SEISMIC ANALYSIS AND DESIGN OF A G+9 RESIDENTIAL BUILDING WITH DUCTILE DETAILING USING MACROS

A REPORT

Submitted in partial fulfillment of the requirements for the project presentation of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

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CERTIFICATE

This is to certify that the first project report entitled **" SEISMIC ANALYSIS AND DESIGN OF A G+9 RESIDENTIAL BUILDING WITH DUCTILE DETAILING USING MACROS"** is a bonafide record of the work carried out by Mr. Aneesh, Mr. Jai and Mr. Sarthak under my supervision and guidance. This report is submitted in partial fulfillment of the Project for the award of B-Tech at Jaypee University of Information Technology.

The above statement made is correct to the best of our knowledge.

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ACKNOWLEDGEMENT

"We have taken efforts in this project. However, this would not have been possible without the support and help of many individuals. We would like to thank all of them.
We are highly obliged to **Mr. Bibhas Paul** for his guidance and continuous supervision as well as for providing necessary information regarding the project & also for his support in completing the project.
The topic **"SEISMIC ANALYSIS AND DESIGN OF A G+9 RESIDENTIAL BUILDING WITH DUCTILE DETAILING USING MACROS"** was very helpful for us in giving the necessary background information and motivation in choosing this topic for the project. Our sincere thanks to our project guide **Mr. Bibhas Paul** and project co-coordinator **Mr. Abhilash Shukla** who helped us with the work related query to this project.

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ABSTRACT

The predictable growth in economic actions during the 21st century is very huge in construction and in transportation. With the increase in population and globalisation there is very less land available for construction purposes, thus the need of high rise buildings which are capable enough to resist seismic forces are of much importance.

Designing the members of such a high rise building is being very repetitive and time consuming, so the concept of MACROS which is widely used for design purposes and can easily eliminate these disadvantages.

In this project the structure is first analysed using Staad Pro and members of the building

are designed on the same .Manual calculations are compared with the Staad Pro results.

After this code is developed for different members of structure to remove human errors and time consumption.

At the end ductile detailing is done . In general, ductility of a structure is the capacity to undergo elastic deformationswithout singnificant loss of strength. So , if the structure is ductile , its coccupantswill have sufficient warning of the impending failure, thus reducing the probability of loss of life in the event of collapse.

Chapter 1: INTRODUCTION

1.1 GENERAL INTRODUCTION

In every aspect of human civilization we need structures to live in or to get what we need. But it is not only building structures but to build efficient structures so that it can fulfill the main purpose for what it was made for. Here comes the role of civil engineering and more precisely the role of analysis of structure.

There are many classical methods to solve design problem, and with time, new software's also coming into play. Here in this project work based on software named staad pro has been used.

Few standard problems also have been solved to show how staad pro can be used in different cases. These typical problems have been solved using basic concept of loading, analysis, condition as per IS code. These basic techniques may be found useful for further analysis of problems. Amongst the natural hazards, earthquakes have the potential for causing the greatest damages to engineered structures. Since earthquake forces are random in nature and unpredictable, the engineering tools needs to be sharpened for analyzing structures under the action of these forces. India has a number of the world's greatest earthquakes in the last century. In fact, more than fifty percent area in the country is considered prone to damaging earthquakes. The northeastern region of the country as well as the entire Himalayan belt is susceptible to great earthquakes of magnitude more than 8.0.

1.2 AIM OF PROJECT

This project aims for learning and implementing the concept of structural design in real life with the help of computer aids. This project helps us to get to know more about implementation of the concepts.

- Understanding of design and detailing concept.
- Learning of analysis and design methodology which can be very helpful in the field.
- Understanding of earthquake resistance design concept. Approach for professional practice in the field of structural engineering.
- Comparing the results shown by software with manual computation and thereby finding the efficiency of system.
- Design of concrete and steel elements of the structure.

- Designing and understanding of earthquake resistance structure design concept.

- Design of foundations.

1.3 OBJECTIVES

- Carrying out manual design of the structural elements of a residentia building and thereby comparing results with software.
- Studying the effect of seismic force on structure and methods to make it earthquake resistant.
- To learn the concepts of ductile detailing and design the elements.
- To implement our basic design concepts in real life.

1.4 SCOPE OF PROJECT

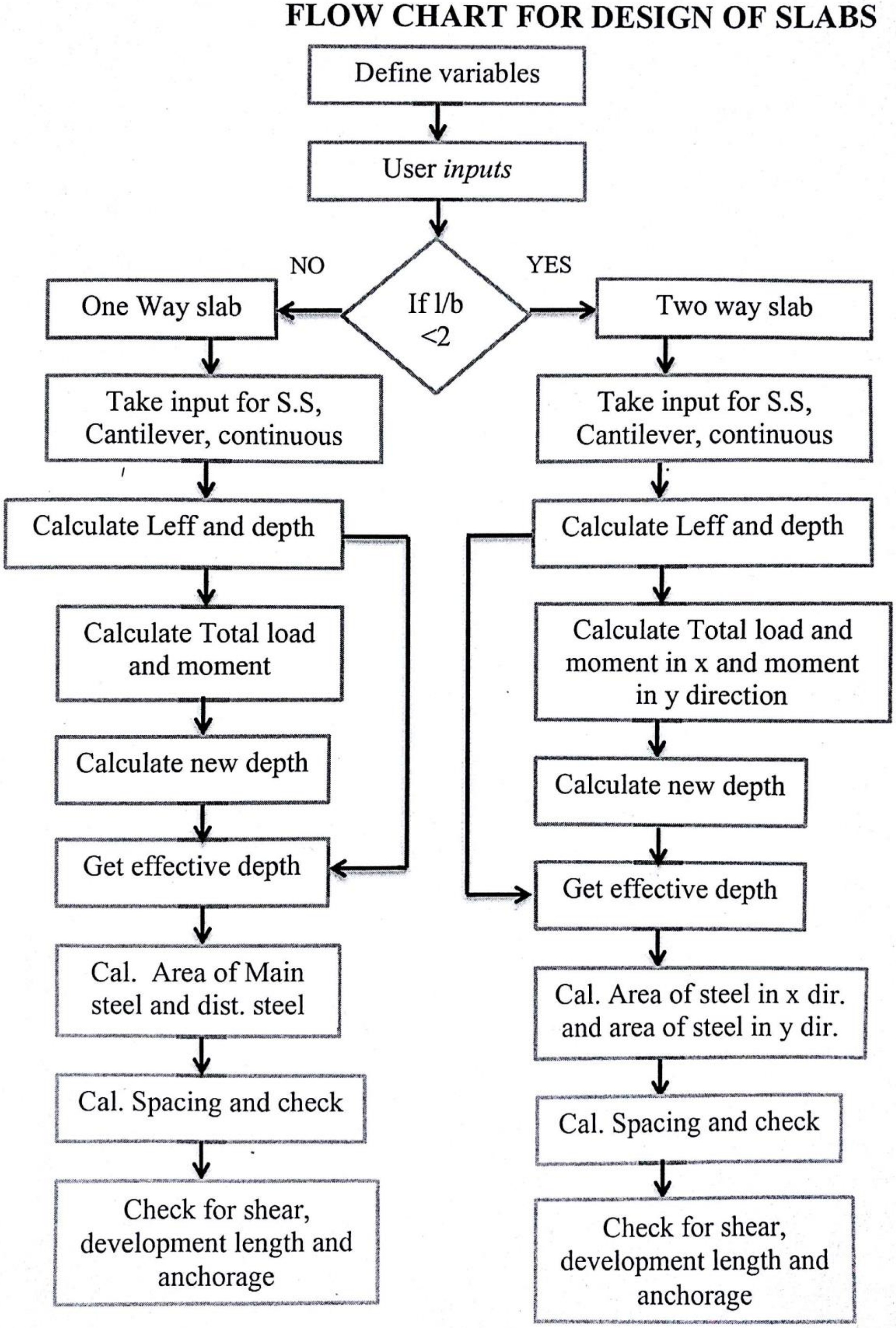
Following points will be covered in project work –

- Study of design of various elements of building and to get familiar with ductile detailing.
- Planning of various components of a building with column positioning in AUTOCAD.
- Manual load calculations of various elements of the structure.
- Modelling of the building in the STAAD.Pro giving all boundary conditions (supports, loading etc.)
- Analysis and Design of various structural components of the modal building
- Study of analysis Data of the software
- Detailed design and drawing of slabs, beams, columns, staircases, footings.
- Design of various elements for earthquake forces as per IS:13920

1.5 WORK PLAN

Table no. 1

JULY-SEPTEMBER 2016	Give a gener outline to project, draw model on Au Cad, plot the same on STAAD PRO apply genera loads mainly dead and live load
OCTOBER-DECEMBER	Manually computing th wind load an earthquake lo and apply sa loads on structure, des of structure elements bot manual and software des and comparin the results w manual computation.


FLOW CHART FOR DESIGN OF SLABS
Define variables
User *inputs*
If l/b
<2
NO
YES
One Way slab
Two way slab
Take input for S.S, Cantilever, continuous
Calculate Leff and depth
Calculate Total load and moment
Calculate new depth
Get effective depth
Cal. Area of Main steel and dist. steel
Cal. Spacing and check
Check for shear, development length and anchorage
Take input for S.S, Cantilever, continuous
Calculate Leff and depth
Calculate Total load and moment in x and moment in y direction
Calculate new depth
Get effective depth
Cal. Area of steel in x dir. and area of steel in y dir.
Cal. Spacing and check
Check for shear, development length and anchorage

2016	
JAN-MARCH 2017	Design of structure elements(bea slabs, columr staircases,) a building VBA code of the same.
MARCH-APRIL 2017	Design of footings, duc detailing of structure and detailed drav of reinforcemen

1.6 IS CODES

- IS:456-2000 :Design Code For RCC Structures
- IS:875(Part-1) :Code For Dead Loads
- IS:875(Part-2) :Code For Imposed Loads
- IS:875(Part-3) :Code For Wind Loads
- IS:1893.1.2002:code For Earthquake Load
- IS:1904 – 1986- Code of practice for design and construction of foundations in soils : general requirements
- IS:13920-Ductile Detailing

1.7 STATEMENT OF PROJECT

1. Utility of building: Residential building(hotel)
2. No. of storeys: G+9
3. No. of staircases: 10
4. Shape of the building: Rectangular
5. Construction Material: R.C.C frame structure
6. Type of walls : Brick wall

1.8 SOFTWARES USED

- STAAD PRO V8i
- Auto-Cad:2013

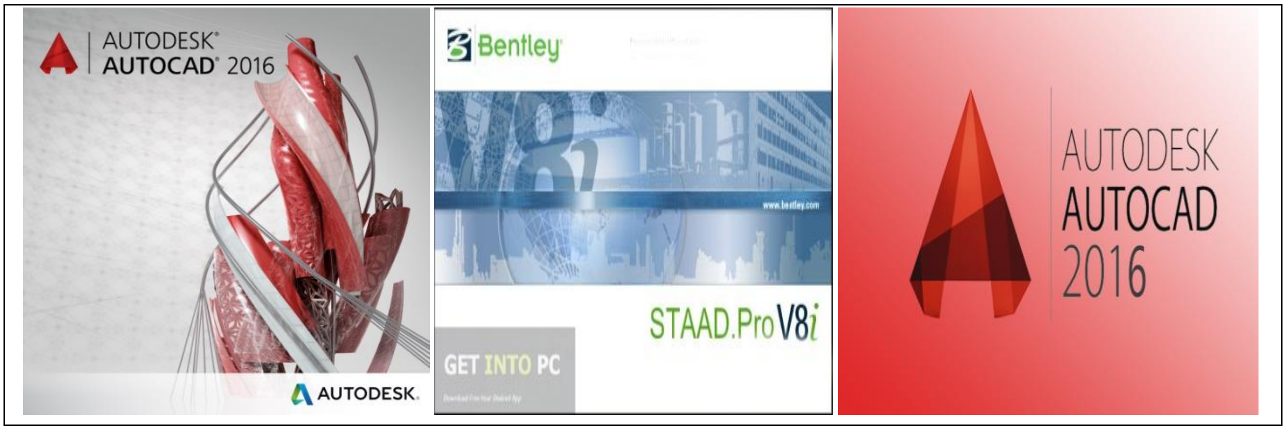


Figure no. - 1

CHAPTER 2: LITERARY REVIEW

2.1 GENERAL

HOW EARTHQUAKE RESISTANT CONSTRUCTION IS DIFFERENT?

Since the magnitude of a future earthquake and shaking intensity at a particular site cannot be estimated with a practical accuracy, the seismic forces are difficult to quantify for the purposes of design. Further, the actual forces that can be generated in the structure during an earthquake are very large and designing the structure to respond elastically against these forces make it too expensive.

Therefore, in the earthquake resistant design post yield inelastic behavior is usually relied upon to dissipate the input seismic energy. Thus the design forces of earthquakes may be only a fraction of maximum (probable) forces generated if the structure is to remain elastic during the earthquake. For example, the design seismic for buildings may at times be as low as one tenths of the maximum elastic seismic force. Thus, the earthquake resistant construction and design does not intend to achieve a structure that will not get damaged in a strong earthquake having low probability of occurrence; it aims to have a structure that will perform properly and without collapse in the event of such a shaking.

Ductility is the capacity of the structure to undergo deformation beyond yield without loosing much of its load carrying capacity. Higher is the ductility of the structure; more is the reduction possible in its design seismic force over what one gets for linear elastic response. Ensuring ductility in a structure is a major concern in a seismic construction

2.2EFFECT OF EARTHQUAKE ON REINFORCED CONCRETE BUILDINGS

A typical RC building is made of horizontal members (beams and slabs) and vertical members (columns and walls) and supported by foundations that rest on the ground. The system consisting of RC columns and connecting beams is called a RC frame.

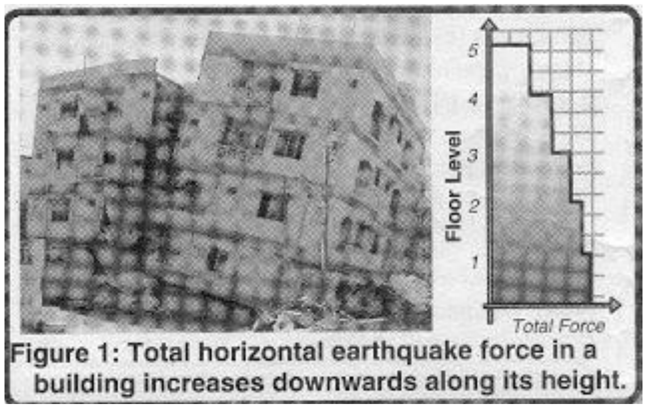


Figure no. - 2

The RC frame participates in resisting earthquake forces. Earthquake shaking generates inertia forces in the building, which are proportional to the building mass. As most of the building mass is present at the floor levels, earthquake induced inertia forces mainly develop at the floor levels. These forces travel down through slabs to beams, beams to columns and walls and then to foundations from where they are dispersed to the ground. As the inertia forces collect downward from the top of the building, the columns and walls at the lower storey experience high earthquake induced forces and are therefore designed to be stronger than the storey above.

2.3 Roles of floor slabs and masonry walls:

Floor slabs are horizontal elements, which facilitate the functional use of buildings. Usually, beams and slabs at single storey level are cast together. In residential multistoried buildings, the thickness of slab is only about 110mm-150mm. when beams bend in vertical direction during earthquakes, these thin slabs bend along with them. When beams move in horizontal direction, the slab usually forces the beams to move together with it.

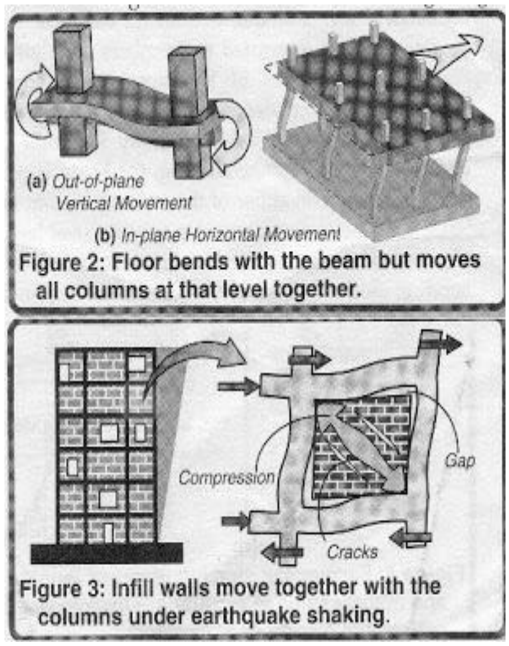


Figure no. - 3

In most of the buildings, the geometric distortion of the slab is negligible in the horizontal plane; the behavior is known as rigid diaphragm action. After columns and floors in a RC building are cast and the concrete hardens, vertical spaces between columns and floors are usually filled in with masonry walls to differentiate a floor area into functional spaces. Normally, these masonry walls are called infill walls, are not connected to adjoining RC beams and columns. When the columns receive horizontal forces at floor levels, they try to move in the horizontal direction, but masonry wall tend to resist this movement.

Due to their heavy weight and thickness, these walls develop cracks, once their ability to carry horizontal load is exceeded. Thus, infill walls act like sacrificial fuse in the buildings, they develop cracks under rigorous ground shaking but help share the load of beams and columns until cracking.

2.4 Strength hierarchy:

For a building to remain safe during earthquake shaking columns (which receive forces from beams) should be stronger than beams and foundations (which receive forces from columns) should be stronger than columns. Further the connections between beams and columns, columns and foundations should not fail so that beams can safely transfer forces to columns and columns to foundation.

When this approach is adopted in the design, damage is likely to occur first in the beams. When beams are detailed properly to have large ductility, the building as a whole can deform by large amounts despite progressive damage caused due to following yielding of beams.

If columns are made weaker, localized damage can lead to the collapse of building, although columns at storey above remain almost undamaged.

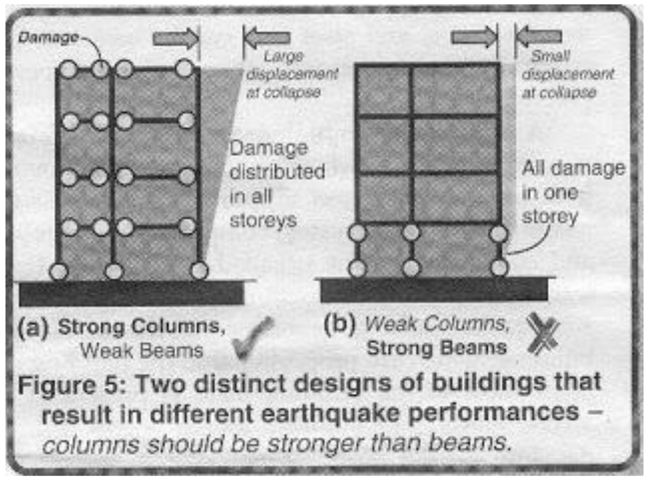


Figure no. 4

2.5 SEISMIC DESIGN PHILOSOPHY

Severity of ground shaking at a given position during earthquake can be minor, moderate and strong. Relatively speaking, minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely. For instance, on average annually about 800 earthquakes of magnitude 5.0-5.9 occurs in the world, while the number is only 18 for the magnitude ranges 7.0-7.9. Since it costs money to provide additional earthquake safety in buildings, a conflict arises ‘should we do away with the design of buildings for earthquake effects? Or should we design the building to be earthquake proof wherein there is no damage during strong but rare earthquake shaking. Clearly the formal approach can lead to a major disaster and second approach is too expensive. Hence the design philosophy should lie somewhere in between two extremes.

The earthquake design philosophy may be summarized as follows*:*

1. Under minor, but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however the building parts that do not carry load may sustain repairable damage.
2. Under moderate but occasional shaking, the main member may sustain repairable damage, but the other parts of the building may be damaged such that they may even have to be replaced after the earthquake. Under strong but rare shaking, may sustain severe (even irreparable) damage, but the building should not collapse.

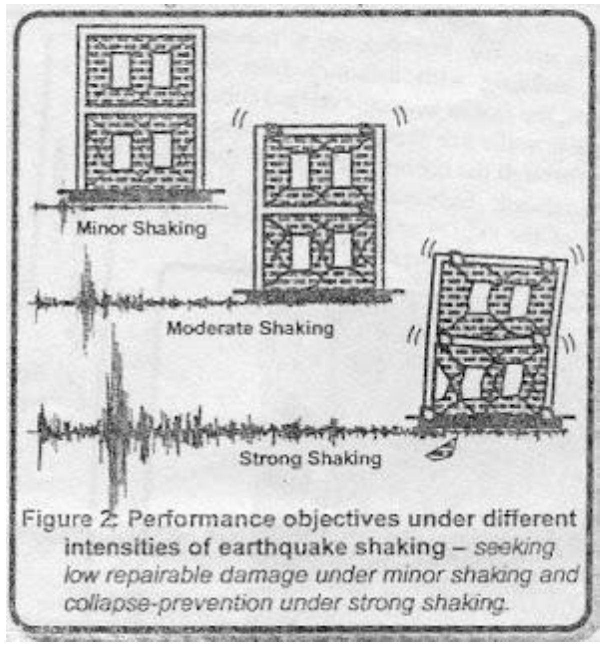


Figure 2: Performance objectives under different intensities of earthquake shaking – *seeking low repairable damage under minor shaking and collapse-prevention under strong shaking.*

Figure no. - 5

Thus after minor shaking, the building will be operational within a short time and repair cost will be small and after moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But, after a strong earthquake, the building may become dysfunctional for further use, but will stand so that people can be evacuated and property recovered.

The consequences of damage have to be kept in view in the design philosophy. For example, important buildings like hospitals and fire stations play a critical role in post earthquake activities and must remain functional immediately after earthquake. These structures must sustain very little damage and should be designed for a higher level of earthquake protection. Collapse of dams during earthquake can cause flooding in the downstream reaches, which itself can be a secondary disaster. Therefore, dams and nuclear power plants should be designed for still higher level of earthquake motion.

2.6 What is Ductility Detailing?

For a given seismic demand (dynamic forces generated by a given ground motion), if your structure (or your design) does not remain elastic, i.e., it undergoes plasticity/fracture/damage, then stiffness could drop dramatically, deformations will increase significantly. Under these increased deformations, you should ensure that your structure remains stable without collapsing, i.e., it should not loose vertical load carrying capacity; you have to detail it in such a way that it can undergo large deformations without collapsing. The ability of a structure to undergo large deformations without collapsing is called ductility, and the detailing of the structure that enables the structure to have large ductility is called ductile or ductility detailing. The word ductility has a strict definition in material science where it is defined as the ratio of ultimate strain to yield strain of the material. The term is loosely used in earthquake engineering to indicate the degree to which an assembled structure that is damaged can undergo large deformations without collapsing.

When do you need Ductility Detailing?

If you can design your structure to remain elastic under the maximum expected level of earthquake shaking, then there is no need for ductility detailing. However, engineers recognized long ago that it was not practically possible to come up with economically or architecturally viable designs if the structures were to remain elastic during maximum considered earthquake motions (every structure would look like a Fort Knox). So, the next best option was to allow the structure to undergo damage (plasticity, fracture, crushing, etc.), but make sure that the structure does not lose its vertical load carrying capacity as it is undergoing large deformations when damaged. For example, if you want concrete columns to continue carrying vertical loads even when there is significant cracking, concrete crushing and/or steel yielding, you would want to confine the concrete (keep chunks of concrete from falling out of the steel cage) and you would want to prevent buckling of the longitudinal bars. You can achieve both of these characteristics by providing very closely spaced closed ties with 135 degree hooks to sturdily anchor them against a longitudinal bar.

Because the material has now become compliant the stiffness forces in the components and structure will drop. In other words, if the structure were to remain elastic the internal forces (and the total base shear which is the sum of the internal shear forces in all the vertical load carrying elements) would be far higher than if the structure were to be damaged. This reduction in base shear arising out of the fact that we have allowed the structure to get damaged is embodied in the "Dynamic Response Modification" factor or the "Ductility" factor, Rd, in building codes worldwide. Rd is specific to each type of lateral force resisting system. Rd is higher for systems where ductility detailing can enable the system to undergo larger deformations without collapsing when damaged,

CHAPTER 3: PROJECT PLAN AND MODEL

3.1 AUTOCAD PLAN

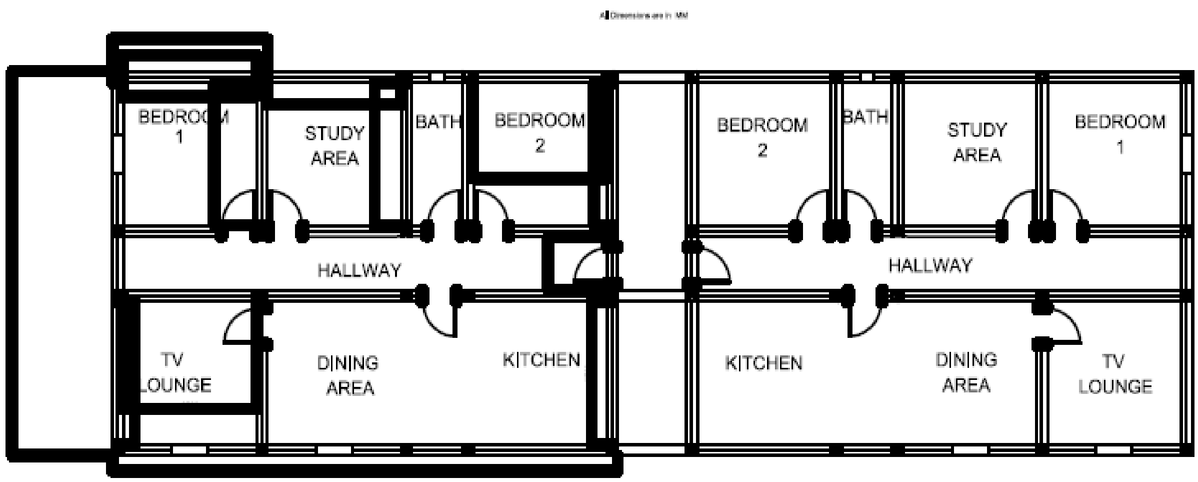


Figure no. - 6

3.1.1 AUTOCAD FRONT VIEW

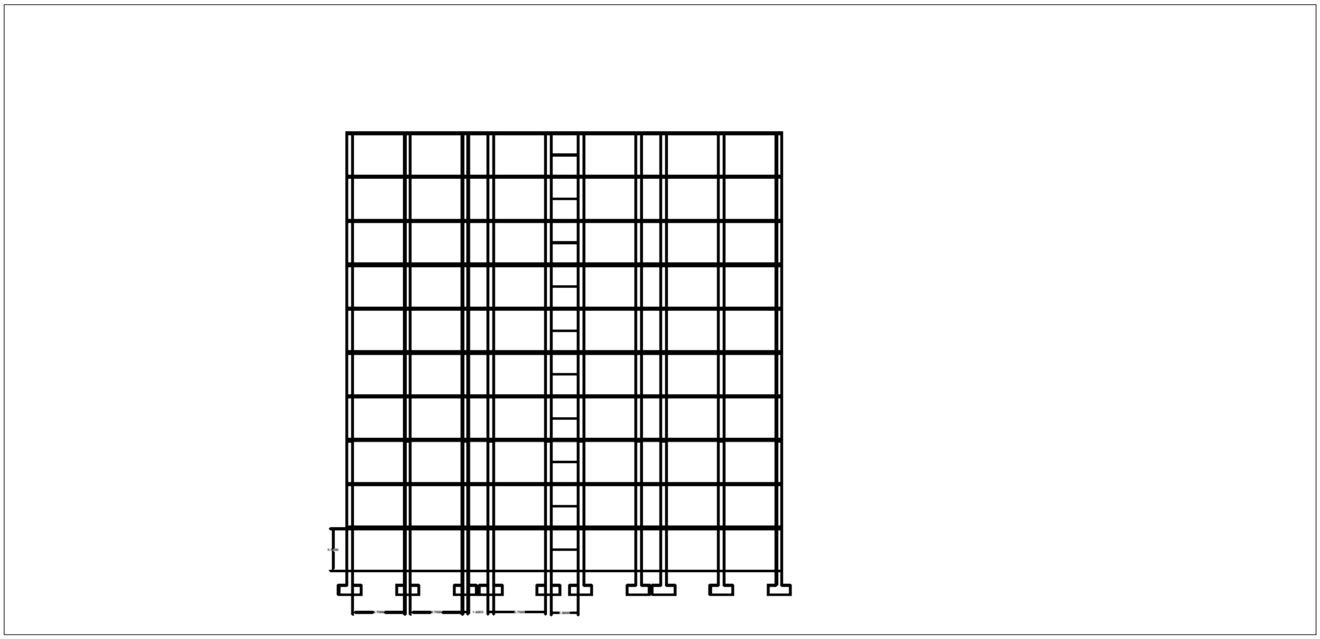


Figure no. - 7

3.1.2 AUTOCAD SIDE VIEW

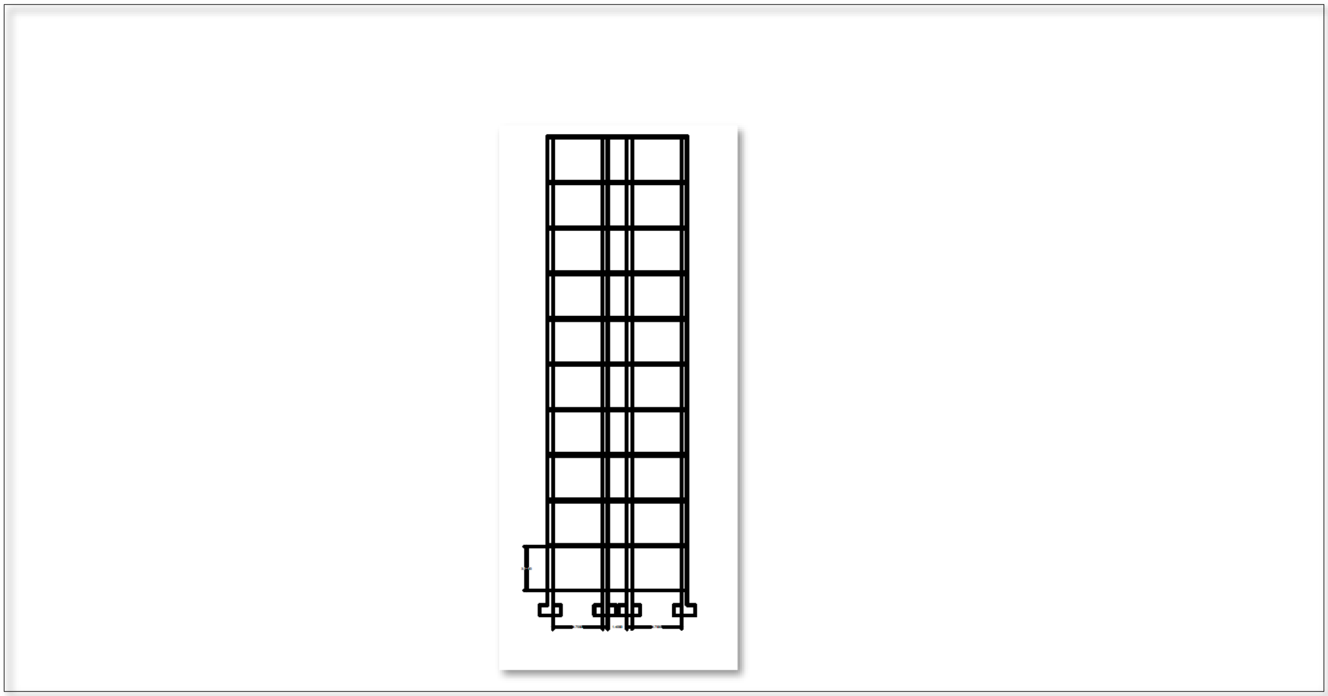


Figure no. - 8

3.2 STAAD Model

A) Modelling

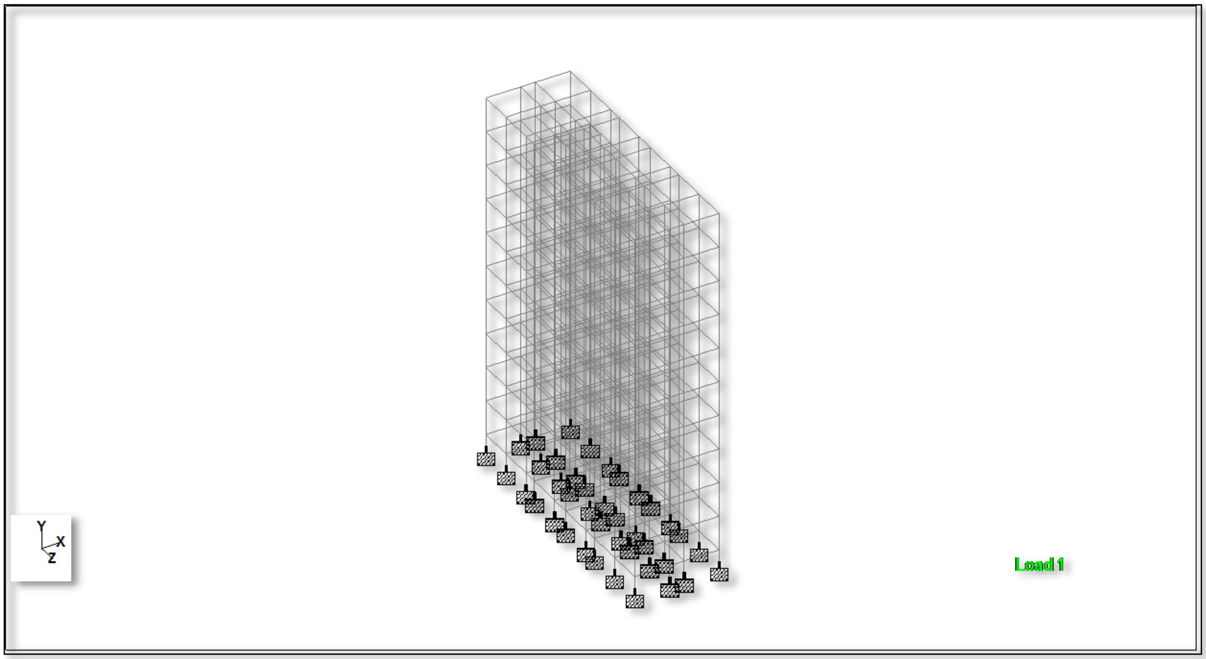


Figure no. - 9

3D-Rendered View

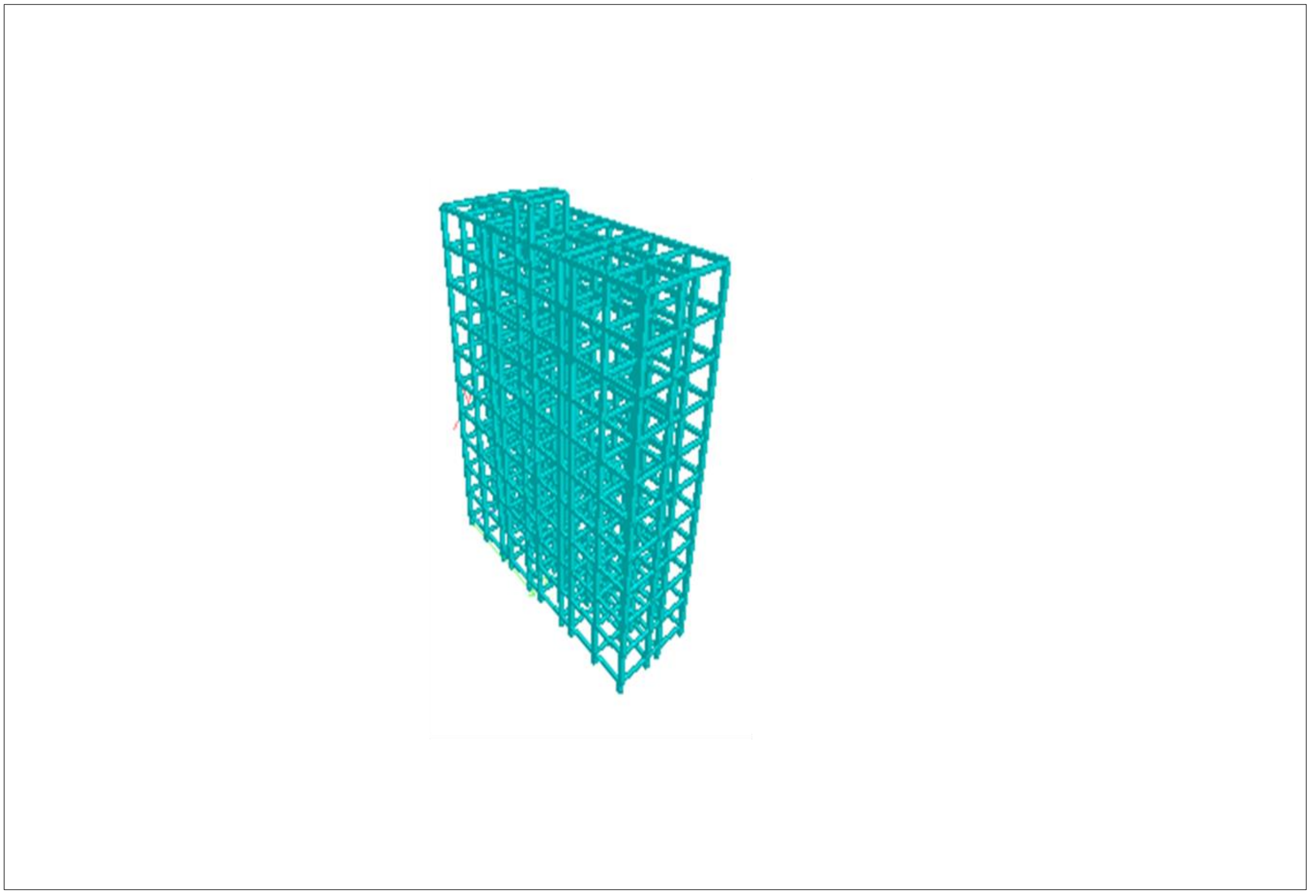


Figure no.- 10

C) Load assigned

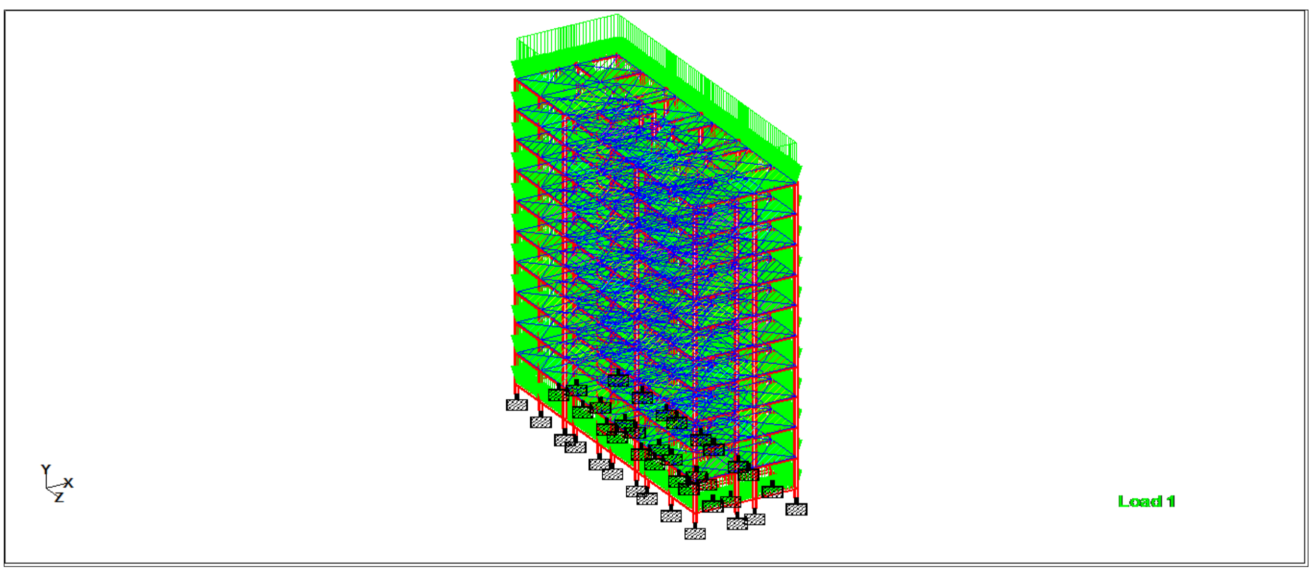


Figure no.- 11

4. LOADING

According to the specifications given in IS:875-1987:Part-1,2,3 and IS:1893:Part-1-2002 the following loads will be acting on the structure.

1. Dead load- IS:875-1987 (Part-1)
2. Live load- IS:875-1987 (Part-2)
3. Wind load- IS:875-1987 (Part-3)
4. Earthquake load IS:1893 (Part-1):2002

4.1 Dead Load

- Dead load refers to the self weight of the structure covering the weight of each and every element of the structure like the beam, column, slab and the walls.
- The unit weight of various building material are given in IS:875 (part 1).

Description	**Dimensions(mm)**	**Unit wt. of material used* (KN/m*2)**	**Dead load (KN/m)**
1. Columns	**400*400**	**25**	**4**
2. Floor Beams	**250*400**	**25**	**2.5**
3.Terrace and ground beams	**250*350**	**25**	**2.1875**
4.Slab	**150**	**25**	**3.75**
5.Brickwall	**230**	**19 for wall and 20 for plaster**	**4.85*3.5= 16.795**

Table no. 2

4.2 Imposed Load

- The use of the term ‘live load’ has been modified to ‘imposed load’ to cover not only the physical contribution due to people but also due to nature of occupancy , the furniture and other equipments which are a part of the character of the occupancy. (Clause-0.3.2)
- The IS 875 (part 2) gives specification of various live loads.
- Residential Buildings - These shall include any building in which sleeping accommodation is provided for normal residential purposes with or without cooking or dining or both facilities. It includes one multi-family dwellings, apartment houses flats), lodging or rooming houses, restaurants, hostels, dormitories and residential hotels. (Clause 2.2.1 of IS:875-1987 (Part-2)).

Component	**Terrace** **DL+LL**	**Typical** **DL+LL**
Self(150mm)	3.75+0	3.75+0
Water Proofing	2+0	0+0
Floor Finish	1+0	1+0
Live Load	0+1.5	0+4

Table no. - 3

4.3 Wind Load

The part-3 deals with wind loads to be considered when designing buildings, structures and components thereof. In its second revision in 1987, the important modifications were made from those covered in the 1964 version of IS: 875.
Wind causes a random time-dependent load, which can be seen as a mean plus a fluctuating component.
All structures will experience dynamic oscillations due to the fluctuating component (gustiness) of wind. In short rigid structures these oscillations are insignificant, and therefore can be satisfactorily treated as having an equivalent static pressure. A structure may be deemed to be short and rigid if its natural time period is less than one second. The more flexible systems such as tall buildings undergo a dynamic response to the gustiness of wind.

Basic Wind Speed (V_b)

Figure 1 gives basic wind speed map of India, as applicable at 10 m height above mean ground level for different zones of the country. Basic wind speed is based on peak gust speed averaged over a short time interval of about 3 seconds and corresponds to 10m height above the mean ground level in an open terrain (Category 2).

Design Wind Speed (V_z)

The basic wind speed for any site shall be obtained from Fig. 1 and shall be modified to include the following effects to get design wind speed, Vz at any height, Z for the chosen structure: (a) Risk level, (b) Terrain roughness and height of structure, (c) Local topography, and (d) Importance factor for the cyclonic region. It can be mathematically expressed as follows:
$Vz = Vb\ k1\ k2\ k3\ k4$,
Where, Vz = design wind speed at any height z in m/s,
$k1$ = probability factor (risk coefficient) (see5.3.1),
$k2$ = terrain roughness and height factor (see 5.3.2),
$k3$ = topography factor (see 5.3.3)
$k4$ = importance factor for the cyclonic region (see 5.3.4)
NOTE: The wind speed may be taken as constant upto a height of 10 m. However, pressures for buildings less than 10m high may be reduced by 20% for stability and design of the framing

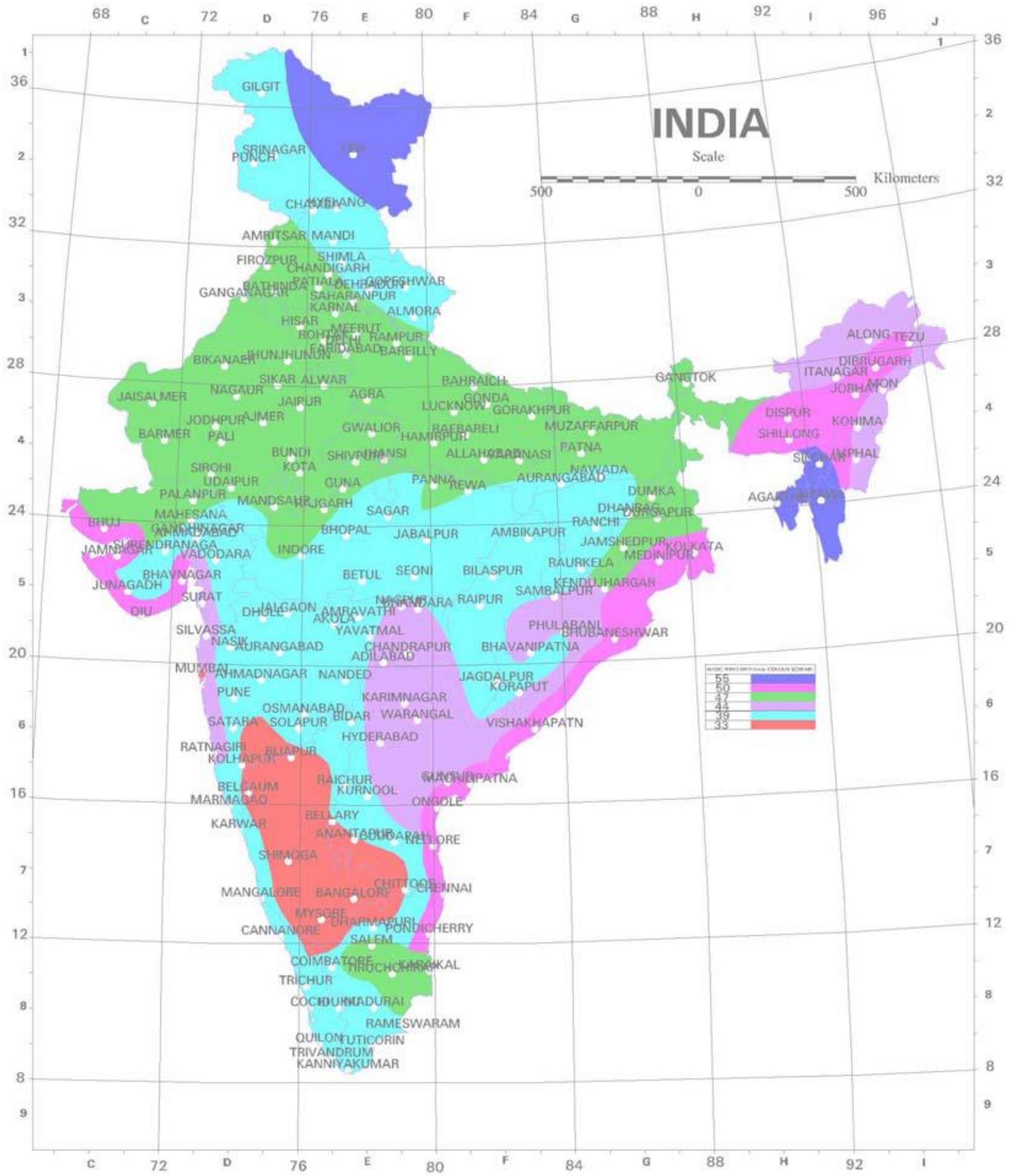


Figure no. - 12: Basic wind speed in m/s (based on 50 year return period)

Risk coefficients k1 for different classes of structures in different wind speed zones [*Clause 5.3.1*]

Table no. 4

Class of Structure	Mean Probable design life of structure in years	k_1 factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
Temporary sheds, structures such as those used during construction operations (for example, formwork and false work), structures during construction stages, and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings, etc.	25	0.94	0.92	0.91	0.90	0.90	0.89
Important buildings and structures such as hospitals, communication buildings, towers and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

k2* factors to obtain design wind speed variation with height in different terrains *[Clause 5.3.2.2]

Table no. - 5

Height (*z*)	Terrain and height multiplier (k_2)			
(*m*)	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
10	1.05	1.00	0.91	0.80
15	1.09	1.05	0.97	0.80
20	1.12	1.07	1.01	0.80
30	1.15	1.12	1.06	0.97
50	1.20	1.17	1.12	1.10
100	1.26	1.24	1.20	1.20
150	1.30	1.28	1.24	1.24
200	1.32	1.30	1.27	1.27
250	1.34	1.32	1.29	1.28
300	1.35	1.34	1.31	1.30
350	1.37	1.36	1.32	1.31
400	1.38	1.37	1.34	1.32
450	1.39	1.38	1.35	1.33
500	1.40	1.39	1.36	1.34
NOTE: For intermediate values of height z and terrain category, use linear interpolation.				

Terrain – Selection of terrain categories shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Terrain in which a specific structure stands shall be assessed as being one of the following terrain categories:

Category 1– Exposed open terrain with a few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5 m.

NOTE – This category includes open sea coasts and flat treeless plains.

b) Category 2 – Open terrain with well scattered obstructions having height generally between 1.5 and 10m.

NOTE – This is the criterion for measurement of regional basic wind speeds and includes airfields, open parklands and undeveloped sparsely built-up outskirts of towns and suburbs. Openland adjacent to seacoast may also be classified as Category 2 due to roughness of large sea waves at high winds.

c) Category 3– Terrain with numerous closely spaced obstructions having the size of building-structures up to 10m in height with or without a few isolated tall structures.

NOTE – This category includes well-wooded areas, and shrubs, towns and industrial areas fully or partially developed.

d) Category 4 –Terrain with numerous large high closely spaced obstructions.

NOTE – This category includes large city centers, generally with obstructions taller than 25m and well-developed industrial complexes.

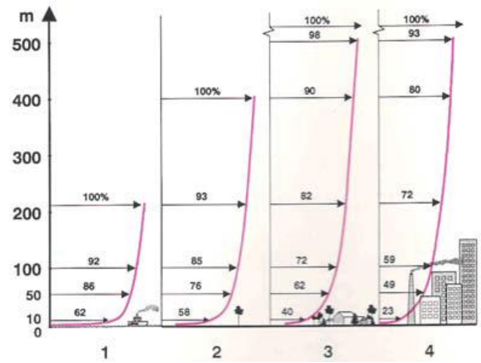
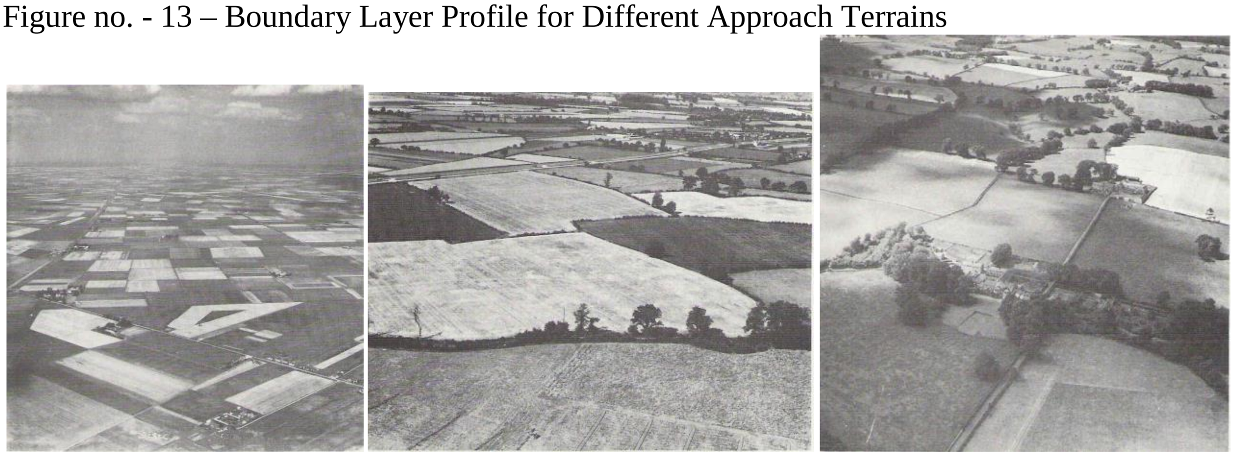


Figure no. - 13 – Boundary Layer Profile for Different Approach Terrains



Topography (k3 factor): No increase in wind speed is indicated for upwind ground slopes upto 3º, while a maximum increase of 36% is specified for slopes beyond 17o. Maximum effect is seen to occur at the crest of a cliff or escarpment and reduces gradually with distance from the crest. Also, locally k3 reduces from the base of a structure to its top.

Importance Factor for Cyclonic Region (k4)

Cyclonic storms usually occur on the east coast of the country in addition to the Gujarat coast on the west. In order to ensure greater safety of structures in this region (60 km wide on the east coast as well as on the Gujarat coast), the following values of k4 are stipulated, as applicable according to the importance of the structure:

Structures of post–cyclone importance 1.30
Industrial structures 1.15
All other structures 1.00
For non-cyclonic regions, the factor k4 shall obviously be taken as 1.0.

Design Wind Pressure

The wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind speed:

$pz = 0.6\ Vz^{2}$

where

pz = wind pressure in N/m2 at height z, and
Vz = design wind speed in m/s at height z.
The design wind pressure pd can be obtained as,
pd = Kd. Ka. Kc. Pz where
Kd = Wind directionality factor
Ka = Area averaging factor
Kc = Combination factor

Wind Directionality Factor, K_d

Considering the randomness in the directionality of wind and recognizing the fact that pressure or force coefficients are determined for specific wind directions, it is specified that for buildings, solid signs, open signs, lattice frameworks, and trussed towers (triangular, square, rectangular) a factor of 0.90 may be used on the design wind pressure. For circular or near – circular forms this factor may be taken as 1.0. For the cyclone affected regions also, the factor *Kd* shall be taken as 1.0.

Area Averaging Factor, K_a

Pressure coefficients given in Section 6.2 area result of averaging the measured pressure values over a given area. As the area becomes larger, the correlation of measured values decrease and vice-versa. The decrease in pressures due to larger areas may be taken into account as given.

Table no. - 6

Tributary Area (A) (m^2)	Area Averaging Factor (K_a)
≤ 10	1.0
25	0.9
≥ 100	0.8

Wind load calculation

p_d	z	V_z	K_1	K_2	K_3	V_b	p_z	K_a	K_d	K_c
1168.61472	4.7	38.22	1	0.98	1	39	1460.768	0.8	1	1
1168.61472	8.2	38.22	1	0.98	1	39	1460.768	0.8	1	1
1201.2748	11.7	38.7504	1	0.99	1	39	1501.594	0.8	1	1
1270.92813	15.2	39.858	1	1.02	1	39	1588.66	0.8	1	1
1321.66485	18.7	40.6458	1	1.04	1	39	1652.081	0.8	1	1
1369.77731	22.2	41.379	1	1.06	1	39	1712.222	0.8	1	1
1415.33583	25.7	42.0615	1	1.08	1	39	1769.17	0.8	1	1
1461.63963	29.2	42.744	1	1.1	1	39	1827.05	0.8	1	1
2629.38312	32.7	57.33	1	1.47	1	39	3286.729	0.8	1	1
2781.78002	36	58.968	1	1.51	1	39	3477.225	0.8	1	1

Table no. - 7

Where:-
V_b = Basic Wind Speed
V_z = Design Wind Speed
k_1 = probability factor (risk coefficient)
k_2 = terrain roughness and height factor
k_3 = topography factor
k_4 = importance factor for the cyclonic region
p_z = wind pressure in N/m2 at height z
p_d =Design Wind Pressure
K_d = Wind directionality factor
K_a = Area averaging factor
K_c = Combination factor

4.4 Seismic Loading

Himalayan-Nagalushai region, Indo-GangeticPlain, Western India, Kutch and Kathiawarregions are geologically unstable parts of the country, and some devastating earthquakes of the world have occurred there. **A** major part of the peninsular India has also been visited by strong earthquakes, but these were relatively few in number occurring at much larger time intervals at any site, and had considerably lesser intensity. The earthquake resistant design of structures taking into account seismic data from studies of these Indian earthquakes has become very essential, particularly in view of the intense construction activity all over the country. It is to serve this purpose that IS 1893 : 1962 ‘Recommendations for earthquake resistant design of structures’ was published and revised first time in 1966.
The seismic zone map is revised with only four zones, instead of five. Erstwhile Zone I has been merged to Zone 11. Hence, Zone I does not appear in the new zoning; only Zones II, III, IV and V .

In the seismic zoning map, Zone I and II of the contemporary map have been merged and assigned the level of Zone 11. The Killari area has been included in Zone III and necessary modifications made, keeping in view the probabilistic hazard evaluation. The Bellary isolated zone has been removed. The parts of eastern coast areas have shown similar hazard to that of the Killari area, the level of Zone II has been enhanced to Zone III and connected with Zone III of Godawari Graben area.

The seismic hazard level with respect to ZPA at 50 percent risk level and 100 years service life goes on progressively increasing from southern peninsular portion to the Himalayan main seismic source, the revised seismic zoning map has given status of Zone III to Narmada Tectonic Domain, Mahanandi Graben and Godawari Graben. This is a logical normalization keeping in view the apprehended higher strain rates in these domains on geological consideration of higher neotectonic activity recorded in these areas.

Zone Factor (Z)

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factor included in this standard are reasonable estimate of effective peak ground acceleration .For zone 3 Z value is 0.16.

Importance Factor (I)

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterised by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.For hotels I value is 1.5.

Response Reduction Factor (R)

It is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force. For OMRF R value is 3and for steel frames R value is 5.

Fundamental Natural Period (T_a)

It is the first (longest) modal time period of vibration.

T_a =0.075$h^{.075}$ for RC frame building

T_a =0.085$h^{.075}$ for steel frame building.

Response Spectrum

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

Design Seismic Base Shear (V_b)

It is the total design lateral force at the base of a structure.

V_b=A_hxW

A_h=design horizontal acceleration spectrum

W=seismic weight of building

A_h=ZxIxS_a/(2xRxg)

Seismic Zone – 4 (Shimla)

Zone Factor (Z) – 0.24 (Annex-E)

Importance Factor (I) – 1 for Residential Building (Table 6)

Response Reduction Factor (R) – 5 for Special RC Moment Resisting Frame (Table 7)

Floor Area - 30.9x10.4 = 321.36 m^2

Taking Live Load – 4kN/m^2

Dead Load – 3.75KN/m^2 (Slab)

Floors – W_1= Area x (DL+0.5LL) (Table-8)

= 321.36 x (3.75+ 2) = 2170 kN

Roof – W_2=Area x (DL) = 321.36 x (3.75) = 1205 kN

Total Seismic Weight of structure = (9xW_1) + W_2 = 20,735 kN

Fundamental Period - T_a= 0.09/$d^{0.5}$

d = 30.9m and 10.4m ; h = 36.5m

T_a = 0.09(36.5)/$30.9^{0.5}$ = 0.59sec

S_a/g = 1.75 [Fig. 2 - Type 1 (Hard Soil or Rock)]

$A_{h\,=}$ 0.042

V_b=A_h x W = 0.042x20715 = 870.8kN

T_a = 0.09(36.5)/$10.4^{0.5}$ = 1.01sec

S_a/g = 1 [Fig. 2 - Type 1 (Hard Soil or Rock)]

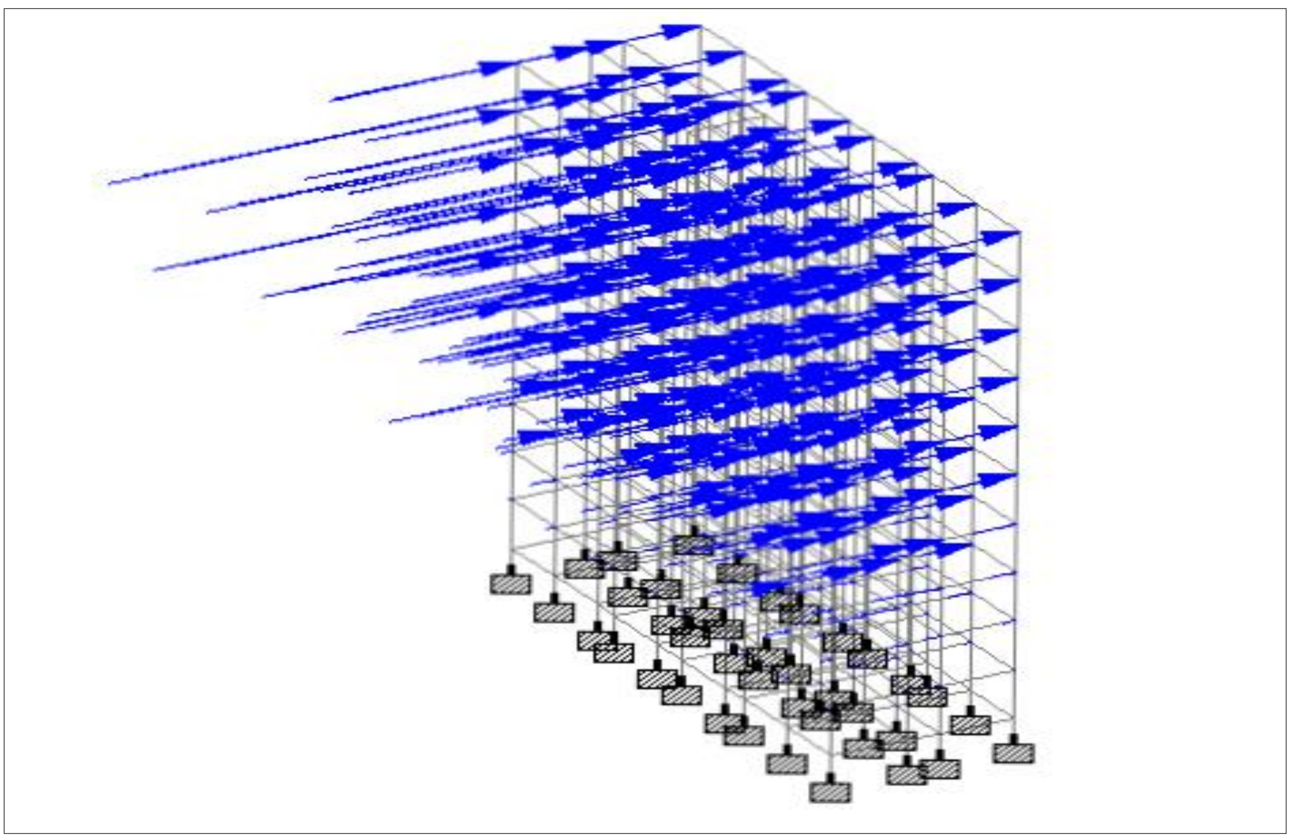
$A_{h\,=}$ 0.024

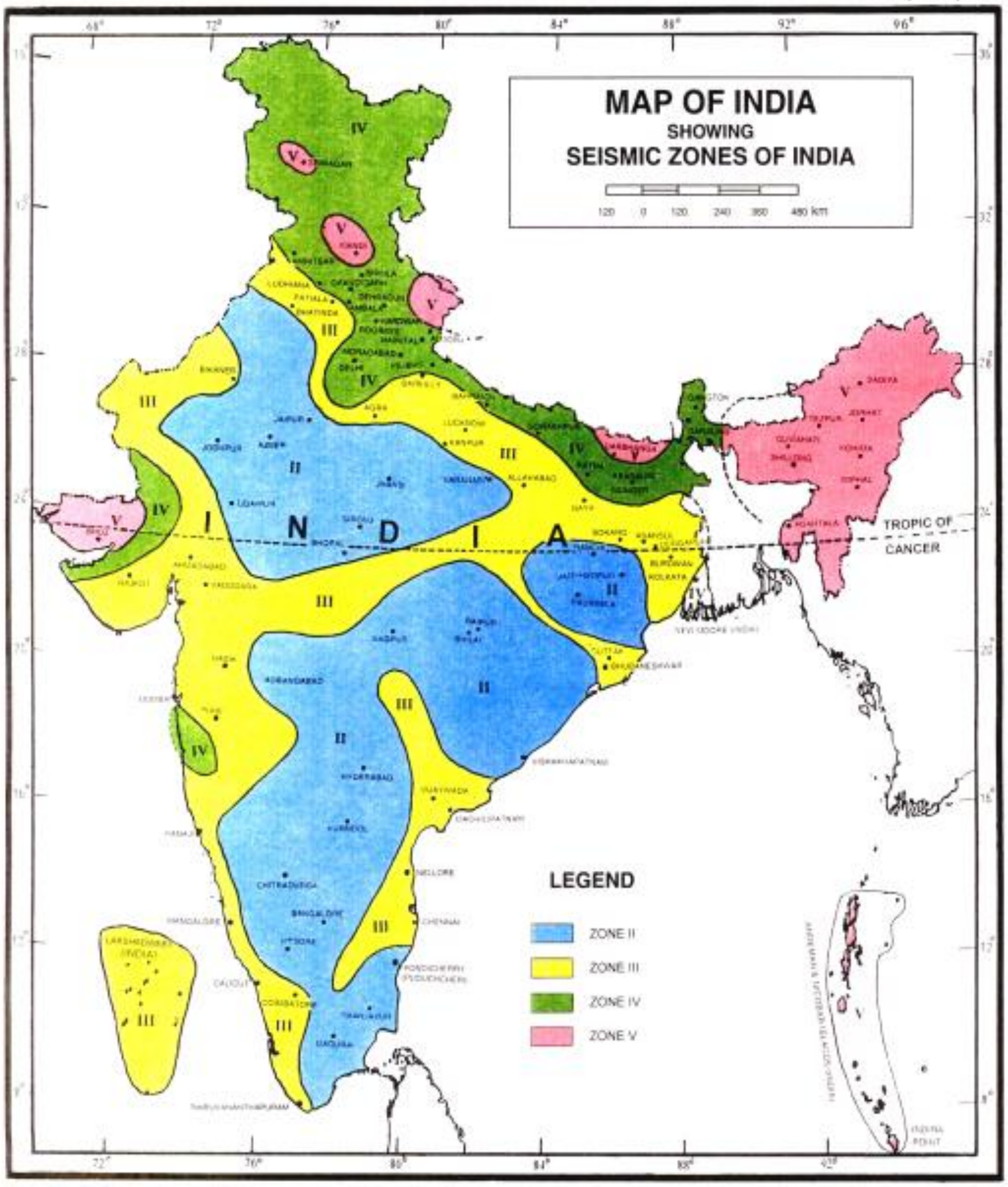
V_b=A_h x W = 0.024x20715 = 497.6kN

Manual Seismic load calculation(Table no. - 8)

Storey	W_i	h_i	$W_ih_i^2$x1000	$W_ih_i^2$/Total	Lateral Force at ith level for EL in direction(kN)	Lateral Force at ith level for EL in direction(kN).
9	1205	36.5	1605.3	0.163	142	81
8	2170	32.85	2314.6	0.237	206	118
7	2170	29.2	1850.2	0.187	163	93
6	2170	25.55	1416.5	0.143	125	71
5	2170	21.9	1040.7	0.105	91	52
4	2170	18.25	722.7	0.073	64	36
3	2170	14.6	762.5	0.046	40	23
2	2170	10.95	260.2	0.026	23	13
1	2170	7.3	115.6	0.011	10	6
G	2170	3.65	28.9	0.002	2	1
TOTAL			9844.2			

Figure no. - 14





NOTE : Towns falling at the boundary of zones demarcation line between two zones shall be considered in High Zone.

- Based upon Survey of India map with the permission of the Surveyor General of India.
- The responsibility for the correctness of internal details rests with the publisher.
- The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base line.
- The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.
- The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.
- The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

FIG. 1

CHAPTER-5 ANALYSIS AND DESIGN

5.1 RCC Beam Design-

Manual calculations-

M=100.083 kN-m F_y = 415MPa F_{ck} = 40MPa

D=400 mm B=250 mm Cover=25 mm

Step-1

$X_{u\ lim}$=700d/(.87xFy+1100)

$X_{u\ lim}$=192mm

M=.36 BF_{ck} BX_u(D-0.42xX_u)

X_u=76.92mm

As X_u<$X_{u\ lim}$

Hence under-reinforced section.

Step-2

Reinforcement (Tesnion side)

A_{st} = .5 F_{ck}/ F_y(1-(1-4.6M/F_{ck}BD^2)$^{.5}$ x BD

A_{st} = 751mm^2

Provide 14 mm bars in tension zone

No. of bars =751x4/π x 14x14

No. of bars = 4.87

Hence provide 6 bars.

Reinforcement(Compression side)

MR=65.3 kN-m

X_u=47.44mm

A_{st}=475mm^2

Provide 14 mm bars in compression zone

No. of bars=3.08

Hence provide =4 bars

According to code it recommends that at least one third of the positive moment reinforcement in simple members shall extend along the same face of the member into the support.

However bend one bar upwards, at a distance x_1 from the support .

B.M. at x_1 =W_ulx_1/2 - W_u x_1^2/2 this should be two third of maximum B.M. which is equal to 2W_uL^2/8 which gives x_1 = 0.211L=0.844m.

Step-3 Shear check

V_u=97.194KN δ_v =V_u/B x d=0.971N/mm^2

100A_{st}/BD =0.159%

Assuming 10 mm dia bars.

From table 7.1 IS 456:2000

δc =0.38N/mm^2

Shear reinforcement required as

$\delta_v > \delta c$

$V_{us}=(\delta v-\delta c) \times B \times d$

$V_{us}=39$ kN

Provide striups at 90°

$S_v = 0.87F_yA_{sv}D/V_{us}$

Spacing Sv=290 mm

Hence provide 10 mm bars @ 290 mm spacing

Step-4 Check for Development length

Since the beam is supported on wall, the compressive reaction will confine the reinforcement.

$Ld=.87 \times F_y \times ø/4 \times \delta_{bd}$

Ld=297 mm $\delta_{bd}=1.9*1.6=3.04$ N/mm^2 From table 8.1

According to clause 26.2.3.3

Provide 2 bent up bars at the centre inclined at 45 degree to the horizontal.

Step-5 Deflection check

L/d=10<26 Hence ok

Software Design Results for beam no. 551-

B E A M N O. 551 D E S I G N R E S U L T S

M40 Fe415 (Main) Fe415 (Sec.)

LENGTH: 4000.0 mm SIZE: 250.0 mm X 400.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	1000.0 mm	2000.0 mm	3000.0 mm	4000.0 mm
TOP REINF.	642.12 (Sq. mm)	189.46 (Sq. mm)	0.00 (Sq. mm)	189.46 (Sq. mm)	748.73 (Sq. mm)
BOTTOM REINF.	0.00 (Sq. mm)	234.25 (Sq. mm)	272.38 (Sq. mm)	189.46 (Sq. mm)	0.00 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	1000.0 mm	2000.0 mm	3000.0 mm	4000.0 mm
TOP REINF.	9-10í 2 layer(s)	3-10í 1 layer(s)	2-10í 1 layer(s)	3-10í 1 layer(s)	10-10í 2 layer(s)
BOTTOM REINF.	2-10í 1 layer(s)	3-10í 1 layer(s)	4-10í 1 layer(s)	3-10í 1 layer(s)	2-10í 1 layer(s)

```
                    SUMMARY OF PROVIDED REINF. AREA
---------------------------------------------------------------------------
SECTION       0.0 mm     1000.0 mm     2000.0 mm     3000.0 mm     4000.0 mm
---------------------------------------------------------------------------
 TOP          9-10í         3-10í         2-10í         3-10í        10-10í
REINF.    2 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)    2 layer(s)

BOTTOM        2-10í         3-10í         4-10í         3-10í         2-10í
REINF.    1 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)

SHEAR    2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8
REINF.   @ 130 mm c/c  @ 130 mm c/c  @ 130 mm c/c  @ 130 mm c/c  @ 130 mm c/
---------------------------------------------------------------------------



SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPO

SHEAR DESIGN RESULTS AT   565.0 mm AWAY FROM START SUPPORT
  VY =     67.30 MX =    -0.04 LD=    12
  Provide 2 Legged   8í  @ 130 mm c/c

SHEAR DESIGN RESULTS AT   565.0 mm AWAY FROM END SUPPORT
  VY =   -73.82 MX =     0.00 LD=    11
  Provide 2 Legged   8í  @ 130 mm c/c
```

VBA Design Results for Beam no. 551

1	Beam design			Fixed Parameters		
2						
3				Column1	Column	Column
4	Characterstic strenght of concrete	40	N/mm^2			
5	Grade of steel	415	N/mm^2	Characterstic strenght of concrete	40	N/mm2
6				Grade of steel	415	N/mm2
7	Bending moment in beam	100.03	kNm	Effective cover at bottom	25	mm
8	Breadth of beam	250	mm	Effective cover at top	25	mm
9	Effective depth of beam	400	mm			
10	Dia of bar in tension	14	mm			
11	Dia of bar in compresion	14	mm			
12	Mu_{lim}	220.9263	kNm			
13						
14	Singly reinforced section			Input Parameters		
15	Area of steel in tension	751.3861	mm^2			
16	Spacing of bar	204.4288	mm	Column1	Column	Column
17	Number of bars	4.76288				
18				Bending moment in beam	100.03	kNm
19				Breadth of beam	250	mm
20	Doubly reinforced section			Effective depth of beam	400	mm
21	Effective cover at bottom	30	mm	Dia of bar in tension	14	mm
22	Effective cover at top	20	mm	Dia of bar in compresion	14	mm

20	Doubly reinforced section			Effective depth of beam	400	mm
21	Effective cover at bottom	30	mm	Dia of bar in tension	14	mm
22	Effective cover at top	20	mm	Dia of bar in compresion	14	mm
23						
24	Fsc (steel in compression)	474.3	mm^2			
25						
26	Ast in compression	0	mm^2			
27				Output Parameters		
28	Ast in tension	0	mm^2			
29	Spacing of bar	87.44073	mm	Column1	Column	Column
30	Number of bars in tension	3		Area of steel in tension	751.3861	mm^2
31	Number of bars in compression	1.172042		Spacing of bar	204.4288	mm
32				Number of bars	4.76288	
33				Fsc (steel in compression)	353.3	mm2
34	shear force	150	kN	Ast in compression	0	mm2
35	tc			Ast in tension	0	mm2
36	diameter of reinforcement			Spacing of bar	87.44073	mm
37	Tcmax			Number of bars in tension	3	
38	legged	2		Number of bars in compression	1.172042	
39				shear force	150	kN
40	spacing					

5.2 Slab Design-

Slab Design(Floor)
Design of critical case,
F_y=415N/mm2 F_{ck}=40N/mm^2
L_x=4m L_y=4m b=1000mm
L_x/L_y=1
Assume thickness of slab=150mm
Dead load=0.15x25=3.75kN/m^2
Live load=4kN/m^2
Finishing load=1kN/m^2
Total factored load(w)=1.5*(4+1+3.75)=11.625KN/m^2
M_x(+)=άx(+)*w*(Lx)^2=.04*11.625*4.15*4.15=10.038KNm/m
M_y(+)=άy(+)*w*(Lx)^2=.035*11.625*4.15*4.15=8.78KNm/m
M_x(-)=άx(+)*w*(Lx)^2=.056*11.625*4.15*4.15=12.673KNm/m
M_y(-)=άy(+)*w*(Lx)^2=.047*11.625*4.15*4.15=11.79KNm/m
Xm=.48*150=86.4mm
d=(M_x(-)/(.138xFck x b))$^{.5}$ =70mm
From the requirements of deflection keep total depth 150 mm

Area of tension steel A_{ty} along long span-
=0.5Fck/Fy x [1-(1-4.6My(-)/(Fck x b x d x d))^.5] x b x d
$A_{t\ y}$=245.189mm^2
Minimum Tension Steel= .0012 x d x b=180mm^2
Adopt $A_{t\ y}$=245.189mm^2
$A_{t\ x}$=$A_{t\ y}$
Adopt 8mm Bars
Spacing=1000 X 50.3/245.189=204mm

Check for shear and development length in short span-
Sf At Long Edges=Wu*Lx*(R/2+R)=11625N
Nominal Shear Stress At Long Edges=11625/(1000*156)
At long edges the diameter of bars should be so restricted that following condition is satisfied

1.3Mu/Vu +Lo>Ld
Ast at supports of short span=1000*50.3/251.5 =201.2mm^2
Xu=0.87Fy*Ast/(0.36*Fck*b)
Mu=0.87Fy*Ast*(d-0.416*Xu)
=10.84*10^6
Vu=11625N ,L_d=47*8=376mm
Assume that support width Ls=230mm
And side cover of 20 mm
Providing no hooks , Lo=Ls/2 - X'=95mm
1.3Mu/Vu + Lo= 1262+45=1307 >L_d

Check for shear and development length in long span-

SF at long edges=Wu x l_x x (r/2+r)=15500kN

Nominal shear stress at long edges=15500/(1000 x 142)=0.109N/mm^2

At long edges the dia of bars should be so restricted that following condition is satisfied

1.3Mu/Vu +Lo>Ld

Ast at supports of short span=1000*50.3/240.5 =209.2mm^2

Xu=0.87Fy x Ast/(0.36 x Fck x b)=5.25mm

Mu=0.87Fy x Ast x (d-0.416 x Xu)

=10.54 x 10^6

Vu=11500N ,L_d=47 x 8=376mm

Assume that support width Ls=230mm

And side cover of 20 mm

Providing no hooks , Lo=Ls/2 - X'=95mm

1.3Mu/Vu + Lo= 779.6+45=824.6 >L_d

Hence codal requirements are satisfied

Torsional reinforcement at corner-

Size =Lx/5=0.8m from center of support

Or 1.03+0.08=1.1 from the edge

Slab area of torsional reinforcement =3/4 $A_{s\,t}$

=3/4 *245

Provide 8 mm bar

Spacing =1000x50.3/187.5= 273.11mm

VBA results for Two way slab design-

1	Slab Design			Fixed Parameters		
2						
3	Design of two way slab			Column1	Column	Column
4	Long span of slab	4	m			
5	Short span of slab	4	m	Characterstic strenght of concrete	40	N/mm^2
6	Effective Cover	20	mm	Grade of steel	415	N/mm^2
7	Characterstic strenght of concrete	40	N/mm2	Live load	4	N/mm^2
8	Grade of steel	415	N/mm2	Floor finish	1	kN
9	Live load	4	kN/mm2	Effective Cover	20	mm
10	Width of support	230	mm			
11	Diameter of main bar	8	mm			
12	Diameter of distrbution bar	8	mm			
13	Floor finish	1	kN			
14	Type of support	Simply Supported		Input Parameters		
15						
16				Column1	Column	Column
17		0.8282				
18				Width of support	230	mm
19	Factored load	12.7575	kN	Diameter of main bar	10	mm
20				Diameter of distrbution bar	10	mm
21				Type of support	Simply Supported	
22	Effective depth	141		Long span of slab	4	m

14	Type of support	Simply Supported		Input Parameters		
15						
16				Column1	Column	Column
17		0.8282				
18				Width of support	230	mm
19	Factored load	12.7575	kN	Diameter of main bar	10	mm
20				Diameter of distrbution bar	10	mm
21				Type of support	Simply Supported	
22	Effective depth	141		Long span of slab	4	m
23	Area of main steel	245.1893		Short span of slab	5	m
24	Area of distribution steel	245		alphax	0.053	
25	Spacing of main bar	204.9029				
26	Spacing of distribution bar	205.0612				
27	lxeff/5			Output Parameters		
28	3*astX/4	183.892				
29		0		Column1	Column	Column
30	alphax	0.056		Factored load	12.757	kN
31				Area of main steel	245.1893	mm^2
32				Area of distribution steel	245	mm^2
33				Spacing of main bar'	204.9029	mm
34				Spacing of distribution steel	205.0612	mm
35						

5.3 RCC Column-

RCC Column
Effectively held in position at both ends.
Effective length=0.85L

P_u=2796 kN M_{ux}=43.14kNm M_{uy}=2.744kNm
F_{ck}=40MPa F_y=415MPa

Assume cover of 40 mm
Let the dimensions of column be 400x400 mm
D=400 mm B=400 mm d'=40mm H=1.2 m(bottom column with maximum load)

$M_u = a(M_{ux}^2 + M_{uy}^2)^{0.5}$
=49.92kN-m

d'/D =0.1 Hence chart 44 will be used.

$P_u/f_{ck}BD$=0.524
$M_u/f_{ck}BD^2$ =0.031
P_t/f_{ck} =0.0255
P_t = 0.0255x40=1.02%
$A_{st} = p_t$ x BD/100
A_{st} = 1632mm^2

Let us provide 10 bars , uniformly distributed along the 4 sides, hence diameter of bars is given by
Diameter = (1632x4/10x3.14)
Diameter = 14.4mm
Hence provide 10 bars of 14 mm diameter. Actual area A_s = 2010

Uniaxial moment capacity of section-
From chart 44
P_t/f_{ck} = 0.031 $M_u/f_{ck}BD^2$ = 0.0323

M_{ux}=0.04323xf_{ck} x b x D^2
M_{ux1}=82.688 kN-m
M_{uy1}=82.688 kN-m

Computation of P_{uz} –

From chart 63, corresponding to p=1.2 we got P_{uz}/A_g=23 N/mm^2
P_{uz} = 3680kN
CHECK-
P_u/ P_{uz}=0.9 <1 OK
ά=2/3(1+5/2(.9))=2.166
$(M_u/ M_{ux1})^{2.16} + (M_u/ M_{uy1})^{2.16} < 1$
0.255<1 OK

Design of transverse reinforcement-
Diameter of lateral ties =8 mm

Pitch should be less than B or 16 x dia or 300 whichever is less.
Provide 8 mm diameter @ 192 mm.

Staad Results for Column number 90 –

```
============================================================================

         C O L U M N   N O.    90  D E S I G N   R E S U L T S

      M40                    Fe415 (Main)               Fe415 (Sec.)

LENGTH:  1200.0 mm   CROSS SECTION:  400.0 mm X  400.0 mm  COVER: 40.0 mm

** GUIDING LOAD CASE:    7 END JOINT:    24  SHORT COLUMN


REQD. STEEL AREA   :     1630.12 Sq.mm.
REQD. CONCRETE AREA:   158369.88 Sq.mm.
MAIN REINFORCEMENT : Provide  16 - 12 dia. (1.13%,   1809.56 Sq.mm.)
                     (Equally distributed)
TIE REINFORCEMENT  : Provide  8 mm dia. rectangular ties @ 190 mm c/c

SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)
----------------------------------------------------------
Puz :   3358.03   Muz1 :     83.58   Muy1 :     83.58

INTERACTION RATIO: 0.90 (as per Cl. 39.6, IS456:2000)
```

5.4 Footing Design-

1. Soil design –
   Axial factored load = 2706kN
   Self wt. of footing @10% = 270.6kN
   Total load on soil = P+P' = 2976kN
   Factored soil bearing capacity = 300kN/m^2
   Area of footing required = 9.92m^2

   Provide width (B) and length (L) in meter: L=B
   LxB = 9.92
   L^2 = 9.92
   L=B= 3.15m

2. Flexure Design
   Depth of footing,
   $\tan\alpha \;\; 0.9\sqrt{\frac{100.9}{fck}+1}$
   $q_o = \frac{2972*1000}{3150*3150} = 0.299\text{N/mm}^2$
   $\tan\alpha \;\; 0.9\sqrt{\frac{100.9}{40}+1} \geq 1.19$
   $\tan\alpha = 4/450 \geq 1.19$
   i.e 4 ≥ 535.8mm
   Provide 800 mm deep pedestal, 300 mm above and 500 mm below the ground level.

   Bending moment at section x-x,
   O_y = (3.15-0.4)/2 = 1.375

   Net upward soil pressure = 2706/9.92 = 272.78kN/m^2
   M_x = 272.78x1x1.375x0.4 = 150kN.m

   Bending moment at section y-y,
   M_y = 272.78x1x1.375x0.4 = 150kN.m

   $M_{u.max}$ = 1.5x150 = 225 kN.m
   $d_{reqd} = \sqrt{\frac{225\times1000}{0.138\times40}} = 201\text{mm}$

   Assume or provide 10 mm dia. bars @ clear cover of 45 mm.
   D = 201 + 45 + 10/2 = 251 mm = 250mm

Assume overall depth 30% more than the overall depth required to resist shear stress,
Overall depth = 1.3x250 = 325 mm
d = 325 – 45 - 5 = 275 mm

$Ast_{reqd} = M_{u.max}/0.87f_y.d = 2266.5\ mm^2$
No. of 10 mm dia. bars = $\frac{2266.5}{\pi/4\times100}$ = 29
$Ast_{provided}$ = 2277.6 = $2300mm^2$

Check for development length –
Anchorage available = 1375-50 = 1325mm
Hence, OK.

3. One way-shear
   Shear force at x-x,
   V_x = 272.78x(1.375 - 0.275)
   =300.058 kN

   Shear force at y-y,
   V_y = 272.78(1.375 - 0.275)
   =300.058 kN

   $\tau_v = \frac{300.058\times1000}{1000\times325} = 0.92$
   $100xA_s/ b.d = 0.707$
   At $A_s/ b.d$ M40 – $\tau_c = 0.6$
   $\tau_v < \tau_c$
   But in our case it's not the same, therefore depth required must be increased to eliminate one way shear.

4. Two shear
   Punching shear force = $P\text{-}W_o(a+d)(b+d)$
   Punching shear stress,
   $\tau_{vp} = \frac{P-Wo(a+d)(b+d)}{2(a+b+2d)d}$
   $= \frac{2709-272.7(0.4+0.325)(0.4+0.325)}{2(0.4+0.4+0.65)0.325}$
   $= 2.72N/mm^2$
   K = $(0.5 + \frac{b}{a})$, max. = 1
   = (1.5 , 1)
   = 1

$\tau_{cp} = 0.25\sqrt{40} = 1.58$

$\tau_{vp} \leq K.\tau_{cp}$

The case is same here too, thus we have to increase depth of the footing for change the dimensions of the footing.

Let us take, d=550mm

By taking trial values of d, we have found that d = 550 mm is most suitable in which both one way shear and two way shear is satisfied.

Primary reinforcement-

A_{st} =M/0.87f_y x 250

=2677mm^2

Provide 20mm dia. bars@120mmc/c

Secondary reinforcement –

=0.12x1000x550/100 = 660mm^2

Provide 10mm dia. bars @120mm c/c

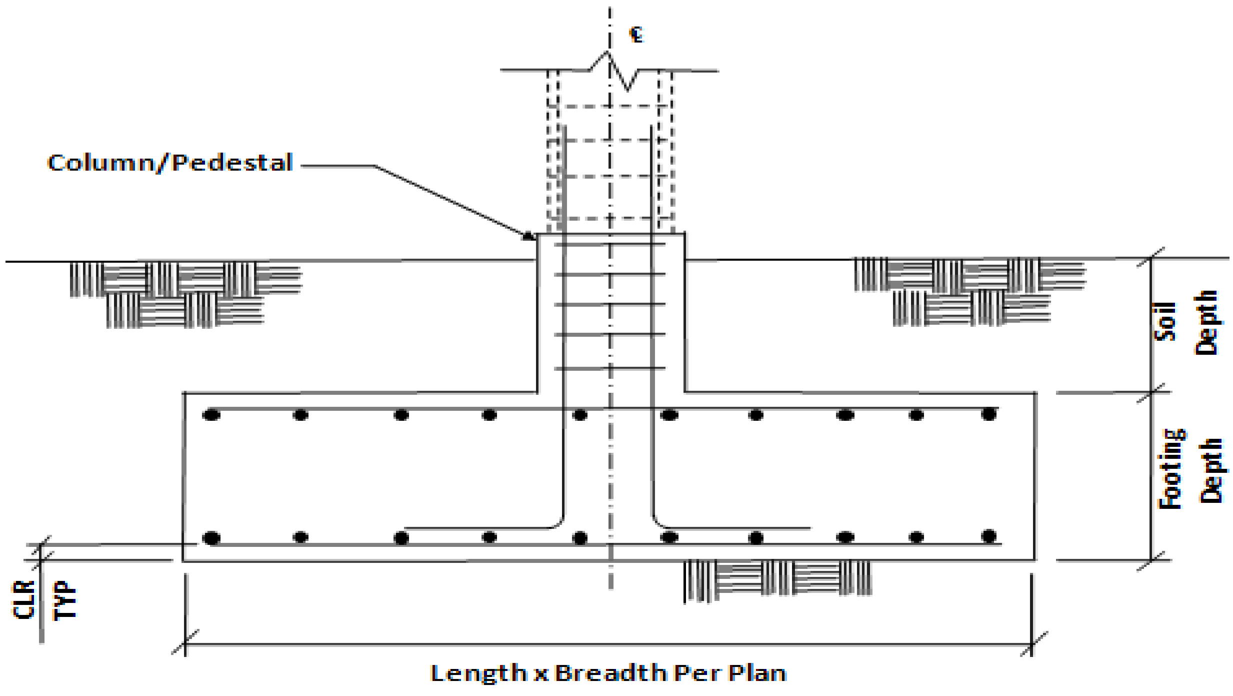
Check for deflection and cracking –

Deflection is not important and may not be checked.

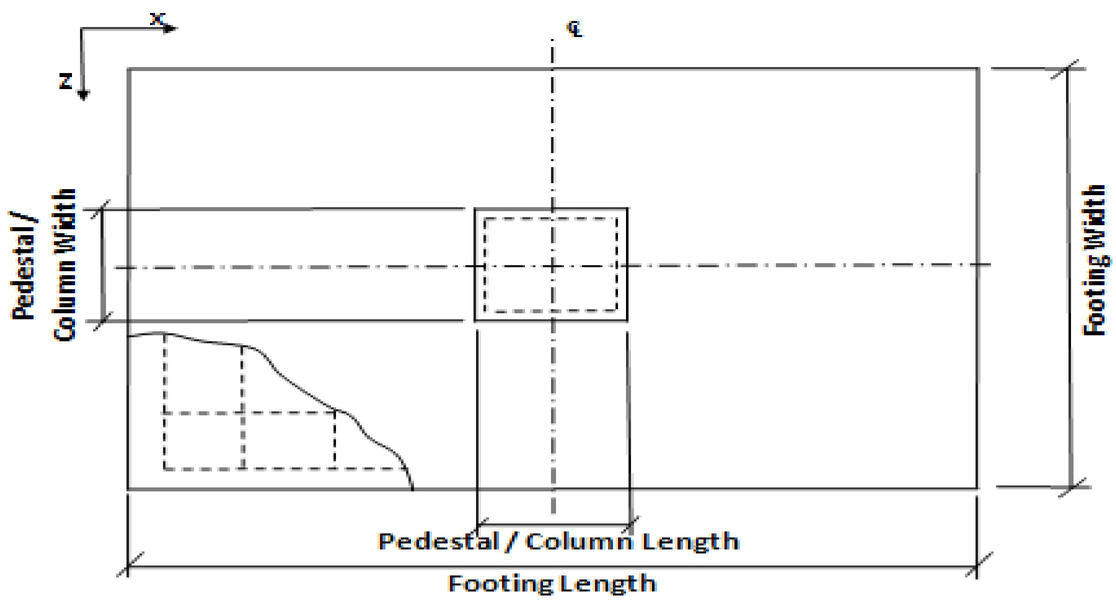
Spacing of bars –

Main – 120mm < 3d Hence, OK.

Secondary – 120mm < 5d Hence, OK.



ELEVATION



PLAN

Figure no. - 15

CHAPTER-7

Visual Basic for Applications - VBA

Definition of 'Visual Basic for Applications - VBA

Visual Basic is a computer programming language allows the creation of user-defined functions and the automation of specific computer processes and calculations. Users do not have to buy a copy of Visual Basic professional because Visual Basic for Applications is a standard feature of Microsoft Office products. Visual Basic for Applications (VBA) allows users additional customization beyond what is normally available in Microsoft Office products, such as Excel, Access, Word and PowerPoint.

How It Is Used-

In addition to Microsoft, numerous companies have used VBA to add the programming capabilities to their own applications. Visual Basic for Applications is also implemented in applications published by other companies including AutoCAD, ArcGIS, CATIA, CorelDraw and SolidWorks. VBA enables an application, such as Excel, to run a program internally and provide a customized version of Excel for a specific purpose, such as how long it takes to earn $1 million in an investment portfolio based on a specific interest rate and the number of years until retirement

VBA ADVANTAGES-

We can automate almost anything you do in Excel. To do so, we write instructions that Excel carries out. Automating a task by using VBA offers several advantages:

- Excel always executes the task in exactly the same way. (In most cases, consistency is a good thing.)
- Excel performs the task much faster than you can do it manually (unless, of course, you’re Clark Kent).
- Excel always performs the task without errors.
- If we set things up properly, someone who doesn’t know anything about Excel can perform the task by running the macro.

- We can do things in Excel that are otherwise impossible — which can make us a very popular person around the office.

BREAKING DOWN 'Visual Basic for Applications – VBA

- In VBA, users type commands into an editing module to create a macro. Macros allow users to automatically generate customized charts, reports and perform other data processing functions. Macros automate tasks and merge program functions that enable developers to build custom solutions using Visual Basic. Visual Basic for Applications requires code to run within a host application such as Excel because it cannot run as a standalone application.
- In the finance industry, VBA for Excel is frequently used to create and maintain complex financial spreadsheet models. Visual Basic for Applications for Excel is also used to create trading, pricing and risk management models, forecast sales and earnings and generate financial ratios.
- It is user intuitive, which allows users with little or no computer programming knowledge to learn VBA. It is an event-driven programming language specifically designed to customize applications that contain the Visual Basic for Applications application-programming interface (API). It is used to control the functionality of Microsoft Excel and any other Microsoft Office application.
- It can be used to keep lists of customers' names or other data, create invoices and forms, develop charts, analyze scientific data and for budgeting and forecasting. It can be used to control several features of a host application such as Excel, including manipulating user interface features, such as toolbars and menus and working with dialog boxes or customized user forms.

FLOW CHART FOR DESIGN OF BEAM

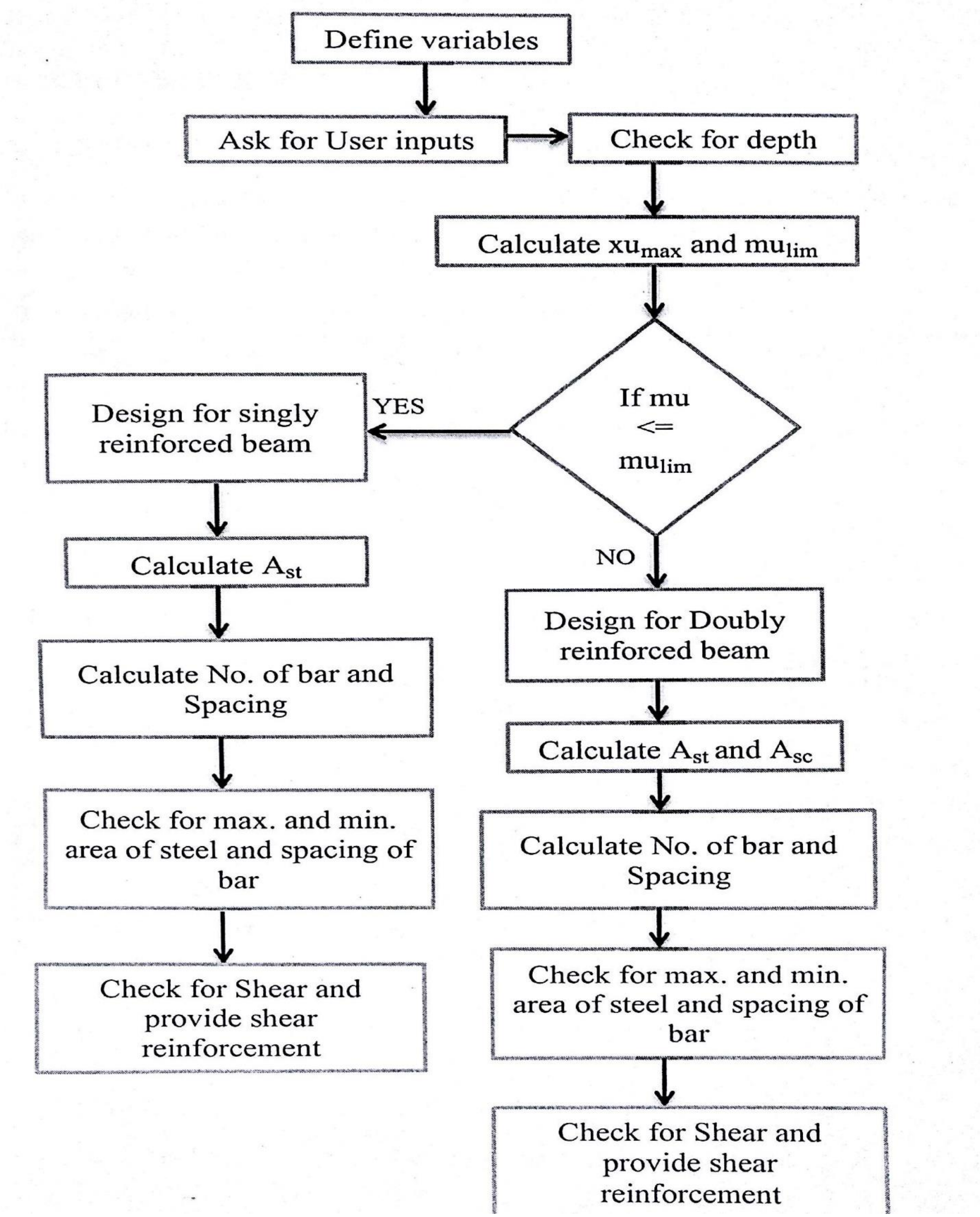


Figure no. - 16

Design Code for Beam -

```
Sub beams()

'Design of beam

Dim xumax As Single

Dim ast As Double

Dim asc As Single

Dim mulim As Single

Dim dia As Single

Dim spacing As Single

Dim number1 As Single

Dim number2 As Single

Dim Vu As Single

Dim astlim As Single

Dim ast2 As Single

Dim d1 As Single

Dim fsc As Single

Dim Tc As Single

Dim Tv As Single

Dim Tcmax As Single

Dim Asv As Single

Dim Sv As Single

Dim sheardia As Single

Dim diaincomp As Single

Dim legged As Single

Dim vol As Single

Dim length As Single
```

```
Dim numberbeam As Single

Dim designmix As Single

Dim nobc As Single


Range("B15").Font.Color = rgbRed

Range("B16").Font.Color = rgbRed

Range("B17").Font.Color = rgbRed

Range("B12").Font.Color = rgbRed

Range("B24").Font.Color = rgbRed

Range("B26").Font.Color = rgbRed

Range("B28").Font.Color = rgbRed

Range("B29").Font.Color = rgbRed

Range("B30").Font.Color = rgbRed

Range("B31").Font.Color = rgbRed

  b = InputBox("Enter the width of beam in mm")

  Range("b8").Value = b

  d = Range("B9").Value

  fy = Range("b5").Value


    If fy = 250 Then

    xumax = 0.531 * d

    End If

    If fy = 415 Then

    xumax = 0.479 * d

    End If

    If fy = 500 Then
```

```
xumax = 0.45 * d
End If

Range("K12").Value = xumax
fck = Range("B4").Value
b = Range("B8").Value

If d > 2 * b Or d < 1.5 * b Then
MsgBox ("it is advisable to have depth of beam  between 1.5 times to 2 times of width  ")
d = InputBox("enter new depth in mm", "do you want to change depth")
Range("b9").Value = d                     'return value

End If

mulim = 0.36 * fck * b * xumax * (d - 0.416 * xumax) * 10 ^ (-6)
  mu = Range("B7").Value
Range("B12").Value = mulim

Vu = Range("B34").Value
legged = Range("B38").Value
Tv = Vu * 10 ^ 3 / (b * d)

sheardia = Range("B36").Value
Asv = legged * 3.14 * sheardia ^ 2 / 4
```

```
If mu < mulim Then
MsgBox "singly reinforced section"
Ast = ((0.5 * fck / fy) * (1 - (1 - (4.6 * mu * 10 ^ 6 / (fck * b * d ^ 2))) ^ 0.5) * b * d)

Range("B15").Value = Ast
dia = InputBox("Enter the diameter of bar in tension in mm")
Range("b10").Value = dia

number1 = 4 * ast / (3.14 * dia ^ 2)
spacing = 1000 / number1
Asc= 0
Range("B28").Value = 0
Range("B26").Value = Asc
Range("B16").Value = spacing
Range("B17").Value = number1

 If Ast < (0.85 * b * d / fy) Then
 Ast = (0.85 * b * d / fy)
 MsgBox "minimum area of steel provided as per IS456:2000"
 End If
 If Ast > (0.04 * b * d) Then
 MsgBox "Ast exceeding is code limits"
 End If
 If (0.85 * b * d / fy) < ast < (0.04 * b * d) Then
 MsgBox "area of steel is within the permissible limits"
 End If
```

```
If spacing < dia Then
MsgBox "spacing provided is less than the given limits"
End If
If spacing > dia Then
MsgBox "spacing is within the limits"
End If
End If

 If mu = mulim Then
MsgBox "balanced section"
ast = ((1.5 * mu * 10 ^ 6) / 0.87 * fy * (d - 0.42 * xumax))
End If
 If mu > mulim Then
 MsgBox "doubly reinforced secton"
 MsgBox "increase the depth if u want singly reinforced section"
d1 = Range("B22").Value
fsc = Range("B24").Value
 dia = Range("B10").Value
Astlim = (mulim * 10 ^ 6) / (0.87 * fy * (d - 0.42 * xumax))
asc = ((mu - mulim) * 10 ^ 6 / ((fsc - 0.446 * fck) * (d - d1)))
ast2 = ((mu - mulim) * 10 ^ 6) / (0.87 * fy * (d - d1))
ast = astlim + ast2
Range("B28").Value = ast
Range("B26").Value = asc
number2 = (Range("B28").Value / (3.14 * dia ^ 2 / 4))
 b = Range("b8").Value
```

```
spacing = 1000 / number2
Range("B29").Value = spacing
diaincomp = InputBox("Enter the diameter of bar in compression in mm")
Range("B11").Value = diaincomp
nobc = (4 * asc) / (3.14 * diaincomp ^ 2)
Range("B31").Value = nobc
Range("B15").Value = 0
Range("B16").Value = 0
Range("B17").Value = 0
 If asc > 0.04 * b * d Then
MsgBox "ast is exceeding limits"
End If
If asc < 0.04 * b * d Then
MsgBox "area of steel is within permissible limits"
End If
End If
 If fck = 20 Then
Tcmax = 2.8
End If
If fck = 25 Then
Tcmax = 3.1
End If
If fck = 35 Then
Tcmax = 3.7
End If
If fck = 30 Then
```

```
 Tcmax = 3.5
End If
If fck = 40 Then
Tcmax = 4
End If


If Tcmax < Tv Then
MsgBox "redesign the beam due to shear failure"
sv = 0
End If
End Sub
```

FLOW CHART FOR DESIGN OF SLABS

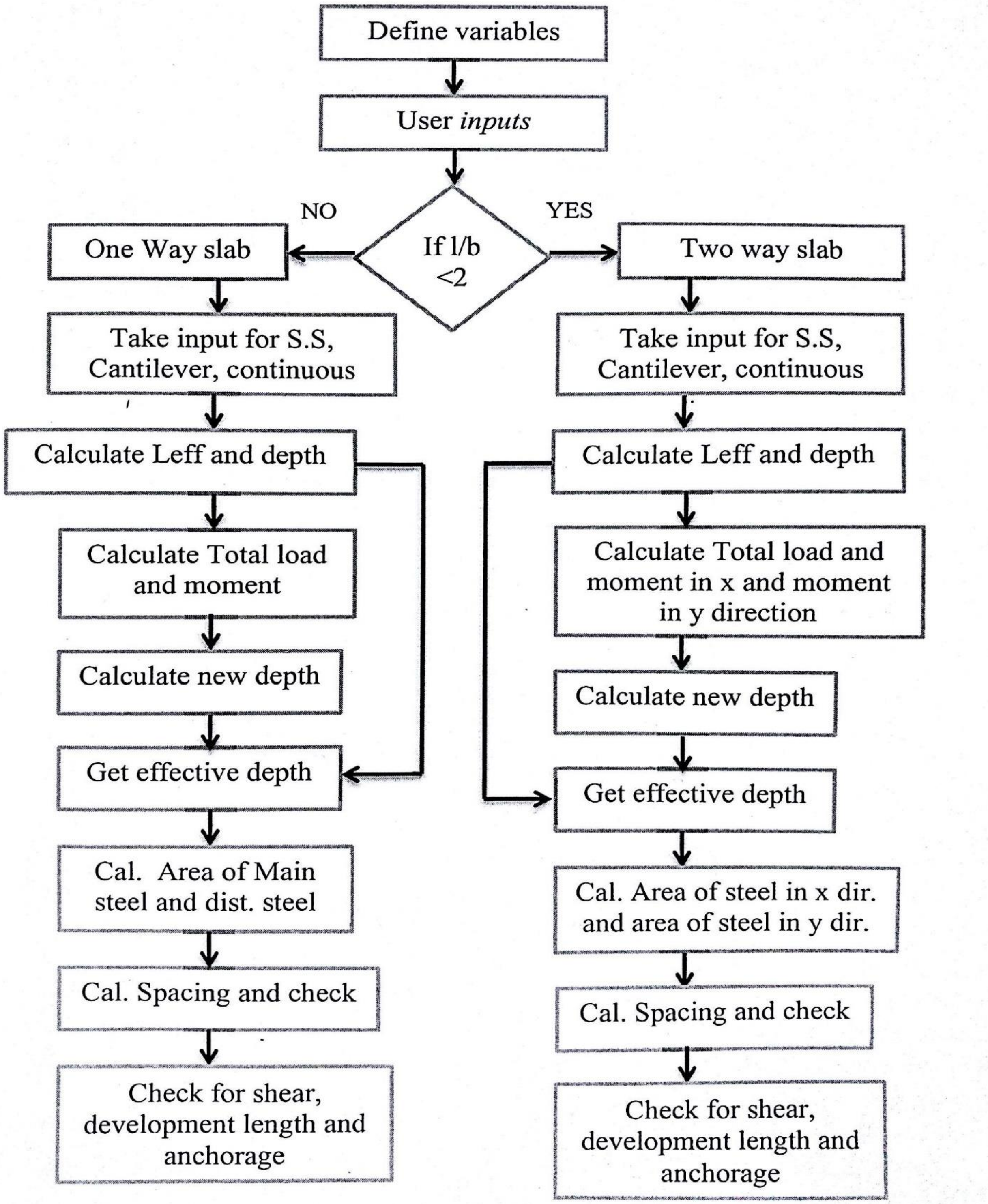


Figure no. – 18

Design Code for one way slab-

```
Sub slabs()
'design of slab
Dim lx As Single
Dim ly As Double
Dim LL As Single
Dim DL As Single
Dim FF As Single
Dim fy As Single
Dim w As Single
Dim diaM As Single
Dim diaD As Single
Dim d1 As Single
Dim d2 As Single
Dim leff As Single
Dim mu As Single
Dim vu As Single
Dim astm As Single
Dim astD As Single
Dim ec As Single
```

```
Dim ru As Single
Dim s As Single
Dim sd As Single
Dim vud As Single
Dim beta As Single
Dim tc As Single
Dim d As Single
Dim k As Single
Dim tao As Single
Dim ld As Double
Dim tbd As Single

ly = InputBox("Enter the lenght of long span   in m")
Range("B4").Value = ly
 lx = InputBox("Enter the lenght of short span   in m")
Range("B5").Value = lx
Range("B21").Font.Color = rgbRed
Range("B24").Font.Color = rgbRed
Range("B25").Font.Color = rgbRed
Range("B26").Font.Color = rgbRed
Range("B21").Interior.Color = rgbWhite
Range("B19").Font.Color = rgbRed
Range("B22").Font.Color = rgbRed
If (ly / lx) >= 2 Then
MsgBox"design for one way slab"
If Range("B14").Value = "Simply Supported" Then
w = Range("B10").Value * (10 ^ (-3))
```

```
leff = lx + w
d1 = leff / (20 * 1.5)
ec = Range("B6").Value * (10 ^ (-3))
Range("B17").Value = leff
LL = Range("B9").Value
FF = Range("B13").Value
FL = 1.5 * (LL + (d1 + ec) * 25 + FF)
Range("B19").Value = FL
If lx + d1 < lx + w Then
leff = lx + d1
End If
mu = (FL * (leff) ^ 2) / 8
Range("B20").Value = mu
Range("B20").Font.Color = rgbRed
vu = (FL * leff) / 2
Range("B21").Value = vu
Range("B21").Font.Color = rgbRed
fck = Range("b7").Value
fy = Range("b8").Value

ru = 0.36 * fck * 0.48 * (1 - (0.416 * 0.48))
d2 = ((mu * 10 ^ 6 / (ru * 1000)) ^ 0.5) * 10 ^ (-3)
If d1 > d2 Then
Range("b22").Value = d1 * 1000
Else
Range("b22").Value = d2 * 1000
End If
```

```
astm = Int((0.5 * fck / fy) * (1 - (1 - (4.6 * mu * 10 ^ 6 / (fck * 1000 * (d1 * 1000) ^ 2))) ^ 0.5) *
1000 * d1 * 1000)

Range("B23").Value = astm

Range("B23").Font.Color = rgbRed

astD = 0.12 * 1000 * ((Range("b22").Value) + (ec * 10 ^ (3))) / 100

Range("B24").Value = astD

Range("B24").Font.Color = rgbRed
```

```
diaM = InputBox("Enter the dia of main bar    in mm")

 Range("B11").Value = diaM

s = 1000 * 3.14 * (diaM ^ 2) / (4 * astm)

Range("B25").Value = s

diaD = InputBox("Enter the dia of distribution bar  in mm")

 Range("B12").Value = diaD

sd = 1000 * 3.14 * (diaD ^ 2) / (4 * astD)

Range("B26").Value = sd

vud = vu - FL * (Range("b22").Value + (Range("b22").Value / 2))

tao = vud / (1000 * Range("b22").Value)

beta = (0.8 * fck * 1000 * Range("b22").Value) / (689 * astm)

tc = ((0.85 * ((0.8 * fck) ^ 0.5)) * (((1 + 5 * beta) ^ 0.5) - 1)) / (6 * beta)

d = Range("b22").Value + 15

'k = 1 + ((0.005 * (300 - d)) / 25)

If d <= 150 Then

k = 1.3

End If

If d > 300 Then

k = 1
```

```
End If
If tao <= (k * tc) Then
MsgBox ("section is safe in shear")
Else
MsgBox("section is unsafe in shear")
End If
tbd = Range("K17").Value
ld = ((0.87 * fy * diaM) / (4 * tbd))
If (w * (10 ^ -3)) > ((ld / 3) + 25) Then
MsgBox ("encorage condition is satisfied")
End If
End If

'cantilever beam
If Range("b14").Value = "Cantilever" Then
w = Range("b10").Value * (10 ^ (-3))
leff = lx + (w / 2)
d1 = leff / (7 * 1.68)
ec = Range("B6").Value * (10 ^ (-3))
Range("B17").Value = leff
LL = Range("B9").Value
FF = Range("B13").Value
FL = 1.5 * (LL + (d1 + ec) * 25 + FF)
Range("B19").Value = FL
If lx + (d1 / 2) < lx + (w / 2) Then
leff = lx + d1 / 2
End If
```

m_u = (FL * (l_{eff}) ^ 2) / 2

Range("B20").Value = m_u

vu = (FL * l_{eff})

Range("b21").Value = vu

f_{ck} = Range("b7").Value

f_y = Range("b8").Value

ru = 0.36 * f_{ck} * 0.48 * (1 - (0.416 * 0.48))

d2 = ((m_u * 10 ^ 6 / (ru * 1000)) ^ 0.5) * 10 ^ (-3)

If d1 > d2 Then

Range("b22").Value = d1 * 1000

Else

Range("b22").Value = d2 * 1000

End If

a_{stm} = Int((0.5 * f_{ck} / f_y) * (1 - (1 - (4.6 * m_u * 10 ^ 6 / (f_{ck} * f_{ck} * 1000 * (DL * 1000) ^ 2))) ^ 0.5) * 1000 * d1 * 1000)

Range("b23").Value = a_{stm}

a_{stD} = 0.12 * 1000 * ((Range("b22").Value) + (ec * 10 ^ (3))) / 100

Range("b24").Value = a_{stD}

s = 1000 * 3.14 * (diaM ^ 2) / (4 * a_{stm})

Range("E25").Value = s

diaD = Range("B12").Value

s_d = (1000 * 3.14 * (diaD ^ 2)) / 4 * a_{stD}

Range("E26").Value = s_d

v_{ud} = vu - FL * (Range("b22").Value + (Range("b22").Value / 2))

t_{ao} = v_{ud} / (1000 * Range("b22").Value)

beta = (0.8 * f_{ck} * 1000 * Range("b22").Value) / (689 * a_{stm})

tc = ((0.85 * ((0.8 * f_{ck}) ^ 0.5)) * (((1 + 5 * beta) ^ 0.5) - 1)) / (6 * beta)

d = Range("b22").Value + 15

```
'k = 1 + ((0.005 * (300 - d)) / 25)
If d <= 150 Then
k = 1.3
End If
If d > 300 Then
k = 1

End If
If tao <= (k * tc) Then
MsgBox ("section is safe in shear")
Else
MsgBox ("section is unsafe in shear")
End If
tbd = Range("K17").Value
ld = ((0.087 * fy * diaM) / (4 * tbd))
If (w * (10 ^ -3)) > ((ld / 3) + 25) Then
MsgBox ("encorage condition is satisfied")
End If
End If
'continuous beam
If Range("b14").Value = "continuous" Then
w = Range("b10").Value * (10 ^ (-3))
leff = lx + w
d1 = leff / (26 * 1.5)
ec = Range("B6").Value * (10 ^ (-3))
Range("b17").Value = leff
LL = Range("B9").Value
```

```
FF = Range("B13").Value

Range("B19").Value = FL

If lx + d1 < lx + w Then

leff = lx + d1

End If

mu = 1.5 * (((((d1 + ec) * 25) + FF) * leff ^ 2) / 10) + (LL * (leff ^ 2) / 9)

Range("b20").Value = mu

fck = Range("b7").Value

fy = Range("b8").Value


ru = 0.36 * fck * 0.48 * (1 - (0.416 * 0.48))

d2 = ((mu * 10 ^ 6 / (ru * 1000)) ^ 0.5) * 10 ^ (-3)

If d1 > d2 Then

Range("b22").Value = d1 * 1000

Else

Range("b22").Value = d2 * 1000

End If

astm = Int((0.5 * fck / fy) * (1 - (1 - (4.6 * mu * 10 ^ 6 / (fck * 1000 * (d1 * 1000) ^ 2))) ^ 0.5) *
1000 * d1 * 1000)

Range("b23").Value = astm

astD = 0.12 * 1000 * ((Range("b22").Value) + (ec * 10 ^ (3))) / 100

Range("b24").Value = astD

s = 1000 * 3.14 * (diaM ^ 2) / (4 * astm)

Range("B25").Value = s

diaD = Range("B12").Value

sd = (1000 * 3.14 * (diaD ^ 2)) / 4 * astD


Range("B26").Value = sd
```

```
vud = vu - FL * (Range("b22").Value + (Range("b22").Value / 2))

tao = vud / (1000 * Range("b22").Value)

beta = (0.8 * fck * 1000 * Range("b22").Value) / (689 * astm)

tc = ((0.85 * ((0.8 * fck) ^ 0.5)) * (((1 + 5 * beta) ^ 0.5) - 1)) / (6 * beta)
```

```
d = Range("b22").Value + 15

'k = 1 + ((0.005 * (300 - d)) / 25)

If d <= 150 Then

k = 1.3

End If

If d > 300 Then

k = 1

End If

If tao <= (k * tc) Then

MsgBox ("section is safe in shear")

Else

MsgBox ("section is unsafe in shear")

End If

tbd = Range("K17").Value

'ld = ((0.87 * fy * diaM) / (4 * tbd))


'If (w * (10 ^ -3)) > ((ld / 3) + 25) Then

MsgBox ("encorage condition is satisfied")

End If

End If

'End If
```

```
If (ly / lx) < 2 Then
MsgBox "design for two way slab"
End If
End Sub
```

VBA Design Code for Two Way Slab-

```
Sub slabs()
'Design of slab
Dim lx As Single
Dim ly As Double
Dim fck As Single
Dim fy As Single
Dim xumax As Single
Dim d1 As Single
Dim d2 As Single
Dim w As Single
Dim lxeff As Single
Dim ec As Single
Dim lyeff As Single
Dim r As Single
Dim mux As Single
Dim muy As Single
```

Dim alphax As Single

Dim alphay As Single

Dim a_{stX} As Single

Dim a_{stY} As Single

Dim dia_X As Single

Dim dia_Y As Single

Dim s_x As Single

Dim sy As Single

Dim v_{uy} As Single

Dim beta As Single

Dim t_c As Single

Dim d As Double

Dim v_{ux} As Single

Dim x_{ux} As Single

Dim x_{uy} As Single

Dim t_{aoy} As Single

Dim m_{ulx} As Single

Dim m_{uly} As Single

Dim l_d As Double

Dim t_{bd} As Double

Dim t_{aox} As Single

l_y = Range("B4").Value

l_x = Range("B5").Value

f_{ck} = Range("B7").Value

f_y = Range("B8").Value

```
b = Range("B10").Value

If ly / lx < 2 Then

MsgBox ("design of two way slab")

End If

If Range("B14").Value = "Simply Supported" Then

w = Range("B10").Value * (10 ^ (-3))

lxeff = lx + w

d1 = lxeff / (20 * 1.5)

ec = Range("B6").Value * (10 ^ (-3))

Range("B17").Value = lxeff

LL = Range("B9").Value

FF = Range("B13").Value

FL = 1.5 * (LL + (d1 + ec) * 25 + FF)

Range("B19").Value = FL

If lx + d1 < lx + w Then

lxeff = lx + d1

End If

lyeff = ly + d1

r = lyeff / lxeff

alphax = InputBox("Enter the value of alphax(bending moment coefficient on the basics of end condition ")

 Range("B30").Value = alphax

mux = alphax * (FL * (lxeff) ^ 2)

muy = alphax * (FL * (lyeff) ^ 2)

Range("K7").Value = mux

fck = Range("B7").Value

fy = Range("B8").Value
```

```
ru = 0.36 * fck * 0.48 * (1 - (0.416 * 0.48))

If mux > muy Then

d1 = ((mux * 10 ^ 6 / (ru * 1000)) ^ 0.5) * 10 ^ (-3)

Else

d2 = ((muy * 10 ^ 6 / (ru * 1000)) ^ 0.5) * 10 ^ (-3)

End If




If d1 > d2 Then

Range("B22").Value = d1 * 1000

Else

Range("B22").Value = d2 * 1000

End If
astX = ((0.5 * fck / fy) * (1 - (1 - (4.6 * mux * 10 ^ 6 / (fck * 1000 * (d1 * 1000) ^ 2))) ^ 0.5) * 1000 * d1 * 1000)

Range("B23").Value = astX

Range("B23").Font.Color = rgbRed

astY = Int((0.5 * fck / fy) * (1 - (1 - (4.6 * muy * 10 ^ 6 / (fck * 1000 * (d1 * 1000) ^ 2))) ^ 0.5) * 1000 * d1 * 1000)

Range("B24").Value = astY

Range("B24").Font.Color = rgbRed

diaX = InputBox("Enter the dia of main bar in mm")

 Range("B11").Value = diaX

sx = 1000 * 3.14 * (diaX ^ 2) / (4 * astX)

Range("B25").Value = sx

Range("B25").Font.Color = rgbRed

diaY = InputBox("Enter the dia of distribution bar  in mm")
```

```
Range("B12").Value = diaY
sy = 1000 * 3.14 * (diaY ^ 2) / (4 * astY)
Range("B26").Value = sy
Range("B26").Font.Color = rgbRed
vuy = FL * lyeff * (r / (2 + r))
taoy = vuy / (1000 * Range("b22").Value)
beta = (0.8 * fck * 1000 * Range("b22").Value) / (689 * astX)
tc = ((0.85 * ((0.8 * fck) ^ 0.5)) * (((1 + 5 * beta) ^ 0.5) - 1)) / (6 * beta)
d = Range("b22").Value + ec
'k = 1 + ((0.005 * (300 - d)) / 25)
If d <= 150 Then
k = 1.3
End If
If d > 300 Then
k = 1
End If
If taoy <= (k * tc) Then
MsgBox ("short span  is safe in shear")
Else
MsgBox ("short span is unsafe in shear")
End If
vux = FL * lxeff * (r / (2 + r))
taox = vux / (1000 * Range("B22").Value)

If taox <= (k * tc) Then
MsgBox ("long span is safe in shear")
```

```
Else
MsgBox ("long span is unsafe in shear")
End If
xux = (0.87 * fy * astX) / (0.36 * fck * b)
xuy = (0.87 * fy * astY) / (0.36 * fck * b)
mulx = 0.87 * fy * astX * (Range("B22").Value - (0.416 * xux))
muly = 0.87 * fy * astY * (Range("B22").Value - (0.416 * xuy))
tbd = Range("K17").Value
ld = ((0.87 * fy * diaM) / (4 * tbd))
If (1.3 * ((mulx / vuy) + ((w / 2) - ec))) > ld Then
MsgBox ("code requirement for development lenght is satisfied for short span")
End If
If (1.3 * ((muly / vux) + ((w / 2) - ec))) > ld Then
MsgBox ("code requirement for development lenght is satisfied for long span")
End If
Range("B17").Value = lxeff / 5
Range("B28").Value = (3 * astX) / 4
Range("B29").Value = (1000 * 3.14 * (daiY ^ 2)) / (3 * astX)
End If
End Sub
```

VBA Design Code for Column-

```
Sub colunn()
'design of column
Dim lex As Single
Dim ley As Double
Dim fy As Single
Dim emin As Single
Dim exmin As Single
Dim eymin As Single
Dim Ast As Single
Dim sl As Single
Dim lf As Single
Dim pu As Single
Dim l As Single
Dim d As Single
Dim leff As Single
Dim Ag As Single
Dim B As Single
Dim Astlongitudinal As Single
Dim nob As Single
Dim dob As Single
Dim lx As Single
Dim ly As Single
Dim astx As Single
```

```
Dim asty As Single
Dim y1 As Single
Dim y2 As Double
Dim ec As Single
Dim e1 As Single
Dim e2 As Single
Dim fs1 As Single
Dim fs2 As Single
Dim fc1 As Single
Dim fc2 As Single
Dim puc As Single
Dim pusi As Single
Dim muc As Single
Dim musi As Single
Dim mu1 As Single

Range("B11").Font.Color = rgbRed
Range("B13").Font.Color = rgbRed
Range("B18").Font.Color = rgbRed
Range("B19").Font.Color = rgbRed
Range("B20").Font.Color = rgbRed
Range("B22").Font.Color = rgbRed
Range("B25").Font.Color = rgbRed

leff = InputBox("Enter the effective lenght according to end conditions   in mm")
Range("B2").Value = leff
d = InputBox("Enter the depth of section wrt major axis in mm")
Range("B7").Value = d
B = InputBox("Enter the width of section wrt to minor axis in mm")
Range("B10").Value = B
If (leff / d) < 12 Then
MsgBox "short column"
```

```
sl = Range("B3").Value
lf = Range("B4").Value
pu = sl * lf
Range("K5").Value = pu
emin = (leff / 500) + (d / 30)
Range("K6").Value = emin


If emin > 20 Then
Range("B6").Value = emin
Else
Range("B6").Value = 20
End If
d = Range("B7").Value
fy = Range("B9").Value
fck = Range("B8").Value

B = Range("B10").Value
Ag = B * d
Astlongitudinal = 0.008 * Ag
Range("K11").Value = Astlongitudinal

If emin < 0.05 * d Then
Ast = ((pu - 0.4 * fck * Ag) / (0.67 * fy - 0.4 * fck))
If Ast > Astlongitudinal Then
Range("B11").Value = Ast
Else
Range("B11").Value = Astlongitudinal
End If
dob = InputBox("Enter the diameter of main bar in mm")
Range("B12").Value = dob
nob = Int((4 * Range("B11").Value) / (3.14 * (dob ^ 2)))
```

```
Range("B13").Value = nob
sot = 300
Range("B15").Value = 300
MsgBox "According to IS456:2000 provide dia of 6mm for ties and spacing @300mm"
```

'Next case

ElseIf e_{min} > 0.05 * d Then

e_{xmin} = (leff / 500) + (d / 30)

If e_{xmin} > 20 Then

Range("B18").Value = e_{xmin}

Else

ex_{min} = 20

Range("B18").Value = 20

End If

End If

e_{ymin} = (leff / 500) + (B / 30)

If e_{ymin} > 20 Then

e_{ymin} = Range("B18").Value

Else

e_{ymin} = 20

Range("B19").Value = 20

End If

End If

m_u = p_u * e_{xmin} * 10 ^ (-3)

Range("B19").Value = m_u

'Ask user no of bars and dia of bars from user

```
astx = (Range("B20").Value * 3.14 * (Range("B21").Value ^ 2)) / 4
Range("B22").Value = astx

ec = Range("B24").Value

y1 = -(d / 2 - ec)
y2 = (d / 2 + ec)
'trial to find neutral axis
ku = Range("B23").Value
xu = ku * d
e1 = (0.0035 * ec) / d
e2 = (0.0035 * (d - ec)) / d
es = 2 * 10 ^ 5
fs1 = e1 * es
fs2 = e2 * es
fc1 = 446 * e1 * fck * (1 - 250 * e1)
fc2 = 0.446 * fck
puc = 0.36 * fck * xu * B * 10 ^ (-3)
Range("K26").Value = puc
pusi = ((fs1 - fc1) + (fs2 - fc2)) * astx * 10 ^ (-3)
Range("F25").Value = pusi
pu1 = puc + pusi
Range("B25").Value = pu1
If 0.5 * pu < pu1 < 1.3 * pu Then
muc = puc * (0.5 * d - 0.416 * xu)
musi = ((fs1 - fc1) * y1 + (fs2 - fc2) * y2) * astx * 10 ^ (-6)
mu1 = muc + musi
If mu1 > mu Then
MsgBox "design is safe"
Else
MsgBox "redesign the members"
```

```
End If
End If
End Sub
```

CHAPTER-7

DUCTILE DETAILING-

The requirements for designing and detailing of Monolithic reinforced concrete building so as to give them sufficient toughness and ductility to resist severe earthquake shock without collapse.

In general ductility of a structure or its members is the capacity to undergo large elastic deformations beyond the initial deformations without significant loss of strength and stiffness.

A ductile material is the one that can undergo large strange while resisting loads according to bloom structure must have both strength as well as ductility for satisfactory performance during an earthquake the main structural elements and their connections should be designed to have a ductile failure. This will enable the structure to avoid sudden collapse. Since RC is relatively less ductile in compression and shear, dissipation of flexural energy is best achieved by flexural yielding.

Significance of Ductility in Seismic Design-

- In general ductility denotes does properties of materials which have large energy absorption capacities in the inelastic range .The capability of a structure absorb energy within acceptable deformations and without failure is a very desirable characteristics of any earthquake resistance design. When a ductile structure is subjected to overloading it will tend to deform inelastically and in doing so will distribute the excess load to elastic parts of the structure.
- Open structure is ductile it can be expected to adapt to unexpected overloads, load reversals, impact and structural moments due to foundation settlement and volume changes. Secondly if the structure is ductile its occupants will have sufficient warning of the impending failure thus reducing the probability of loss of life in the event of collapse.

Ductile Design of Beam as per IS 13920-1993-

1. General

   Relevant data are given as follows-

   Grade of concrete = M40

   Grade of Steel = Fe415

   Live load on roof = $1.5kN/m^2$

   Live load of floors = $4kN/m^2$

   Floor finish = $1kN/m^2$

   Brick wall on peripheral beams = 230mm thick

   Density of concrete = $25kN/m^3$

   Density of brick wall including plaster = $20kN/m^3$

2. Design of middle floor beam. (Critical member beam number 551)

- Factored axial stress from staad file = $0.015N/mm^2$

  According to clause 6.1.1; IS 13920:1993

  $$0.015 < 0.1fck$$
  $$< 0.1x40$$
  $$< 4N/mm^2$$

  Hence, design as flexural member.

- Check for member size,

  Width of beam, B =250mm > 200 mm,

  Hence, OK. (Clause 6.1.3; IS 13920:1993)

  Depth of beam, D = 400mm

  According to clause 6.1.2; IS 13920:1993

  B/D > 0.3

  250/400 = 0.625 > 0.3 Hence, OK.

  Span, L = 400mm,

  According to clause 6.1.4; IS 13920:1993

  L/D = 4000/400 = 10 > 4 Hence, OK.

- Check for limiting longitudinal reinforcement:

Effective depth for moderate environmental conditions with 20mm diameter bars in two layers,
= 400 – 20/2 – 20 = 370mm.

Minimum reinforcement that should be provided according to clause 6.2.1 (b) of IS – 13920:1993

$= 0.24 \frac{\sqrt{fck}}{fy} = 0.24 \frac{\sqrt{40}}{415} = 0.37\%$

$= \frac{0.37 \times 250 \times 370}{100} = 342.25\text{mm}^2$

Maximum reinforcement that should be provided according to clause 6.2.2 of IS 13920:1993

= 2.5%

$= \frac{2.5 \times 250 \times 370}{100} = 342.25\text{mm}^2$

3. Design of Flexure –

- Design for Hogging moment (-ve)

  $\frac{Mu}{bd2} = 2.97$ d'/d = 30/270 = 0.08

  A_{st} at top = 1.029%

  = 1.029x250x370/100 = 951.825mm^2

  >Minimum reinforcement (342.5mm^2)

  <Maximum reinforcement (2312.5mm^2)

- Design for sagging moment (+ve)

  M_u = 62.8kN.m

  Beam is designed as singly reinforced beam,

  A_{st} at bottom $= 0.5 \frac{fck}{fy} \left(1-\sqrt{1-\frac{4.6Mu}{fck \times b \times d^2}}\right)bd$

  = 519.306mm^2

  >Minimum reinforcement (342.5mm^2)

  <Maximum reinforcement (2312.5mm^2)

- Details of requirements-

  Using 16 mm diameter bars,

  Spacing $= \frac{201.6 \times 1000}{951.825}$ = 210mm c/c

  Provide 5-16 dia. bars provided throughout the length of the beam at both top and bottom.

4. Design for Shear –
   Tensile Steel provided = 1.129%
   Permissible design shear stress of concrete
   τ_c = 0.68 (From IS – 456:2000 Table-9)
   Design shear strength of concrete
   = τ_c.bd
   = 0.68x250x370/100
   = 62.9kN

- Shear force due to plastic hinge formation at the ends of the Beam,
  $V_{\text{sway to right}} = \pm 1.4(M_u^{As}+M_u^{Bh})/L$
  $V_{\text{sway to left}} = \pm 1.4(M_u^{Ah}+M_u^{Bs})/L$

  Sagging and hogging moments are calculated on the basis of actual area,
  At top 1.12% which gives = 1.12x250x370/100 = 1036mm^2
  At bottom = 550mm^2

  From SP-16 we get the value of M_u^{Ah}/bd^2 at p^+ = 1.12 and d'/d = 0.08
  Therefore M_u^{Ah}/bd^2 = 3.3
  Hogging Capacity = $3.3 \times 250 \times 370^2/10^6$ = 112.9kN.m
  Sagging Capacity = $0.87 f_y.A_{st}.d\ (1- \frac{Ast \times fy}{b \times d \times fck})$
  = 0.87x415x5
  =6902kN.m

  $V_{\text{sway to right}} = \pm 1.4(112.9 + 69.2)/4$
  = 63.735kN

- Design Shear
  V_v = 1.2(DL+LL)/2 – 63.735
  = 1.2(126.7+64)/2 – 63.735
  50.685kN

  As per clause 6.3.3 of IS 13920:1993 the design shear force to be resisted shall be taken max. of:
  1) Factored Shear forces as per analysis (91.399kN from staad file)
  2) Shear forces due to plastic hinges

  The required capacity of shear reinforcement is given by

$V_{us} = V_u - V_c$

$= 91.399 - 62.9 = 28.499$kN

From table 62, SP-16

2 legged 6 mm diameter stirrups spacing 210 mm approximately.

As per IS-456:2000 spacing of stirrups at mid span shall not exceed d/2 = 185mm.

Minimum shear reinforcement as per IS-456:2000

$S_v = A_{sv}.0.87f_y/0.4b$

$= 205$mm

Spacing of links over a length of 2d at either end of beam as per clause 6.3.5 of IS 13920:1993 shall be let of:

1) d/4 = 370/4 = 92.5mm
2) 8 times diameter of smallest bar = 8x16 = 128mm

It should not be less than 100mm.

Hence, provide 2 legged – 6 dia. stirrups at 128mm.

Ductile Design of Column as per IS 13920-1993

Column Dimensions - 400 x 400 mm (Interior Column 103 selected)

1. Design Checks-

- Check for axial stress –
  Lowest factored axial force among all the load combinations = 918.3KN
  Factored axial stress = $\frac{918.3\times1000}{400\times400}$
  = 5.74MPa > $0.1f_{ck}$
  >0.1x4
  >4MPa
  Hence, design as a column member.
  This comes under clause 7.1.1; IS 13920:1993

- Check for member size –
  Width of column, B = 400mm > 300mm.
  Hence OK.
  Comes under clause 7.1.2; IS-13920:1993.

  Depth of column, D=400mm.
  $\frac{B}{D} = \frac{400}{400}$ = 1 > 0.4 Hence, OK
  Comes under clause 7.1.3; IS-13920:1993
  L = 400mm. (Effective span calculation – Annex E of IS-456:2000)
  $\frac{L}{D} = \frac{(4000-400)\times0.85}{400}$ = 7.65 < 12.
  i.e short column. Hence OK.
  Clause 25.1.2 of IS-456:20000

  Minimum dimension of column = 400mm.
  ≥ 15 times the largest diameter of beam
  longitudinal reinforcement = 15x20 =300
  Hence OK.
  Clause 7.1.2 – IS – 13920

- Check for Limiting longitudinal reinforcement -
  Minimum reinforcement,
  = 0.8%.
  $= \frac{0.8\times400\times400}{100}$

= 1280mm^2

Clause 26.5.3.1 of IS 456:20000.

Maximum reinforcement -

= 4%

= $\frac{4\times400\times400}{100}$

=6400mm^2

Clause 26.5.3.1 of IS 456:2000

2. Design of Column

- Sample Calculations for Column -

Approximate Design-

Design for Earthquake in x-direction,

$P_{u\,=}$ 1953kN

M_u = 66.207

$\frac{Pu}{fck\ BD}$= 0.305 for $\frac{d'}{D} = \frac{40+\left(\frac{20}{2}\right)}{400}$ = 0.125

$\frac{Mu}{fck\ BD^2}$ = 0.026

from chart 44 and 45 of SP-16

$\frac{P}{fck}$ = 0.07

Design for Earthquake in y-direction

P_u = 2286kN

M_u = 45.182kN.m

$\frac{Pu}{fck\ BD}$ = 0.36,

$\frac{Mu}{fck\ BD^2}$ = 0.02

from chart 44 and 45 of SP-16

$\frac{Pt}{fck}$ = 0.065

Longitudinal Steel-

Higher of the above is taken, i.e

$\frac{Pt}{fck}$ = 0.07

required steel = 0.07x40

= 2.8%

= $\frac{2.8\times400\times400}{100}$

= 4480mm^2

Provide 10 – 20 dia. bars + 8 – 16 dia. bars

A_{st} provided = 4750mm^2

Checking Section -

Column should be checked for biaxial moment. Moment about other axis may occur due to torsion of building or due to minimum eccentricity of the axial load.

Checking for critical Combination -

Eccentricity = $\frac{4000-400}{500} + \frac{400}{30}$ = 20.53 > 20

Hence e =20.53

M_u = 1953x0.02053 = 40.10kN.

From chart 44 and 45 of SP-16

$\frac{Mu}{fck\ BD^2}$ = 0.1

$M_{u2, 1}$ = 0.1x40x400x400^2=256KN.m

$M_{u3, 1}$=$M_{u2, 1}$

$P_{u,2}$ =0.45fck .A_c +0.75fy.A_{sc} (Clause 39.6 of IS 456:2000)

=0.45fck .A_g + (0.75fy – 0.45fck).A_{sc}

=0.45x40x400x400 + (0.75x4.5-0.45x40)x4750

=4272.94 kN

$\frac{Pu}{Pu{,}2}$ = $\frac{1953}{4272.94}$ = 0.46

for calculation of α_n = 1.4 (Clause 39.6 IS- 456:2000)

$\left[\frac{Mu2}{Mu2{,}1}\right]^{\alpha n} + \left[\frac{Mu3}{Mu3{,}1}\right]^{\alpha n} = \left[\frac{66.207}{256}\right]^{2.4} + \left[\frac{40.1}{256}\right]^{1.4}$

= 0.1 < 1

Hence OK.

Design for Shear -

Shear Capacity of column,

A_{st}% = $\frac{4750\times100}{400\times400}$ = 2.96%

Assume 50% steel provided as tensile steel,

which implies, A_{st} =2.96/2 = 1.48%

Permissible shear stress, τ_c = 0.79MPa (Table 19-IS 456)

Considering lowest P_u = 918.3KN

Multiplying factor = δ = 1 + ($3P_u/A_g f_{ck}$)

= 1.0004 < 1.5 (Clause 40.2.2 of IS 456:2000)

τ_c = 0.79x1 = 0.79MPa

Effective depth in x-direction or y-direction,

= 400-40-20/2 = 350mm

$V_{cx} = \frac{0.79 \times 400 \times 350}{1000}$ = 110.6KN = V_{cy}

Shear force due to Plastic hinge for motion at the ends of the beam -
Calculating for maximum of the two directions (x and y)
which is y,
$V_u = \frac{1.4(112.9+69.2)}{4}$
= 62.9kN (From beam design already done)

Details of Transverse reinforcement -
Design of links in x-direction and y-direction,
V_s = 110.6 – 62.9 = 47.7kN
Spacing of 4 legged 8 Ø links,
$= \frac{4 \times 50.3 \times 0.87 \times 415 \times 350}{47.7 \times 1000}$ = 533mm

Nominal links -
Spacing of hoops shall not exceed half the least lateral dimension of the column i.e 400/2 = 200mm
Clause 7.3.3; IS 13920:1993
Provide 8 Ø links @ 200 c/c in mid height portion of the column.

Confining links –
Clause 7.4.8 of IS 13920:1993
A_y = ((0.18x5xhxf_{ck})/f_y)(A_8/A_k -1)
h = longer dimension of the rectangular link
=(400-40-20)/2
= 170mm.
A_g= 400x400 = 160000mm^2

A_k = (400 - 2*40 + 2*8) * (400 – 2*40 + 2*8)
= 112896mm^2

Assuming 8 Ø stirrup, A_{sh} = 50mm^2
$50 = \frac{0.18 \times S \times 170 \times 40}{415} \times (\frac{180000}{112896} - 1)$
S= 40.63
Link spacing for confining one shall not exceed,
a) 1/4 of minimum column dimension i.e,
= 1/4 x 400 = 100mm
b) Should not be less than 75mm not more than 100m

Clause 7.4.6 of IS 13902:1993.

Provide 8 Ø confining links 100c/c for a distance l_o which shall not be less than,

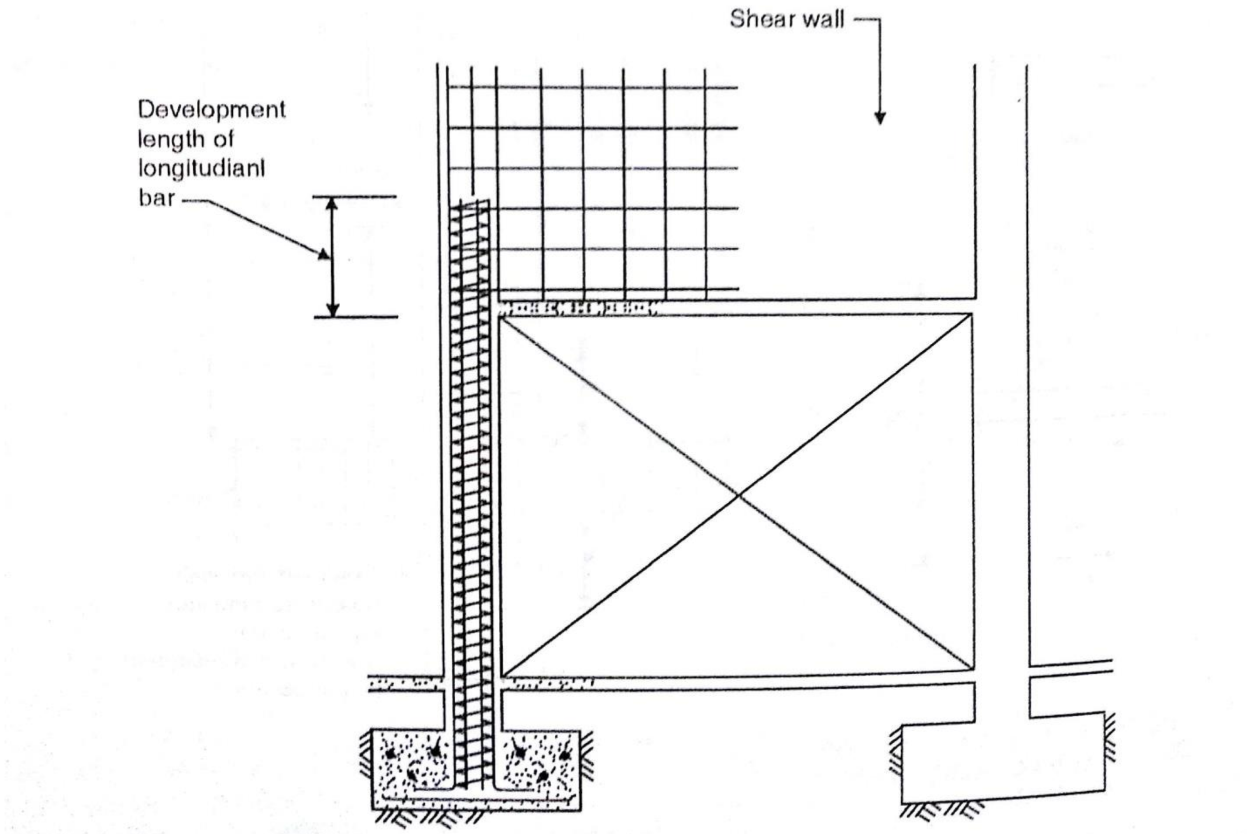
i) Larger, Lateral dimension = 400mm

ii) 1/6 clear span = (4000-400)/6 = 600mm

iii) 450mm

Clause 7.4.1. if IS 13902:1993

Provide confining reinforcement for l_o = 600mm on either side of the joint.



Confining reinforcement drawing

Figure no. - 18

CONCLUSION

In the coming years the work in the field of Earthquake Resistant Design is very important to have safe structures which can take the effect of earthquake with minimum damage. In our project after applying all load combinations it has been concluded that the effect of earthquake comes out to be more severe even though the maximum wind load is acting. (with a wind intensity of 39m/s).

Earthquake load is applied for different combinations and the worst case is taken for design. Members are analyzed and designed in the STAAD Pro. Also the design is done by two different ways in STAAD Pro, one is simple design method and the other is Interaction concrete design.

Manual computation of design of different critical members is done and results of the critical members by STAAD Pro are checked. The results shown by STAAD Pro and the manual computation of design are almost same.

A VBA Code is developed for design of different members of the structure like beam, one-way slab, two-way slab and column. User inputs are taken and desired calculations for the design of different members are performed by the MACROS. All the checks as per IS 456:2000 are performed. This work is user friendly at it provides the range of standard values of inputs to the user, and hence one can easily get results. This code helps to eliminate human error in design calculations and time consumption.

All the members are also designed for seismic forces and reinforcement for the same is provided as per IS 13920 -1993 (Ductile detailing of reinforced concrete structures subjected to seismic forces). Ductility in seismic design has a great importance. If a structure is ductile it can be expected to adapt unexpected overloads, load reversals and structural moments due to settlement of foundation and also if a structure is ductile it gives much time for its occupant to leave the structure with minimum probability of loss of life.

The results of STAAD Pro, VBA Code and manual computation are compared and found to be almost same. So finally we have formulated a code which omitted human errors and gives us results same as given by STAAD output file .

REFERENCES

- Design of Steel Structures by S.K. Duggal (Chapter 2.5 & 2.6)
- Document no. ::IITK-GSDMA-EQ26-V2.0 (Seismic load calculation)
- Document no. : IITK-GSDMA-EQ26-V3.0 (Explanatory example for ductile detailing of RC Building.)
- Indian Standard (IS).(2000). “Design Code For Rcc Structures”.
- Indian Standard (IS).(2007). “Structural Steel Design”.
- IS 1893 (Part 1) : 2002 – Criteria for Earthquake Resistant Design of Structures.
- IS:875(Part-1): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures Part-1 Dead Loads - Unit Weights of Building Materials and Stored Materials
- IS:875(Part-2): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures Part 2: Imposed Loads
- IS:875(Part-3): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures Part 3 : Wind Load.
- IS:456-2000: Design Code for RCC Structures,
- Limit State Design of Reinforced Concrete by B.C. Punmia. (Chapter 27.9)
- Wise owl – YouTube channel (VBA Coding tutorials)