \times 450 \times 600^2$

$= 607.50$ KNm

$Muy1 = 0.15 \times 25 \times 600 \times 450^2$

$= 455.62$ KNm

Calculation of P_{Uz}

$P_{uZ} = 0.45 \times fck \times Ac + 0.75 \times fy \times Asc$

$= 0.45 * 25 * 0.974 * 450 * 600 + 0.75 * 500 * 0.026 * 450 * 600$

$= 5591.02$ KN

Now,

$$\frac{Pu}{Puz} = \frac{2546.614}{5591.02} = 0.455$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in ta

	Pu/Puz	α_n
	≤0.2	1.0

	≥ 0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \quad \alpha_n = 1 + \left(\frac{0.455 - 0.2}{0.80 - 0.2} \right) = 1.425$$

$$\frac{Mux}{Mux1} = \frac{11.9316}{607.50} = 0.0196$$

$$\frac{Muy}{Muy1} = \frac{433.3147}{455.62} = 0.951$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(\frac{Mux}{Mux1} \right)^{\alpha_n} + \left(\frac{Muy}{Muy1} \right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1} \right)^{\alpha_n} + \left(\frac{Muy}{Muy1} \right)^{\alpha_n}$$

$= 0.0196^{1.425} + 0.951^{1.425}$

$= 0.934 < 1$

The column is safe. Hence OK.

DESIGN DATA(7)

Pu	**=**	**2348.056**	**KN**
Mux	**=**	**287.535**	**Knm**
Muy	**=**	**20.5712**	**Knm**

Let us assume percentage of reinforcement (pt) 2.6

$$\frac{pt}{fck} = \frac{2.60}{25} = 0.10$$

$$\frac{Pu}{fck \times bD} = \frac{2348.056 \times 1000}{25 \times 450 \times 600} = 0.347$$

Assuming 25 mm dia bars with 40 mm clear cover and distributing reinforcem
side. $d' = 40 + 25 / 2 = 52.5$ mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{650} = 0.08$$

Chart for
d'/D = 0.10 will be used.
Referring to *chart 48 ,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.14$$

$Mux1 = 0.14 \times 25 \times 450 \times 600^2$

$= 567.0$ KNm

$Muy1 = 0.14 \times 25 \times 600 \times 450^2$

$= 425.25$ KNm

Calculation of P_{Uz}

$P_{uZ} = 0.45 \times fck \times Ac + 0.75 \times fy \times Asc$

$= 0.45 * 25 * 0.974 * 450 * 600 + 0.75 * 500 * 0.026 * 450 * 600$

$= 5591.02$ KN

Now,

$$\frac{Pu}{Puz} = \frac{2348.056}{5591.02} = 0.420$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in table

Pu/Puz	α_n
≤0.2	1.0
≥0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \alpha_n = 1 + \left(\frac{0.420 - 0.2}{0.80 - 0.2} \right) = 1.367$$

$$\frac{Mux}{Mux1} = \frac{287.535}{567.0} = 0.507$$

$$\frac{Muy}{Muy1} = \frac{20.571}{425.25} = 0.0484$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(\frac{Mux}{Mux1} \right)^{\alpha_n} + \left(\frac{Muy}{Muy1} \right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1} \right)^{\alpha_n} + \left(\frac{Muy}{Muy1} \right)^{\alpha_n}$$

$= 0.507^{1.367} + 0.0484^{1.367}$

$= 0.411 < 1$

Hence The column is safe.

Design Summary of Column

Provide 16-25 mm dia. bars

Therefore Provide lateral ties **8 mm Ø @ 100 mm C/C up to 600 mm** below and above Connection and **150 mm C/C** in remaining portion

5.5 Design of Beam

Design of moment for design is taken as maximum value of B.M obtained from analysis using software Etabs 2017

Size of beam = 300*600 mm

Maximum sagging moment(M_U)=106.86KNm

From SP 16, Limiting Moment (Mulim) = 0.133f_{ck}*bd^2

If Mu<Mulim section is designed as singly reinforcement

If Mu≥Mulim section is designed as doubly reinforcement

At Left end

i)For Sagging moment (+ve Moment)

From diagram Mu =87.229KNm and Vu=150.22 KN

Now,

Mulim = 0.138f_{ck}*bd^2

Where, d = effective depth

Therefore,

Mulim = 0.138*f_{ck}*bd^2

Mulim=0.133*20*300*557^2

=256.88KNm

Here Mu<Mulim so section design is singly beam section

Now,

Mu = 0.87*fy*Ast(d-(fy*Ast/(fck*b))

Or, 87.229*10^6= 0.87*415*Ast*(557-(415*Ast)/(20*300)or, Ast =479.07 mm^2

Provide 16 mm diameter bar

Number of bar = Ast/[(3.14*d^2)/4]

=2.38 ≈3 nos

Ast provided =603.18 mm^2

Check for minimum reinforcement

As per IS: 456-2000, Clause 26.5.1.1

For minimum reinforcement = 0.85*(bd/fy)

= 342.25 mm^2 < Ast Provided(603.18 mm^2)

ii) **For Hogging Moment (-ve moment)**

From diagram Mu =245.86KNm and Vu=150.22 KN

Now,

Mulim = $0.138f_{ck}*bd^2$

Where, d = effective depth

Therefore,

Mulim = $0.138*f_{ck}*bd^2$

Mulim=$0.138*20*300*557^2$

=256.88KNm

Here Mu<Mulim so section design is singly beam section

Now,

Mu = 0.87*fy*Ast(d-(fy*Ast/(fck*b))

Or, $245.86*10^6$= 0.87*415*Ast*(557-(415*Ast)/(20*300))

or, Ast =1511.77 mm^2

Provide 20 mm diameter bar

Number of bar = Ast/[(3.14*d^2)/4]

=4.8 ≈ 5 nos

Ast provided =1570.79 mm^2

a) Check for minimum reinforcement

As per IS: 456-2000, Clause 26.5.1.1

For minimum reinforcement = 0.85*(bd/fy)

= 342.25 mm^2 < Ast Provided(1570.79 mm^2)

b) Check for deflection

l/d≤ "α β γδ λ"

where, α = 26

For modification factor λ

pt = 100*Ast/(b*d)

=100*603.18/(300*557)

=0.36

$fsc = 0.58*fy*(Ast_{req}/Ast_{provided})$

$=0.58*415*(479.07/603.18)$

$=191.17$ N/mm^2

For pt = 0.36% and fsc=212.83N/mm2 from IS 456

1978

$\gamma=1.62$

then,

$6.733/0.555 \leq 20*1.62*1*1=32.4$

$12.13 \leq 32.4$ **ok**

c) Check for shear

Shear strength = Vu/bd

$=150.22*10^3/(300*557)$

$=0.905$N/mm^2

From IS 456-2000

tcmax = 2.8 N/mm^2

p=0.36 %

For M20 concrete and p=0.32%. Interpolate in table for T in IS 456:2000

p	Tc (N/mm^2)
0.25	0.36
0.36	Tc
0.50	0.48

Tc = 0.4128N/mm^2

Since, Tv>Tc

Shear reinforcement is to be provided

Shear resistance of reinforcement

Vus = Vu-tc*b*d

$=150.22*10^3-0.4128*300*555$

=82.038KN

Using 2 legged 8mm diameter vertical stirrups

$Asv = 2*3.14*8^2/4$

$=100.53\ mm^2$

Spacing of stirrups,Sv=0.87*fy*Asv*d/Vus

=0.87*500*100.53*557/82.03*1000

=296.91 mm

Nominal spacing(Sv) =0.87*fy*Asv/(0.4b)

= 302.46mm

The max spacing should be less than

a)0.75*d = 0.75*557=417.75mm

b)300 mm

Provide 8 mm dia two legged stirrups @250mm spacing

But for ductility requirement , According to IS 13920: 2016 clause 8.1

Special confining reinforcement should be provided over a length L_0 ; where L_0 is not less than

- Largest later dimension of member
- 1/6 of clear span of member
- 450 mm

And have spacing not more than

- ¼ of minimum member dimension of beam or column
- 6 times dimension of the smallest longitudinal bars
- 100 mm link

Hence provide 8mm dia two legged stirrups @ 75 mm c/c up to 1100 mm from support and 200 mm c/c in remaining portion.

d) Check for development length

$M1/v+l_0 \geq Ld$

Ld = diameter*0.87*fy/4tbd

= 16*0.87*415/(4*1.6*1.4)

=644.73mm

M1 = 0.87*fy*Ast/2(d-fy*Ast/2fck*b)

= 0.87*415*603.18/2(415-415*603.18/2*25*300)

=58.83 KNm

Lo= 16Ø for U bent

= 256 mm

Ld<1.3M1/V+Lo

< 765.11 mm

OK

At midspan of beam

For Sagging moment (+ve Moment)

From diagram Mu =106.51KNm and Vu=49.29 KN

Now,

Mulim = $0.138f_{ck}*bd^2$

Where, d = effective depth

Therefore,

Mulim = $0.138*f_{ck}*bd^2$

Mulim=$0.138*20*300*557^2$

=256.88KNm

Here Mu<Mulim so section design is singly beam section

Now,

Mu = 0.87*fy*Ast(d-(fy*Ast/(fck*b))

Or, $106.51*10^6$= 0.87*415*Ast*(557-(415*Ast)/(20*300)

or, Ast =572.35 mm^2

Provide 16 mm diameter bar

Number of bar = Ast/[(3.14*d^2)/4]

=2.946

Let us provide 4-16 mm dia bars

Ast provided =804.24 mm^2

Check for minimum reinforcement

As per IS: 456-2000, Clause 26.5.1.1

For minimum reinforcement = 0.85*(bd/fy)

= 342.25 mm^2 < Ast Provided(804.24 mm^2)

Design summary of Beam

Provide 4-16 mm dia bars as bottom reinforcement bars out of which 1 bars be curtailed at 990 mm(0.15l) from support

Provide 2-20 mm dia bars as top reinforcement bars throughout the length and 3-20 mm dia additional bars up to 1650 mm (0.25l) from support.

Provide 8mm dia two legged stirrups @ 75 mm c/c up to 1100 mm from support and 200 mm c/c in remaining portion.

5.6 Design of staircase

Staircase is an inclined structural system for the movement from one level to another. Since it is stepped, it is called staircase. A staircase behaves like an ordinary slab. It may span either in the direction of the steps or in the direction of going. Structurally staircase may be classified largely into two categories, depending on the predominant direction in which the slab staircase component of the stair undergoes flexure- stair slab spanning transversely and stair slab spanning longitudinally.

The design of staircase requires proportioning of its different components and determination of reinforcement and its detailing to satisfy both the serviceability and strength requirements. The design of staircase is made for serviceability requirements of deflection and cracks. The serviceability requirement of deflection is controlled by the effective span to effective depth ratio. The design of reinforcement is made to satisfy the strength requirements for moments and shear. The design for moment is made for maximum moments either by working stress method or by the limit state method. The area of steel is expressed as diameter and spacing of bars. It is provided along the span of staircase and necessary curtailment is made wherever it is not required as in the case of edge supported slab.

Generally the shear reinforcement is not required in the staircase as the shear strength of concrete is much greater than the nominal shear stress. The shear strength of concrete in staircase is determined as in the case of edge supported slabs.

The detailing of reinforcement in staircase shall be similar to that of the edge supported slab except at the junction of landing and flight of staircase where it should ensure that the reinforcement bars in tension tending to straighten out do not cause cracking in concrete. General rules

Between consecutive floors there should be equal rise for every parallel steps. Similarly there should be equal going.

There should be at least 2m headroom measured vertically above any steps.

The sum of going of a single step plus the twice the rise should be between 550mm and 700mm

The rise of steps should not be more than about 200mm and the going not less than 240mm The slope of the staircase should be not more than 38 degrees.

Width of staircase depends upon the usage. The width required in residential building differs from other public building.

5.6.1 Design of Staircase 1

Room size available for staircase=5175mm*3500mm

Effective length=centre to centre distance between supporting beam

=5.17m

Let us take,

Thickness of waist slab=$\frac{L}{25}$

=$\frac{5175}{25}$

=207mm

Let us take overall depth of waist slab (D) =225mm

Therefore, effective depth (d) =200mm

Let us assume riser height (h) =150mm

Total number of riser = $\frac{3000}{150}$

=20

Number of raiser in each flight = $\frac{20}{2}$

=10

Number of tred in each flight =10-1

=9

a) LOADS

I. Weight of slab in plan(per meter of width of fight) =D*$\sqrt{1+\frac{R^2}{T^2}}$ *25

=0.225*$\sqrt{1+\frac{0.150^2}{0.250^2}}$

*25

=6.559 KN/m

II. Weight of steps (per meter width of flight) =$\frac{25RT}{2T}$

=$\frac{25*0.15*0.25}{2*0.25}$

=8.434 KN/m

Total dead load=6.559+1.875

=8.434 KN/m

Live load =5KN/m (as per IS 875)

Total load=Total dead load+live load

=8.434+5

=13.434 KN/m

<u>For landing</u>

Dead load =0.225*25*1

=5.625KN/m

Live load =5 KN/m

∴Total load =Live load +Dead load

=5+5.625

=10.625 KN/m

∴Factored load =1.5*10.625

=15.937 KN/m

Load diagram at stair

R_A+R_B = (15.937*1.5) + (20.151*2.25) + (15.937*1.425)

=91.955KN

M_A=0

$(15.937*1.5*\frac{1.5}{2}) +(20.151*2.250*(1.5+\frac{2.5}{2}))+(15.937*1.425*(\frac{1.425}{2}+2.25+1.5))-$

R_B+5.175=0

R_B=238.29/5.175

R_B=46.046KN

∴ R_A=91.955-46.046

=45.909KN

Position of maximum moment=point of zero shear force

At distance X from A,

45.909-(15.937*1.5)-(20.151*(X-1.5))=0

∴X=2.591m

b) Calculation of depth

Maximum B.M at X=2.591m from A;

Max B.M=R_A*2.591-15.937*1.5*($\frac{1.5}{2}+1.091$)-20.151*1.091*$\frac{1.091}{2}$

=62.94KN/m

For Fe415

M_{uLim}=0.138*f_{ck}*b*d^2

62.94*10^6=0.138*20*1500*d^2

∴d=123.30mm

Adopt d=150mm

Overall depth (D) =150+$\frac{\phi}{2}$+clear cover

=150+$\frac{12}{2}$+20

=176mm

Adopt overall depth (D) =180mm

c) Design of reinforcement

M=0.87*f_y*A_{st}*d{1-$\frac{fy}{fck}*\frac{Ast}{b*d}$}

62.94*10^6=0.87*415*A_{st}*150*{1-$\frac{415}{20}*\frac{Ast}{1500*150}$}

A_{st}=1323.77mm^2

Provide 16mm diameter bars;

Spacing =$\frac{1500}{1323.77}$*$\frac{\pi}{4}*16^2$

=227.282mm

Adopt spacing = 200mm

A_{st} Provided=$\frac{1500*\frac{\pi}{4}*}{200}$*$16^2$

=1507.96mm^2

Distribution steel=0.15% of bd

=$\frac{0.15}{100}*1500*150$

=337.5mm^2

Spacing of 10mm ϕ bars=$\frac{\frac{\pi}{4}*10^2}{337.5}$*1500

=348.88mm

Provide 10ϕ bars @ 300mm c/c

d) Check for shear

Nominal shear stress (τ_v)=$\frac{RB}{bd}$

Percentage of steel used=100*$\frac{Ast\ provided}{bd}$

=100*$\frac{1507.96}{1500*150}$

=0.670%

Shear strength of M_{20} @ 0.67% of steel (τ_c)

0.50→0.48

0.67→ τ_c

0.75→0.56

By interpolation;

τ_c=0.534N/mm

Maximum shear stress for M_{20} concrete (τ_c')=1.3* 0.534

=0.694 N/mm^2

Here, $\tau_c{}'$>τ_c (Hence no shear reinforcement is required)

e) Check for deflection

Percentage of steel used=0.67%

From clause 23.2.1(IS 456-2006)

Using the deflection criteria for continuous slab;

$\frac{span}{d} \leq \alpha\beta\gamma\delta\lambda$

Where,

α=26

β=1

λ=1

δ=1

Now for modification factor(r)

F_s=0.58*f_y*$\frac{Ast\ required}{Ast\ provided}$

=0.58*415*$\frac{1323.77}{11507.96}$

=211.29N/mm^2

Now from graph of modification factor

r=1.4

$\frac{5.175}{0.150} \leq 26*1.4$

34.5≤36.4 (hence ok)

f) Check for development length

$M=0.87*f_y*A_{st}*d*\{1-\frac{f_y*A_{st}}{f_{ck}*b*d}\}$

$=0.87*415*1507.96*150*\{1-\frac{415*1507.96}{20*1500*150}\}$

=70.31 KNm

According to IS-456:2000 clause 26.2.1.1

For M_{20} concrete

$\tau_{bd}=1.2*1.6$

$=1.92N/mm^2$

$L_d=\frac{\emptyset*\sigma_a}{4*\tau_{bd}}$

$=\frac{0.87*f_y*\emptyset}{4*1.92}$

$=\frac{0.87*415*16}{4*1.92}$

$L_d \leq 1.3\frac{M_1}{V_u}$

$\leq 1.3\frac{70.31*10^6}{46.046*10^3}$

752.18mm≤1996.329mm (Hence ok)

5.6.2 Design of staircase 2

Room size available=4500*3750mm

Effective length =centre to centre distance between supporting beam

=4.5m

Let us take thickness of waist slab=L/25

=4500/25

=180mm

Let us take overall depth (D) =205mm

Effective depth =D-clear cove-

=205-15-10/2

=180mm

Let us assume riser height(h) =150m

Total no of riser =3000/150

=20mm

No of riser in each flight =20/2

=10mm

No of tread in each flight =10-1=9

a) Loads

1. Weight of waist slab per meter width of flight =D*$\sqrt{1+\frac{R^2}{T^2}}$ *25

=0.205*$\sqrt{1+\frac{0.150^2}{0.250^2}}$ *25

=5.976KN/m

2. Wt. of steps per meter width of flight= $\frac{1}{2}\ \frac{RT}{T}$*25

= $\frac{1}{2}\ \frac{0.15*0.25}{0.25}$*25

=1.875 KN/m

Total dead load =7.85 KN

Live load =5 KN/m^2

Total =DL+LL

=7.85+5

=12.851

Factored Load =1.5*12.851

=19.276 KN/m

For Landing:

Dead load (D_L) =0.205*25*1

=5.125KN/m

Live Load (L_L) =1*5 KN/m^2

=5KN/m

Total Load =DL+LL

=5.125+5

=10.125KN/m

Factored Load=1.5*10.125KN/M

=15.187KN/m

$R_A + R_B$ = **15.187*1.4+19.276*2.25+15.187*0.85**

=77.542KN/M

+$\boldsymbol{M_{A=0}}$

15.187*1.4*1.4/2+19.276*2.25*(2.25/2+1.4)+15.187*0.85*(0.85/2+2.25+1.4)-R_B*4.5=0

$R_B = \frac{176.99}{4.5}$

R_B=39.33KN

$R_{A=}38.208KN$

Position of maximum moment =point of zero share at a distance from A;

- 38.208-(15.187*1.4)-19.276*(x-1.4=0)
- X=2.279m

c) Calculation of depth

Maximum bending moment=R_A *2.279-15.187*1.4*(1.4/2+0.879)-19.276*0.879*(0.879/2)

=46.056KN/M

For Fe415;

M_{lim} = 0.133*fck*bd^2

46.056*10^6 =0.138*20*1400*d^2

d=109.175*175 <$d_{assumed}$(180mm)

Adopt , d=150mm

Overall depth (D)=150+dai/2+clear cover

=150+20/2+20

=180 mm

c) **Design of reinforcement**

M = 0.87 * Fy * Ast * d * [$1 - \frac{f_y}{f_{ck}} * \frac{A_{st}}{bd}$]

46.056*10^6=0.87*415* Ast*150*[1-$\frac{415}{20} * \frac{A_{st}}{1400*150}$]

Ast =1187.77mm^2

Provide the 16 mm dia. Bars;

Spacing=$\frac{1400*\frac{\pi}{4}*16^2}{1187.77}$

=236.98mm

Hence, provide 16mm dai bars@200mm c/c.

Ast provided =$\frac{1400*\frac{\pi}{4}*16^2}{200}$

=1407.43mm^2

Distribution bars=$\frac{0.15*1400*150}{100}$

=315

Spacing =$\frac{1400*\frac{\pi}{4}*10^2}{315}$

=349.056

Adopt spacing=300mm c/c

d) Check for shear;

Maximum shear(v_u)=139.33KN

Nominal shear stress(T_v)=$\frac{RV_u}{b*d}$

=$\frac{39.33*10^3}{1400*150}$

=*0.187* KN/m^2

Percentage of steel used=$100 * \frac{A_{ST\ provided}}{bd}$

=100*$\frac{1407.43}{1400*150}$

=0.670%

Shear strength of M_{20} @ 0.67% of steel (τ_c)

0.50→0.48

0.67→ τ_c

0.75→0.56

By interpolation;

τ_c=0.534N/mm

Maximum shear stress for M_{20} concrete (τ_c')=1.3* 0.534

=0.694 N/mm^2

Here, $\tau_c{'}$>τ_c (Hence no shear reinforcement is required)

e) Check for deflection

Percentage of steel (P_t)=0.67%

From clause 23.2.1(IS 456-2000)

$$\frac{Span}{depth} \leq \propto \beta\gamma\delta\boldsymbol{\lambda}$$

Where,

$\propto$=26

β, λ, δ =1

$f_{s=}$0.58f_y*$\frac{A_{st\ required}}{A_{st\ provided}}$

=0.56*415*$\frac{1187.77}{1407.43}$

=203.13N/mm^2

∴Now from graph for pt=0.67 and fs=203.13 N/mm^2 ,$\gamma = 1.35$

(l/d)max= 26*1.35

= 35.1

(l/d)provided= (4500/150)

= 30 < 35.1 hence ok

f) Check for development length

M=0.87$\boldsymbol{f_y}$ * Ast*d*[1-$\frac{f_y \text{Ast}}{f_{ck\ bd}}$]

=0.87*415*1407.43*150*[1-$\frac{415*1407.43}{415*1400*150}$]

=75.71KN/M.

According to IS 456:2.1.1;

For M_{20} concrete, T_{bd}=1.2*1.6= 1.92

L_d =$\frac{\emptyset*\sigma_a}{4T_{bd}}$

=$\frac{0.87\boldsymbol{f_y}\emptyset}{4*1.92}$

$= \frac{0.87 * \mathbf{415} * \mathbf{16}}{4 * 1.92}$

$$L_d \ \leq 1.3 * \frac{75.71 * 10^6}{39.33 * 10^3}$$

$752.18 \leq 2502.49$mm Hence ok;

5.7 Design of foundation

If the load transmitted by the column in a structure are so heavy or the allowable soil bearing pressure so small that individual footing would cover more than about one half of the area. It may be better to provide a continuous footing under all the columns and walls. Such a footing is called a **raft** or **mat** foundation. The raft is divided into series of continuous strips centered on approximate column rows in the both directions. The shear and bending moment diagram may be drawn using continuous beam analysis or coefficients for each strip. The depth is selected to satisfy shear requirements. The steel requirement will vary from strip.

Grid	1	2	3	4	5	6
A	1097.94	1793.127	2047.456	1716.45	1487.88	1064.61
B	1477.18	2290.38	2585.52	2115.351	1161.62	1081.96
C	2009.74	1895.65	2118.51	1419.57	872.88	
D	1232.40	1588.77	1134.92			

Centre of mass with respect to column A1

X= ((1793.127+2290.38+1895.65+1588.77)*4.5+(2047.456+2585.52+2118.51+1134.92)*11.1+(1716.45+2115.351+1419.57)*17+(1487.88+1161.62+872.88)*20.7+(1064.61+1081.96)*24.1)/31664

= 10.877 m

Y=
((1477.18+2290.38+2585.52+2115.351+1161.62+1081.96)*5.175+(2009.74+1895.65+2118.51+1419.57+872.88)*9.45+(1432.40+1588.77+1134.92)*13.2)/ 31664

=6.046 m

Centre of buildings with respect to column A1
X1= 10.66 m
Y1= 6.88 m

Now, eccentricity
ey = 0.834 m
ex = 0.217 m

Moment due to eccentricity
Mx= Q*ex =31664.62*0.217
=6871.22 KNm
My=Q*ey = 31664.62*0.834
= 26408.29 KNm
Area(A)= 350.87 m^2

Soil pressure at any point

$$\sigma = P/A \pm (M_y * x) / I_y \pm (M_x * y) / I_x$$

At A6 (13.54, 6.88)

σ= 119.48 KN/ m^2

At B6 (13.54, 1.705)

σ= 109.198 KN/ m^2

At C5 (10.04, -2.57)

σ= 96.67 KN/ m^2

At D3(0.44, -6.32)

σ= 78.18 KN/ m^2

At D1 (-10.66, -6.32)

$\sigma =$ 65.41 KN/ m^2

At A1 (-10.66, 6.88)

$\sigma =$ 91.658 KN/ m^2

In the X-direction the raft is divided into 4 strips, i.e. 4 equivalent beam

i) beam A-A with 3.787 m width and soil pressure 119.48 KN/m2

ii) beam B-B with 4.725 m width and soil pressure 114.339 KN /m2

iii) beam C-C with 4.012 m width and soil pressure 102.394 KN /m2

iv) beam D-D with 3.85 m width and soil pressure 87.425 KN /m2

Let the bending moment coefficient 1/10 and L as center of column distance

$M = WL^2/10$

For strip A-A

M=520.454 KNm

For strip B-B

M= 498.06 KNm

For strip C-C

M= 448.38 KNm

For strip D-D

M= 380.823 KNm

The depth of raft will be governed by two way shear at the one of column.

$\tau_c' = 0.25*\text{sqrt}(Fck)$

$= 1.11803$ N/mm^2

For Column A6

Perimeter = (2d+1000)

$\tau_v = V_u/b_0d$

1.118 =1.5*1064.61/(2d+100)*d

d = 634.21 mm

For Column D1

Perimeter = (2d+1000)

$\tau_v = V_u/b_0d$

1.118 =1.5*1432.40/(2d+100)*d

d = 861.638 mm

For Column B4

Perimeter = (4d+2000)

$\tau_v = V_u/b_0d$

1.118 =1.5*2585.52/(2d+100)*d

d = 690.30 mm

Adopt d= 950 mm

Hence, Overall depth (D) = 950 mm + 50 mm

= 1000 mm

Calculation of steel Required

M=0.87*fy*Ast*[d-{(fy*Ast)/(fck*b)}]

Or, $520.454*10^6$ =0.87 * 500* Ast*(950-((500* Ast)/(20*1000))

- Ast= 1304.17 mm^2 > Ast(min)= 1140 mm^2

Spacing required= 1000* 314.159/ 1304.17

= 240.88 mm

Hence, Provide 20mm dia bars @ 200 mm c/c in both direction .

6 CONCLUSION

This project is the result of group effort of whole group's member and the valuable guidance of our supervisor. The project work enables us to consolidate the knowledge of analysis and design of structure during our B.E course. The design of elements was done using limit state design philosophy, which is economic, safe and reliable. The detailing of structure was done as per IS and IS seismic codes. The Software used in this project is ETABS 2017.

Project has indeed widened our perspective and acquainted us on how to perceive and counteract the worst possible difficulties regarding the analysis and design of structures. This project work has mainly focused towards the structural analysis and design only. Nevertheless, the attempts have been made in the architectural planning and for the presentation of the analysis and design results in the tabular form with necessary drawing and details. A constant painstaking study and devotion to the work by the project group couple with the valuable guidance of the advisor made it possible in bringing up the project work to this level.

Finally, we hope that efforts and coordination during the project work will be much useful in our career and project will be helpful in providing information on the sesmic design and its safe practice and we hope, this project will help us in similar jobs that we might have in our hands in the future.

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