ANALYSIS AND EARTHQUAKE RESISTANT DESIGN OF A MULTI-STOREYED BUILDING

Project Report submitted in partial fulfillment of the requirement for the degree of

Bachelor of Technology.

in

Civil Engineering

under the Supervision of

Mrs. Poonam Dhiman

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CERTIFICATE

This is to certify that the work entitled, "**ANALYSIS AND EARTHQUAKE RESISTANT DESIGN OF A MULTI-STOREYED BUILDING**" submitted by **SUSHANT GUPTA, ASHISH PAMNANI, AJAIN ANAND, ANIMESH TANEJA, WAQAR IMAM and ABHINAV BANSAL** in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under our supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree.

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Certified the above mentioned project work has been carried out by the said group of students.

ACKNOWLEDGEMENT

The success of any project depends largely on the encouragement and guidelines of many others. Therefore we take this opportunity to express our sincere gratitude to the people who have been instrumental in the successful completion of the project.

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The successful compilation of final year project depends on the knowledge and attitude inculcated in the total length of the course. So we want to express our sincere gratitude to all the faculties who taught us during the four years of B.Tech.

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LIST OF ABBREVIATIONS AND SYMBOLS

Symbol	Meaning
DL	Dead load
LL	Live Load
EL	Live Load
WL	Wind load
A_{st}	Area of tension reinforcement
f_y	Characteristic strength of steel
f_{ck}	Characteristic compressive strength of concrete
α_x, α_y	Bending moment coefficient for 2 way slab
e_x	Length of shorter side of slab
e_y	Length of longer side of slab
P_u	Axial load on compression member
d'	Clear cover
A_{sc}	Area of compression reinforcement or area of longitudinal Reinforcement
Q_{ult}	Axial load carrying capacity of the base
C	Cohesion
N_c	Bearing capacity factor
A_b	Cross-sectional area of base
A	Reduction factor
A_s	Surface area of pile shaft
E_x	Eccentricity
E	Young modulus
I	Moment of inertia
q_u	Ultimate bearing capacity of pile
R	Depth of fixity
N_q	Bearing capacity factor
Σ	Effective stress
q_b	Unit end bearing
f_s	Unit skin friction

ABSTRACT

The ground shaking during earthquakes can cause the collapse of structures. In order to save loss of lives and property; the structures need to be designed against the forces coming from ground shaking. In this project, an RCC framed4-storeyed building has been analyzed and designed to withstand the earthquakes which the Indian seismic zone-IV is prone to, the site of building being in Chandigarh. The tool used for computations is STAAD-pro. The superstructure is to be mounted on the pile foundation with silty soil below.

The analysis has been carried out for earthquake and wind forces as per Indian Standard codes IS-1893:2002 (part-1) and IS-875:1987 (part-3) respectively. Reinforced concrete design is done with limit state method conforming to IS-456:2000 inbuilt in the STAAD.pro. The design results are verified also against manual design randomly. Ultimately, the reinforcement detailing is done strictly as per IS-13920:1993 so as to provide ductility to structural members including joints. The reinforcement of various members are as drawings. The piles have been analyzed using method of fixity and designed for the absolute maximum reactions at column bases.

The building was analyzed in absence of brick-infill panels which might have caused underestimation of lateral stiffness of the building making the design safer. As far as scope of further work is concerned, the infill panels may be involved in analysis and more economical design can be obtained.

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CHAPTER 1
INTRODUCTION

"Earthquake don't kill people, buildings do."

1.1 *General*

The chapter deals with an introduction to the main attributes of the earthquake resistant design of structures with a special emphasis on related additional features in comparison to civil engineering design. Designing Earthquake Resistant Structures is indispensable. Every year, earthquakes take the lives of thousands of people, and destroy property worth billions. It is imperative that structures are designed to resist earthquake forces, in order to reduce the loss of life. Structural design plays an important role. Here, different tips and techniques used in designing Earthquake Resistant structures are discussed.

1.2 *What is an Earthquake?*

An earthquake is a sudden, rapid shaking of the Earth caused by the breaking and shifting of rock beneath the Earth's surface. For hundreds of millions of years, the forces of plate tectonics have shaped the Earth as the huge plates that form the Earth's surface move slowly over, under, and past each other. Sometimes the movement is gradual. At other times, the plates are locked together, unable to release the accumulating energy. When the accumulated energy grows strong enough, the plates break free causing the ground to shake. Most earthquakes occur at the boundaries where the plates meet; however, some earthquakes occur in the middle of plates.

Ground shaking from earthquakes can collapse buildings and bridges; disrupt gas, electric, and phone services; and sometimes trigger landslides, avalanches, flash floods, fires, and huge destructive ocean waves (tsunamis). Buildings with foundations resting on unconsolidated landfill and other unstable soil, and trailers and homes not tied to their foundations are at risk because they can be shaken off their mountings during an earthquake. When an earthquake occurs in a populated area, it may cause deaths and injuries and extensive property damage.

The dynamic response of building to earthquake ground motion is the most important cause of earthquake-induced damage to buildings. The damage that a building suffers primarily depends not upon its displacement, but upon acceleration. Whereas displacement is the actual distance the ground and building may move during an earthquake, acceleration is a measure of how quickly they change speed as they move. The conventional approach to earthquake resistant design of buildings depends upon providing the building with strength, stiffness and inelastic deformation capacity which are great to withstand a given level of earthquake-generated force. This is generally accomplished through the selection of an appropriate structural configuration and the carefully detailing of structural members, such as beams and columns, and the connections between them.

1.3 *How do Earthquakes affect 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 ground. The RC frame participates in resisting the earthquake forces. Earthquake shaking generates inertia forces in the building, which are proportional to the building mass. Since most of the building mass is present at floor levels, earthquake induced inertia forces primarily develop at the floor levels. These forces travel downwards - through slabs and beams to columns and walls, and then to foundations from where they are dispersed to ground. As inertia forces accumulate downwards from the top of the building, the columns and walls at lower storey experience higher earthquake- induced forces and are therefore designed to be stronger than those in storey above.

1.3.1 *Horizontal Earthquake Effects*

Under gravity loads, tension in the beams is at the bottom surface of the beam in the central location and is at the top surface at the ends, while during the earthquakes, significant forces act horizontally on the building members. The level of bending moment due to earthquake loading depends on severity of shaking and can exceed that due to gravity loading. Thus, under strong earthquake shaking, the beam ends can develop tension on either of the top and bottom faces. Since concrete cannot carry this tension, steel bars are required on both faces of beams to resist reversals of bending moment.

1.3.2 *Role of Floor Slabs and Masonry*

Floor slabs are horizontal plate like elements, which facilitate functional use of buildings. Usually, beams and slabs at one storey level are cast together. In residential multi-story buildings, thickness of slabs is only about 110-150 mm. When beams bend in the vertical direction during earthquakes, these thin slabs bend along with them and, when beams move with columns in the horizontal direction, the slab usually forces the beams to move together with it. In most buildings, the geometric distortion of slab is negligible in the horizontal plane; this behavior is known as the 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 demarcate a floor into functional spaces (rooms). Normally, these masonry walls, also called infill walls, are not connected to surrounding RC columns and beams. When columns receive horizontal forces at floor levels, they try to move in horizontal direction, but masonry walls tend to resist this movement. Due to their heavy weight and thickness, these walls attract rather large horizontal forces. However, since masonry is a brittle material, these walls develop cracks once their ability to carry horizontal load is exceeded. Thus masonry walls are enhanced by mortars of good strength, making proper masonry courses, and proper packing of gaps between RC frame and masonry infill walls.

1.4 *Protection from Earthquakes*

For a building to remain safe during earthquake shaking, columns should be stronger than beams, and foundations should be stronger than columns. If columns are made weaker, they suffer severe local damage, at the top and bottom of a particular storey.

1.4.1. *Earthquake Resistant Building Design Philosophy*

a) 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.

b) 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.

c) Under *strong* but rare shaking, the main members may sustain severe damage, but the building should not collapse.

There are various new techniques which help in reducing the impact of earthquake forces on buildings. Most of these techniques are expensive to implement. The concept of base isolation is explained through an example building resting on frictionless rollers. When the ground shakes, the rollers freely roll, but the building above does not move. Thus, no force is transferred to the building due to the shaking of the ground; simply, the building does not experience the earthquake. Now, if the same building is rested on the flexible pads that offer resistance against lateral movements, then some effect of the ground shaking will be transferred to the building above. If the flexible pads are properly chosen, the forces induced by ground shaking can be a few times smaller than that experienced by the building built directly on ground, namely a fixed base building. The flexible pads are called base-isolators, whereas the structures protected by means of these devices are called base-isolated buildings.

1.4.2 *Energy Dissipation Devices for Earthquake Resistance*

Another approach for controlling seismic damage in buildings and improving their seismic performance is by installing Seismic Dampers in place of structural elements, such as diagonal braces. These dampers act like the hydraulic shock absorbers in cars where, much of the sudden jerks are absorbed in the hydraulic fluids and only little is transmitted above to the chassis of the car. When seismic energy is transmitted through them, dampers absorb part of it, and thus damp the motion of the building.

1.4.3 *Active Control Devices for Earthquake Resistance*

a. Sensors to measure external excitation and/or structural response.

b. Computer hardware and software to compute control forces on the basis of observed excitation and/or structural response.

c. Actuators to provide the necessary control forces.

1.5 *Wind design theory*

Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term 'wind' denotes almost exclusively the horizontal wind, vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 to 30 metres above ground.

Very strong winds (greater than 80 km/h) are generally associated with cyclonic storms, thunderstorms, dust storms or vigorous monsoons. A feature of the cyclonic storms over the Indian area is that they rapidly weaken after crossing the coasts and move as depressions/lows inland. The influence of a severe storm after striking the coast in general does not exceed about 60 kilometres, though sometimes, it may extend even up to 120 kilometres. Very short duration hurricanes of very high wind speeds called Kal Baisaki or Norwesters occur fairly frequently during summer months over North East India.

The wind speeds recorded at any locality are extremely variable and in addition to steady wind at any time, there are effects of gusts which may last for a few seconds. These gusts cause increase in air pressure but their effect on stability of the building may not be so important; often, gusts affect only part of the building and the increased local pressures may be more than balanced by a momentary reduction in the pressure elsewhere. Because of the inertia of the building, short period gusts may not cause any appreciable increase in stress in main components of the building although the walls, roof sheeting and individual cladding units (glass panels) and their supporting members such as purlins, sheeting rails and glazing bars may be more seriously affected. Gusts can also be extremely important for design of structures with high slenderness ratios.

The liability of a building to high wind pressures depends not only upon the geographical location and proximity of other obstructions to air flow but also upon the characteristics of the structure itself. The effect of wind on the structure as a whole is determined by the combined action of external and internal pressures acting upon it. In all cases, the calculated wind loads act normal to the surface to which they apply. Buildings shall also be designed with due attention to the effects of wind on the comfort of people inside and outside the buildings.

1.5.1 *Wind speed and pressure*

Nature of Wind in Atmosphere — In general, wind speed in the atmospheric boundary layer increases with height from zero at ground level to a maximum at a height called the gradient height. There is usually a slight change in direction (Ekman effect) but this is ignored in the code. The variation with height depends primarily on the terrain conditions. However, the wind speed at any height never remains constant and it has been found convenient to resolve its instantaneous magnitude into an average or mean value and a fluctuating component around this average value. The average value depends on the averaging time employed in analysing the meteorological data and this averaging time varies from a few seconds to several minutes. The magnitude of fluctuating component of the wind speed which is called gust depends on the averaging time. In general, smaller the averaging interval, greater is the magnitude of the gust speed.

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

1.6 *Objective of the Project*

- Objective is to analyze and design of a 4-storeyed building lying in seismic zone-IV.
- The building will be divided into portal frames and these frames have to be analyzed using the STAAD.PRO 2004 software.
- The 4-storeyed portal frame will be analyzed for dead load, live load, wind load and earthquake load combinations.
- The analysis will give the forces arising in the members, namely – transverse beams and columns, due to the above loads and these members were designed for the several forces obtained due to the load combinations.
    1. Slabs
    2. Beams
    3. Columns

- The members will be designed by the Limit State method, according to the guidelines prescribed by IS: 456-2000.
- IS-Codes which are to be used are as follows:-
    - IS-875:1987 PART-1 for dead load
    - IS-875:1987 PART-2 for live load
    - IS-875:1987 PART-3 for wind load
    - IS-1893:2002 for earthquake loads
    - IS-456-2000 for limit state design
    - IS-13920-1993 for R/F Ductile detailing

1.6.1 *Stages of Designing*

The approach for designing the proposed building consisted of the following stages.

Estimation of Loads

For the four-storey building, the analysis was performed and the design done for the following loads:

- Dead load
- Live load
- Earthquake load
- Wind load

The dead load was worked out by assuming a certain thickness for the slab and then the actual thickness was accordingly provided after calculating the required value. The load due to the flooring – screed, finishes, tiles etc. was given due consideration and an allowance was made for future erection of partitions.

The live loads considered were due to the imposed loads in case of educational buildings, as per the specifications of The National Building Code.

Due to increased emphasis being laid on the design of earthquake resistant structures nowadays, the earthquake forces were estimated with the help of the provisions of the revised Seismic Code (IS:1893). The proposed building would lie in Zone IV. The value of the importance factor assigned to the entire structure was 1.

The designing was done after analyzing the structure for the above-mentioned loads – individually and for different load combinations recommended in the code.

Analysis of the Structure

The building was divided into portal frames and these frames have been analyzed using the STAAD.PRO 2004 software. The four-storey portal frame was analyzed for dead load, live load, and earthquake load combinations.

The analysis gave the forces arising in the members, namely – transverse beams and columns, due to the above loads and these members were designed for the severest of forces obtained due to the load combinations.

Member Design

The members were designed by the Limit State method, according to the guidelines prescribed by IS: 456-2000. For the purpose of design, Design Aids to IS: 456 i.e. SP-16 was also referred.

Drawing Details

The architectural drawings enabled the understanding of the layout of the building and gave the locations of the various members. The structural drawings were prepared after designing the individual members, showing the details of the reinforcement to be provided. The analysis, designs, and the drawings were compiled in the end in the form of this project report.

CHAPTER 2

PROBLEM FORMULATION

2.1 *General*

In this chapter, the buildings data have been described. The loads taken from various parts of IS-875 have been imposed on the building and ultimate loads coming on beams from slabs are calculated. Seismic coefficient method will be applied to calculate base-shear due to earthquakes, as per IS-1893:2002. Wind loads are also applied separately as per IS-875-part-III.

2.2 *Building Parameters*

No. of Storeys	=	4
Slab thickness	=	110 mm (all floors)
Clear cover: Slabs	=	20 mm
Beams	=	25 mm
Grade of concrete to be used	=	M25
Steel used	=	Fe 415 (IS-1786:1985)
Beams Dimensions	=	(250mm × 400mm)
Columns : interior	=	(400mm × 400mm)
Flooring finish	=	25-mm thick flooring

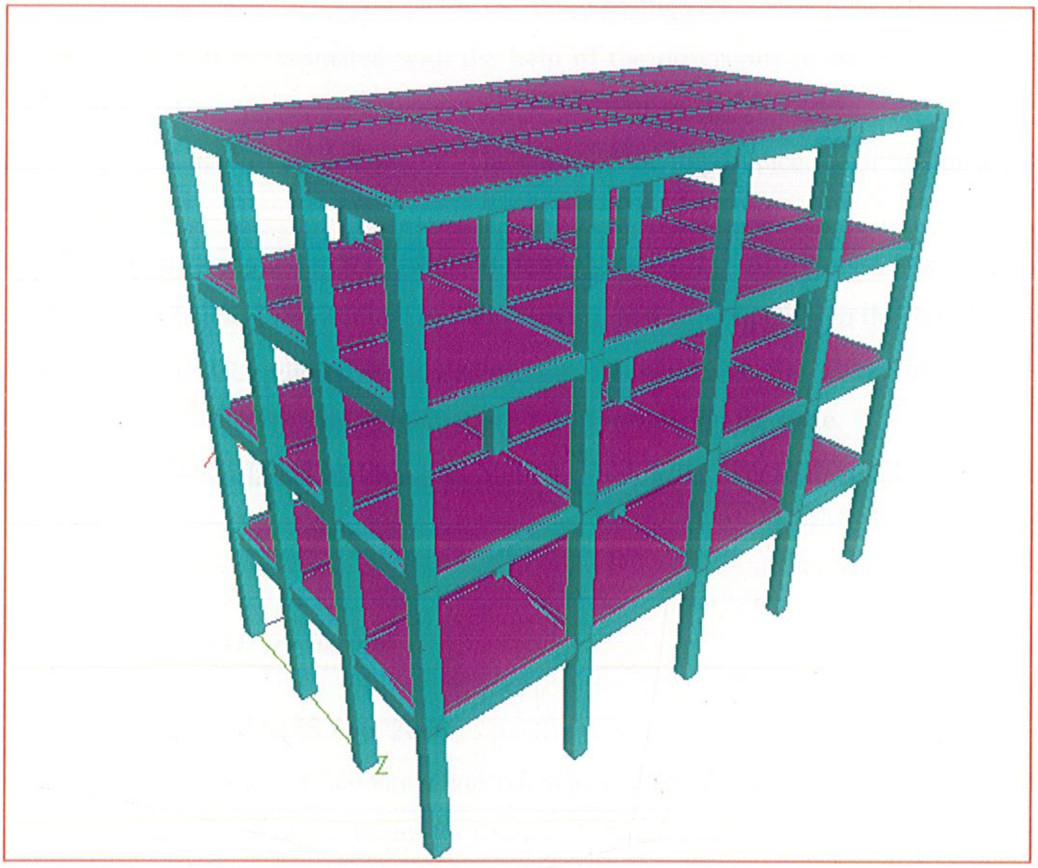

Fig. 2.1, ISOMETRIC VIEW OF BUILDING

2.3 ***Analysis Methodology***

- For the four-storey building, the analysis will be performed and the design will be done for the following loads:

1. Dead load
2. Live load
3. Earthquake load
4. Wind load

- The dead load will be worked out by assuming a certain thickness for the slab and then the actual thickness will be accordingly provided after calculating the required value. The load due to the flooring – screed, finishes, tiles etc. will be given due consideration and an allowance will be made for future erection of partitions.

- Due to increased emphasis being laid on the design of earthquake resistant structures nowadays, the earthquake forces will be estimated with the help of the provisions of the revised Seismic Code (IS:1893-2002).
- The proposed building will lie in Zone IV. The value of the importance factor assigned to the entire structure is 1.
- The load will initially applied to the slabs and through trapezoidal distribution it will be transmitted to the columns via beams (longitudinal and transverse), and consequently to the foundations.
- The designing will be done after analyzing the structure for the above-mentioned loads – individually and for different load combinations recommended in the code.
- The members will be designed by the Limit State method, according to the guidelines prescribed by IS: 456-2000.

2.3.1 *Inputs given to STAAD for design*

- Code was selected as IS 456: 2000
- Compressive strength of concrete was taken as $25N/mm^2$ and Fe 415 steel was used.
- Clear cover for column is 40mm for beam is 25mm and for slab is 20mm.
- Maximum size of main R/F is 25mm and minimum R/F is 16mm
- Maximum size of secondary R/F is 12mm and minimum R/F is 8mm.

CHAPTER 3

CALCULATIONS AND ANALYSIS

3.1 *General*

In this chapter, various loads acting on different beams and columns of the building and coming from slabs are calculated. The intensities of loads have been picked up from IS: 875 part 1, 2 and 3. The distribution of loads from slab to beams has been done as per IS-456: 2000, i.e. trapezoidal method of distribution has been adopted.

The calculation of loads coming from floors and roof and analysis is done by output of STAAD-pro.

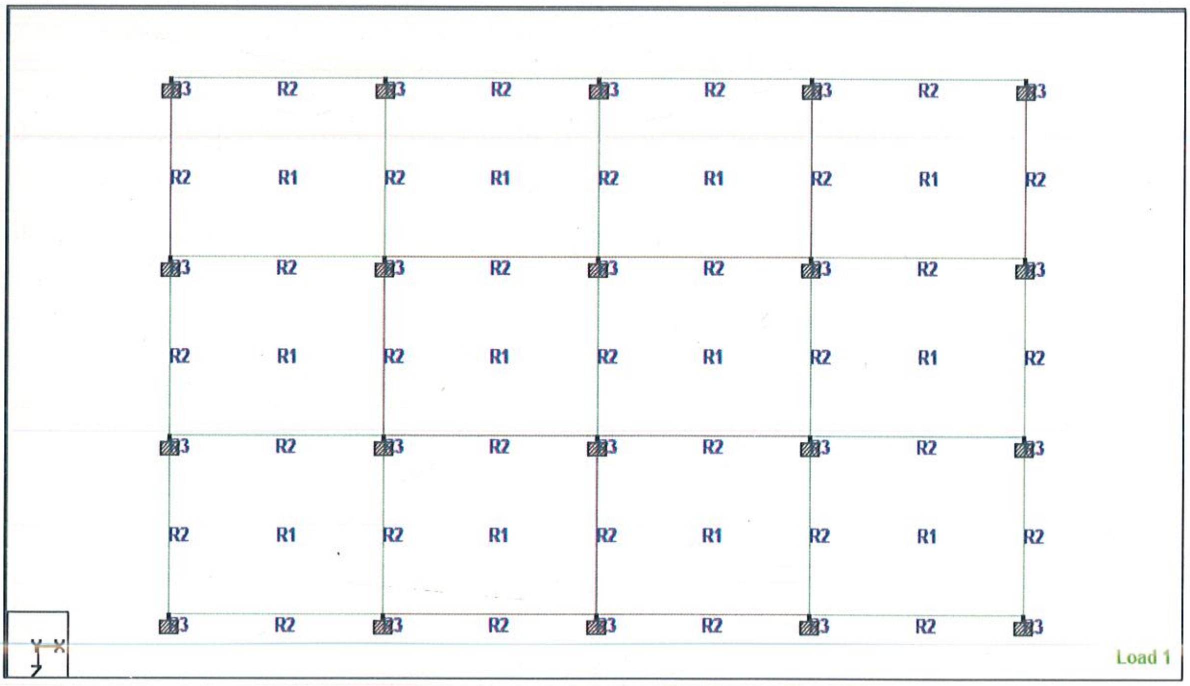

Fig. 3.1, FLOOR PLAN OF THE BUILDING

corresponding to the ultimate safety requirement is often called as the MAXIMUM CONSIDERED EARTHQUAKE (MCE) .Generally DBE is half the MCE"

b) The actual forces that appear on the structures during earthquakes are much higher than the design forces specified in the code. The basic criteria for earthquake resistant design should be based on lateral strength as well as deformability and ductility capacity of the structure with limited damage but no collapse. Ductility in the structures will arise from inelastic material, behavior and detailing of reinforcement in such a manner that brittle failure is avoided and ductile behavior is induced by allowing steel to yield in controlled manner.

c) The design lateral forces specified in the code shall be considered in each of the two orthogonal directions of structures. For structures which have lateral force resisting element in two orthogonal directions only the design lateral force shall be considered along one direction at time and or in both direction simultaneously.

d) Earthquake generating vertical inertial forces is to be considered in design unless it is not significant. Vertical acceleration should be considered in structures with large spans, those in which stability is the criterion for design or for overall stability of the structures.

e) The response of a structure to the ground vibrations is a function of the nature of foundation of the soil; materials; form; size and mode of construction of structures; and the duration and characteristics of ground motion. The map showing seismic zones of India is shown below.

3.2 *Calculation of Loads*

3.2.1 *Intensities of Dead Loads*

Terrace:

Thickness of slab	= 110mm
Self weight of RCC slab on slab	= 0.11×24.8 kN/m^3
	= 2.728kN/m^2
Total dead load on terrace	= 2.728KN/m^2

Floor:

Slab thickness	= 110mm
Self wt. of RCC slab	= 0.11×24.8 kN/m^3
	= 2.728kN/m^2
Total dead load on floor	= 2.728KN/m^2

3.2.2 *Intensities of Live Loads*

Terrace	= 0.75 kN/m^2
Floors	= 2.5 kN/m^2
Wall thickness	= 230mm
Unit weight of brick wall	= 15.17kN/m

3.3 *Seismic Design Philosophy*

The philosophy of seismic design can be summarized as:

a) The design philosophy adopted in the code is to ensure that structures possess at least a minimum strength to

i) Resist minor earthquake (<DBE) which may occur frequently, without damage

ii) Resist moderate earthquake (DBE) without significant structural damage through some non structural damage

iii) Resist major earthquake (MCE) without collapse.

"DESIGN BASIS EARTHQUAKE (DBE) is defined as the maximum earthquake that reasonably can be expected to experience at the site once during lifetime of the structure. The earthquake

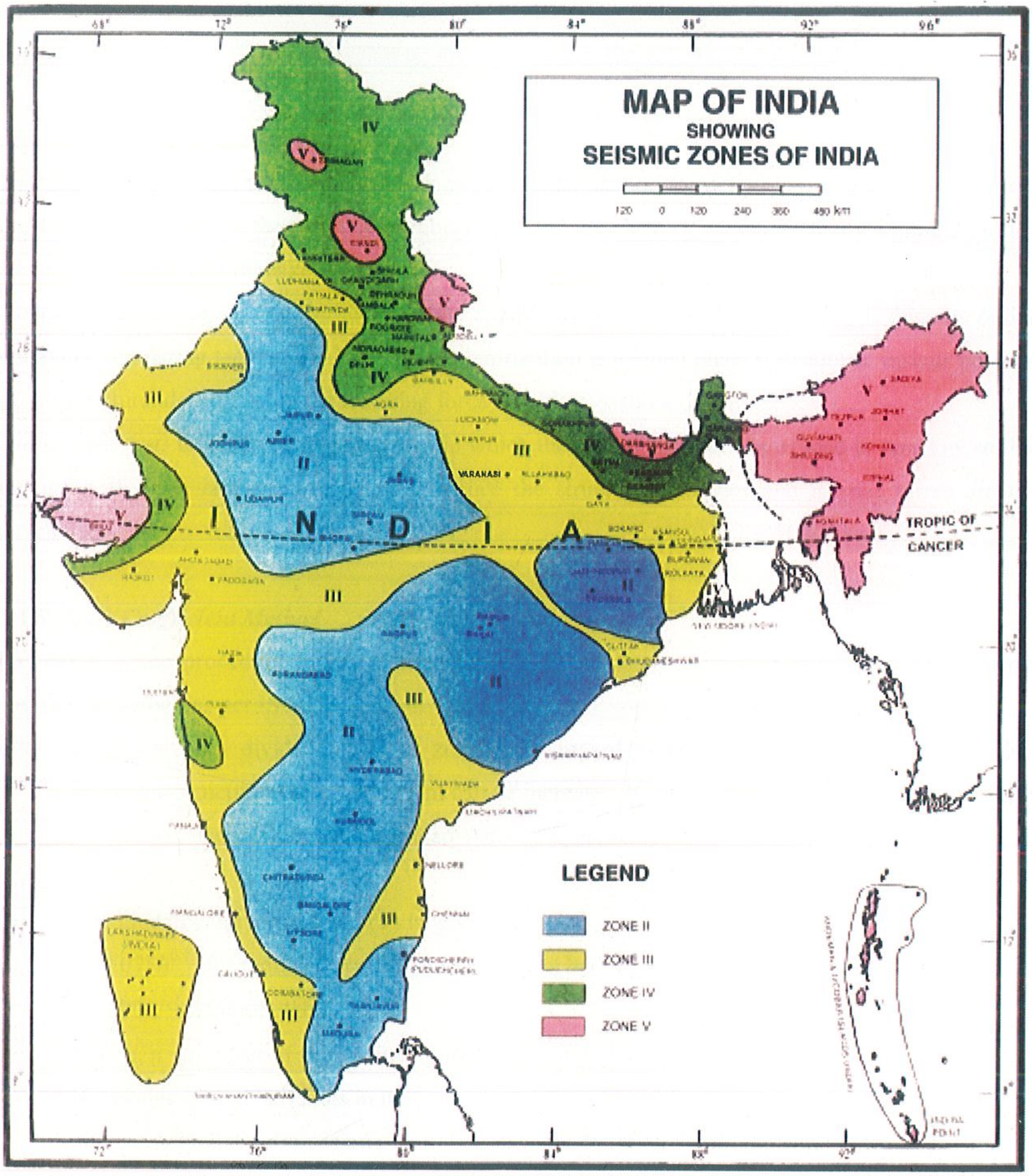

Fig 3.2, SEISMIC ZONES IN INDIA

Seismic design method

Conventional civil engineering structures are designed on the basis of two main criteria that are strength and rigidity. The strength is related to damageability or ultimate limit state, assuming that the force level developed in structures remains in the elastic range, or some limited plastic deformation.

The rigidity is related to the serviceability limit state, for which the structural displacement must remain in some limits. This assures that no damage occur in the non structural elements.

Methods for seismic testing

Shaking table test: This is the most realistic experimental method for determining dynamic response of the structure. In this test the structure is subjected to the load history which is usually a ground motion recorded during the earthquake is simulated.

Pseudo dynamic test: In this test the dynamic conditions are simulated. This testing is done by online computer simulating techniques. The load or deformation is applied quasi statically at various position of the structure, depending on the restoring force directly measured during the test.

Quasi static test: this is not a dynamic test, in which the rate of applications of load is very low so that the material strain rate effects do not influence the structural behavior and inertial forces are not developed.

3.4 *Seismic Coefficient Method*

Following is the procedure for calculating the equivalent lateral loads on buildings using seismic coefficient method as per IS-1893-2002.

i) India has been divided into four zones with regard to horizontal seismic coefficients. For important structures these coefficient can be increased by 50%. The horizontal earthquake force should be calculated for full dead load and some percentage of live loads.

ii) The fundamental time period (T) is given by:

$T=0.075h^{0.75}$ for moment resisting frame without bracing or shear walls,

$T=0.09h/\sqrt{d}$ (for all others)

Where, n = number of storeys including basement

H= total height of buildings in m.

d = maximum base dimension of building in m, in direction parallel to applied seismic force.

The base shear is calculated by the following formula:

$$V_B = A_h W$$

Where

A_h=design horizontal seismic coefficient

W=total dead load and appropriate percentage of live load

Where, A_h is given by

$$A_h = \frac{ZIS_a}{2Rg}$$

Where,

Z = Zone factor, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE)

I = Importance factor, depending upon the functional use of the structures, characterised by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance.

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterised by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0.

S_a/g = Average response acceleration coefficient for rock or soil sites as

Distribution of forces along the height of building is given by

$$Q_i = V_B (W_i h_i^2 / \sum w_i h_i^2)$$

Where

Q_i = Lateral forces at the floor i

V_B = Base shear

W_i = Load of the floor i

h_i = height measured from the base of the building to the floor i

n = number of storeys including the basement.

3.5 *Wind load calculations*

Design Wind Speed (V_z)

The basic wind speed (V_b) for any site shall be obtained and shall be modified to include the following effects to get design wind velocity at any height (V_z) for the chosen structure: It can be mathematically expressed as follows:

$$\boxed{V_z = V_b * k1 * k2 * k3}$$

Where,

***Risk Coefficient* (k1 Factor)** — gives basic wind speeds for terrain Category 2 as applicable at 10 m above ground level based on 50 years mean return period. The suggested life period to be assumed in design and the corresponding $k1$ factors for different class of structures for the purpose of design is given. In the design of all buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used except as specified.

***Terrain, Height and Structure Size Factor* (*k2 Factor*)**

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. Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of any building or structure may be suitably planned. Terrain in which a specific structure stands shall be assessed as being one of the following terrain categories:

a) *Category* 1 — Exposed open terrain with 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 heights generally between 1.5 to 10 m.

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. Open land adjacent to sea coast 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 10 m in height with or without a few isolated tall structures.

NOTE 1 — This category includes well wooded areas, and shrubs, towns and industrial areas full or partially developed.

NOTE 2 — It is likely that the next higher category than this will not exist in most design situations and that selection of a more severe category will be deliberate.

NOTE 3 — Particular attention must be given to performance of obstructions in areas affected by fully developed tropical cyclones. Vegetation which is likely to be blown down or defoliated cannot be relied upon to maintain Category 3 conditions. Where such situation may exist, either an intermediate category with velocity multipliers midway between the values for Category 2 and 3, or Category 2 should be selected having due regard to local conditions.

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

NOTE — This category includes large city centres, generally with obstructions above 25 m and well developed industrial complexes.

Variation of wind speed with height for different sizes of structures in different terrains
(***k2 factor***) — Gives multiplying factors (*k*2) by which the basic wind speed given shall be multiplied to obtain the wind speed at different heights, in each terrain category for different sizes of buildings/structures.

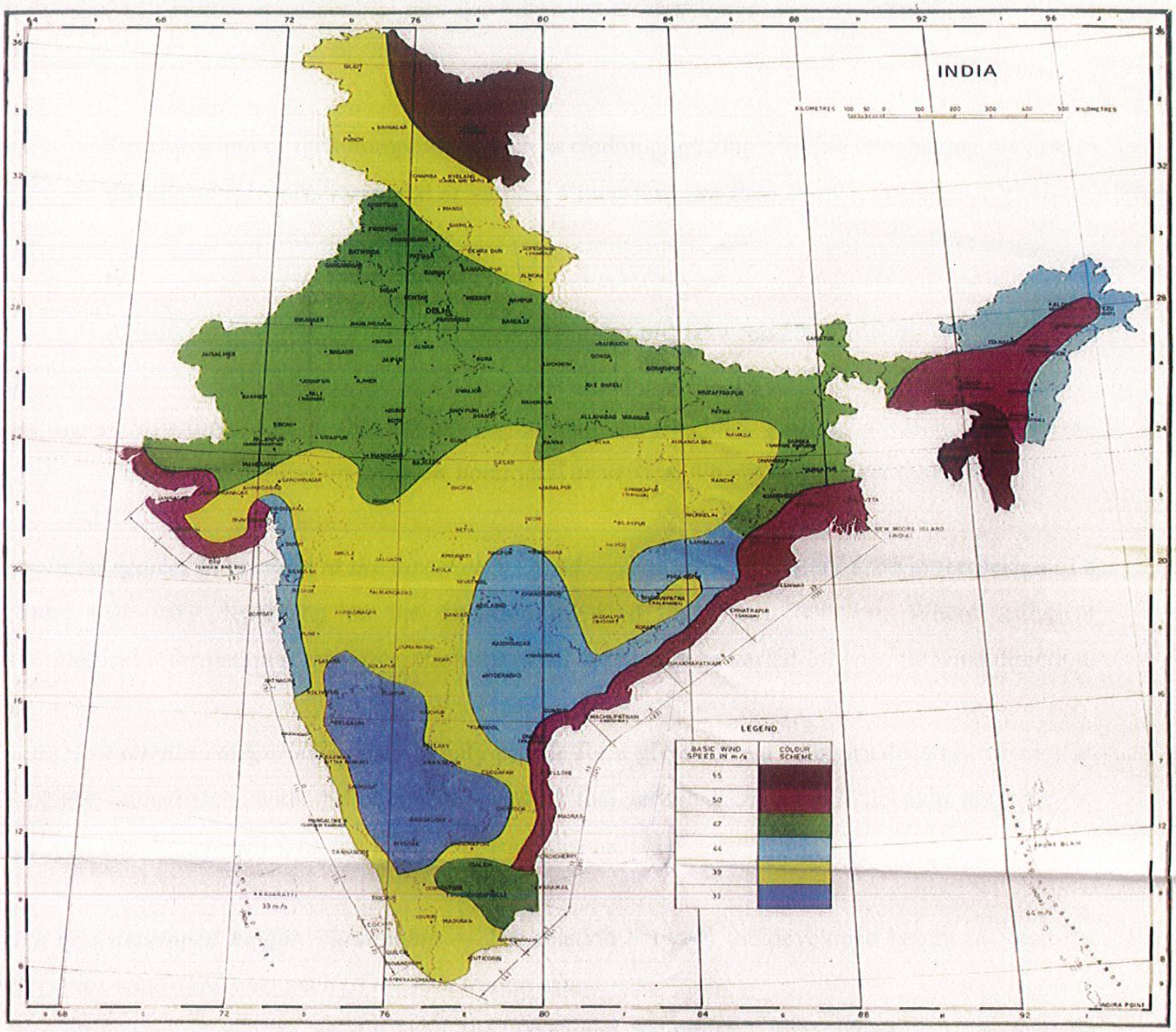

Fig 3.3, WIND ZONES IN INDIA

The buildings/structures are classified into the following three different classes depending upon their size:

Class A — Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) less than 20 m.

Class B — Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) between 20 and 50 m.

Class C — Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) greater than 50 m.

Terrain categories in relation to the direction of wind — The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Where sufficient meteorological information is available, the basic wind speed may be varied for specific wind direction.

Changes in terrain categories —The velocity profile for a given terrain category does not develop to full height immediately with the commencement of that terrain category but develop gradually to height (h_x) which increases with the fetch or upwind distance (x).

Fetch and developed height relationship — The relation between the developed height (h_x) and the fetch (x) for wind-flow over each of the four terrain categories.

b) For structures of heights greater than the developed height (h_x), the velocity profile may be determined in accordance with the recommendations of code.

Topography (k3 Factor) — The basic wind speed V_b takes account of the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliffs, steep escarpments, or ridges.

The effect of topography will be significant at a site when the upwind slope (θ) is greater than about 3^{o}, and below that, the value of $k3$ may be taken to be equal to 1.0. The value of $k3$ is confined in the range of 1.0 to 1.36 for slopes greater than 3^{o}. It may be noted that the value of $k3$ varies with height above ground level, at a maximum near the ground, and reducing to 1.0 at higher levels.

Design Wind Pressure — The design wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind velocity:

$$\boxed{p_z = 0.6\ V_z^2}$$

Where,

p_z = design wind velocity in N/m^2 at height z, and

V_z = design wind velocity in m/s at height z.

The wind load calculated by us that we used in our building design is shown in the table given below.

WIND LOAD CALCULATIONS						
HEIGHT OF BUILDING (m)	SPEED OF WIND V_B (m/s)	K_1	K_2	K_3	V_z	DESIGN WIND PRESSURE kN/m²
1	33	1	1.05	1	34.65	0.720
2	33	1	1.05	1	34.65	0.720
3	33	1	1.05	1	34.65	0.720
4	33	1	1.05	1	34.65	0.720
5	33	1	1.05	1	34.65	0.720
6	33	1	1.05	1	34.65	0.720
7	33	1	1.05	1	34.65	0.720
8	33	1	1.05	1	34.65	0.720
9	33	1	1.05	1	34.65	0.720
10	33	1	1.05	1	34.65	0.720
11	33	1	1.058	1	34.914	0.731
12	33	1	1.066	1	35.178	0.742
13	33	1	1.074	1	35.442	0.754
14	33	1	1.082	1	35.706	0.765
15	33	1	1.09	1	35.97	0.776

TABLE 1, WIND LOAD CALCULATIONS

3.6 *Final analysis*

STAAD.Pro 2004 has been used to finally analyze the building. The inputs given to the program were:

- Geometry of the structure
- Properties of the materials of beams, columns and slabs are given.
- Fixed supports are provided at base nodes.
- Loads combinations adopted as per IS-456:2000 and IS-875:1987 Part-5 are as follows:

TABLE-2: LOAD COMBINATIONS AS PER IS-456:2000 AND IS-800:1987 (PART-5)			
Load Case no.	Combination	Load Case no.	Combination
1	1.5 (DL + LL)	15	1.5 (DL – EL.X)
2	1.2 (DL + LL + EL.X)	16	1.5 (DL + EL.Z)
3	1.2 (DL + LL – EL.X)	17	1.5 (DL – EL.Z)
4	1.2 (DL + LL + EL.Z)	18	1.5 (DL + WL.X)
5	1.2 (DL + LL + EL.Z)	19	1.5 (DL + WL.X)
6	1.2 (DL + LL – EL.Z)	20	1.5 (DL – WL.X)
7	1.2 (DL + LL + WL.X)	21	1.5 (DL – WL.Z)
8	1.2 (DL + LL – WL.X)	22	1.5 (DL + WL.Z)
9	1.2 (DL + LL + WL.Z)	23	0.9DL + 1.5EL.X
10	1.2 (DL + LL - WL.Z)	24	0.9DL – 1.5EL.X
11	1.5 (DL + EL.X)	25	0.9DL + 1.5EL.Z
12	0.9DL – 1.5EL.Z	26	0.9DL + 1.5WL.Z
13	0.9DL + 1.5WL.X	27	0.9DL –1.5WL.Z
14	0.9DL – 1.5WL.X		

3.7 *Output of analysis by STAAD-pro*

The STAAD.pro gives output in many forms. In the present case, the absolute maximum forces coming in the members are obtained from Post-processing mode and compiled in Table-3 below. The members were ultimately designed for these forces as these are critical ones.

Table-3: Maximum Critical Forces in Members

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	3	5 1.5(DL+IL)	3	924.589	0.332	-126.363	-0.097	176.986	0.856
Min Fx	184	21 1.5(DL-WL_Z)	76	-156.633	-0.456	145.015	0.812	-309.057	-0.747
Max Fy	160	21 1.5(DL-WL_Z)	52	-81.751	183.665	0.224	-0.794	0.186	225.980
Min Fy	2	13 1.5(DL+EL_X)	3	-8.647	-52.302	-6.797	-0.851	-4.407	40.637
Max Fz	62	21 1.5(DL-WL_Z)	26	343.266	-3.817	178.718	-0.486	-346.839	-4.356
Min Fz	64	21 1.5(DL-WL_Z)	29	512.179	0.027	-185.773	-0.095	-355.074	0.002
Max Mx	90	21 1.5(DL-WL_Z)	41	-10.507	46.426	7.187	0.917	-4.833	30.874
Min Mx	63	21 1.5(DL-WL_Z)	27	-10.507	44.594	6.950	-0.917	-4.360	27.210
Max My	64	21 1.5(DL-WL_Z)	28	512.179	0.027	-165.981	-0.095	260.495	0.098
Min My	75	21 1.5(DL-WL_Z)	37	513.314	-0.000	-185.705	0.000	-355.090	0.000
Max Mz	38	21 1.5(DL-WL_Z)	2	79.354	182.400	0.151	-0.824	-0.699	252.830
Min Mz	160	21 1.5(DL-WL_Z)	77	-92.354	115.400	0.224	-0.794	0.859	-222.618

Maximum nodal displacement is shown in Table-4.

Table-4: Maximum nodal displacements

	Node	L/C	Horizontal	Vertical	Horizontal	Resultant	Rotational		
			X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	9	13 1.5(DL+EL_X)	3.402	-1.236	-59.056	59.167	-0.001	-0.000	-0.000
Min X	25	9 1.2(DL+IL-EL_X)	-2.722	-1.062	-47.244	47.334	-0.001	0.000	0.000
Max Y	100	21 1.5(DL-WL_Z)	0.007	0.240	-63.233	63.233	-0.001	0.000	-0.000
Min Y	15	5 1.5(DL+IL)	-0.000	-1.843	-59.045	59.073	-0.001	-0.000	0.000
Max Z	9	7 WL_Z	-0.001	0.034	2.780	2.781	0.000	0.000	0.000
Min Z	84	21 1.5(DL-WL_Z)	-0.007	0.240	-63.233	63.233	-0.001	-0.000	0.000
Max rX	2	7 WL_Z	-0.002	0.018	0.893	0.894	0.000	0.000	0.000
Min rX	77	21 1.5(DL-WL_Z)	-0.030	0.158	-19.628	19.629	-0.005	-0.000	-0.000
Max rY	96	21 1.5(DL-WL_Z)	0.030	0.158	-19.628	19.629	-0.005	0.000	0.000
Min rY	77	21 1.5(DL-WL_Z)	-0.030	0.158	-19.628	19.629	-0.005	-0.000	-0.000
Max rZ	21	23 1.5(DL-WL_X)	-0.838	-0.640	-18.286	18.316	-0.005	0.000	0.001
Min rZ	2	13 1.5(DL+EL_X)	0.805	-0.635	-18.285	18.314	-0.005	-0.000	-0.001
Max Rst	15	21 1.5(DL-WL_Z)	-0.000	-1.721	-63.213	63.236	-0.001	-0.000	0.000

Maximum support reactions are shown in Table-5.

Table-5: Maximum Support Reactions.

	Node	L/C	Horizontal	Vertical	Horizontal	Moment		
			Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	1	23 1.5(DL-WL_X)	12.860	657.030	140.350	293.535	0.674	-20.448
Min Fx	22	28 1.5(DL+WL_X)	-12.860	657.030	140.350	293.535	-0.674	20.448
Max Fy	4	5 1.5(DL+IL)	-0.332	924.589	146.155	299.921	-0.097	0.307
Min Fy	97	21 1.5(DL-WL_Z)	-0.456	-156.633	148.795	309.057	-0.812	0.747
Max Fz	29	21 1.5(DL-WL_Z)	-0.027	512.179	185.773	355.074	-0.095	-0.002
Min Fz	12	7 WL_Z	-0.000	-20.160	-11.590	-14.260	0.000	0.000
Max Mx	37	21 1.5(DL-WL_Z)	0.000	513.314	185.705	355.090	0.000	-0.000
Min Mx	37	7 WL_Z	-0.000	1.623	-8.009	-15.905	0.000	0.000
Max My	76	21 1.5(DL-WL_Z)	0.456	-156.633	148.795	309.057	0.812	-0.747
Min My	97	21 1.5(DL-WL_Z)	-0.456	-156.633	148.795	309.057	-0.812	0.747
Max Mz	22	28 1.5(DL+WL_X)	-12.860	657.030	140.350	293.535	-0.674	20.448
Min Mz	1	23 1.5(DL-WL_X)	12.860	657.030	140.350	293.535	0.674	-20.448

CHAPTER -4

DESIGN OF MEMBERS USING SPREDSHEETS

4.1 *General*

In this section, various members of the frame have been designed manually and compared with the design given by STAAD.pro for verification of the work.

4.2 *Design of Slabs*

Formula used in design of slab

Bending Moment at continuous edge in short span

$$M\,(x-) = \alpha_{x-}\, w l_x^2$$

Bending Moment at mid span in short span,

$$M\,(x+) = \alpha_{x+}\, w l_x^2$$

Bending Moment at continuous edge in long span,

$$M\,(y-) = \alpha_{y-}\, w l_x^2$$

Bending Moment at mid span in long span,

$$M\,(y+) = \alpha_{y+}\, w l_x^2$$

Design Bending Moment (Factored BM),

$$M_u = 1.5M$$

Limiting BM for a section is,

$$M_{u,\lim} = 0.138 f_{ck} bd^2$$

If,

$M_u < M_{u,\lim}$, then section is an under-reinforced section

$M_u = M_{u,\lim}$, then section is a balanced section

$M_u > M_{u,\lim}$, then section is an over-reinforced section, and it needs to be redesigned.

Area of steel reinforcement in tension zone

1. For an under-reinforced section,

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

2. For a balanced section,

$$M_u = 0.87 f_y A_{st} d \left[1 - 0.42 \frac{x_{u,\max}}{d}\right]$$

The values of $\frac{x_{u,\max}}{d}$ are given in the table below,

For detailing of reinforcement SP-16: Design Aids to IS 456:2000 are used.

Table 5: Characteristic Strength of Steel and x_{umax}/d	
Characteristic Strength of Steel, f_y	**x_{umax}/d**
250	**0.53**
415	**0.48**
500	**0.46**

We use the above given data for input the data from the users and design the module for designing the various members like beams, columns, slabs and footing. The user form developed is moreover software which is created by using visual basic software to develop user-interface and the calculations are being made by MS-Excel at the backend. During this project we learn to develop the curves used in SP-16 and also met with the shortcomings of SP-16. The curves required for design of columns are developed itself in the software and the curves are not dependent on any condition. The user interface developed is shown in the figures and a example is taken to make the understanding of the software.

4.3 *Slab Design using excel*

Slab panels that deform with significant curvatures in two orthogonal directions must be designed as *two-way slabs*, with the principal reinforcement placed in the two directions

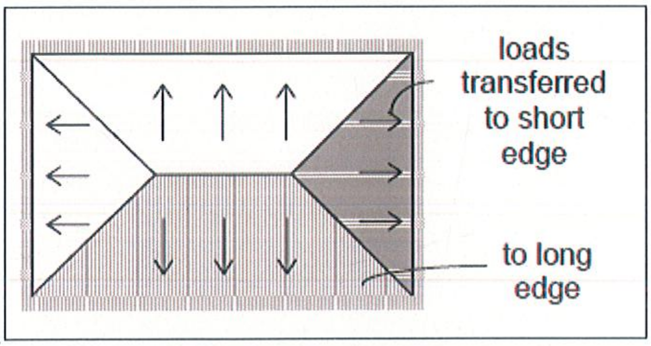

Fig. 4.1, LOAD TRANSFER IN SLAB

The user form of the two-way slab (as all the slabs in our building are two-way) design is shown in the figure:

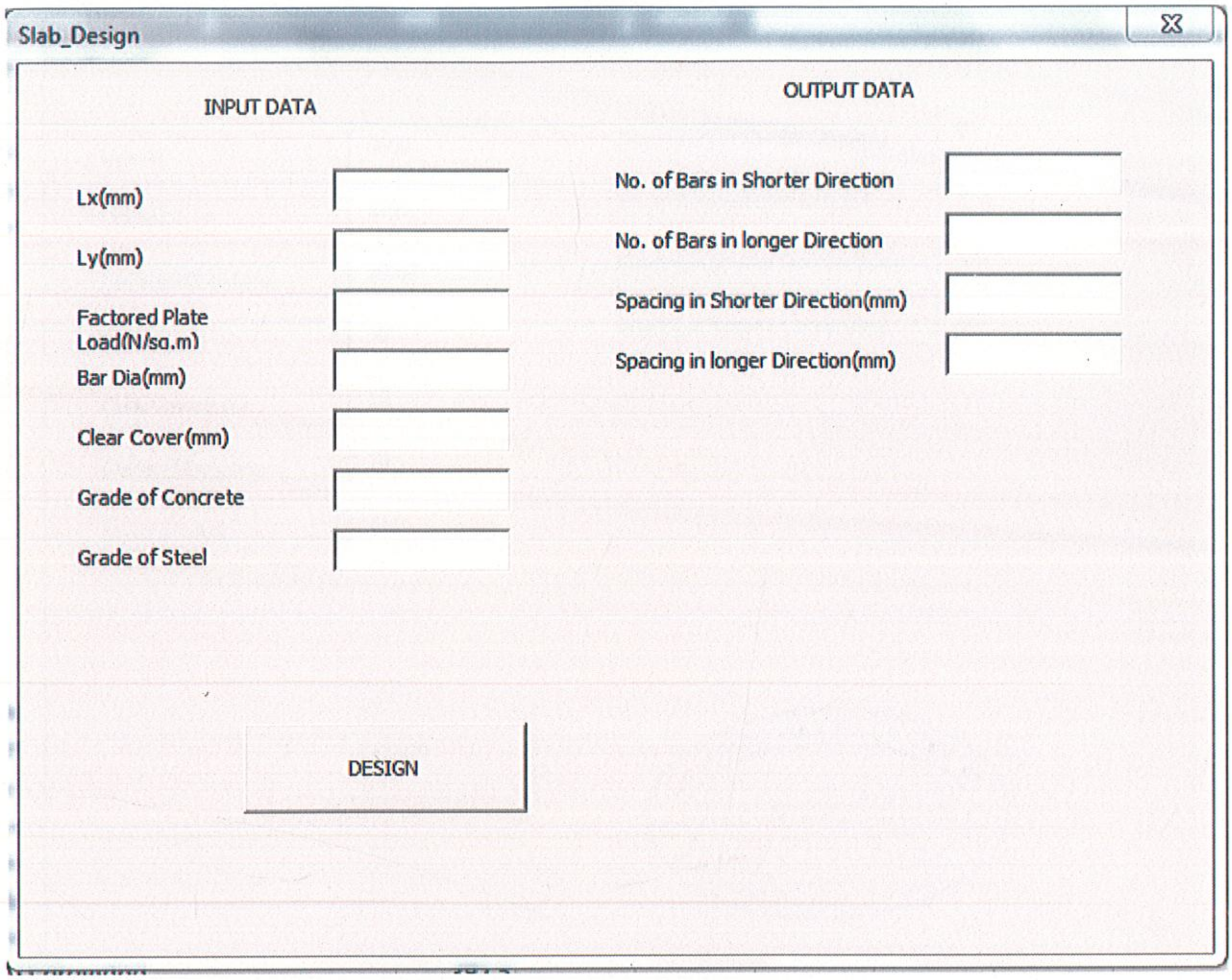

Fig 4.2, MODULE FOR SLAB DESIGN

The thickness of the slab is generally based on deflection control criteria, and the reinforcements in the two orthogonal directions are designed to resist the calculated maximum bending moments in the respective directions at the critical sections. [Additional reinforcement may be required at the corners of two-way slabs in some cases]. The slab thickness should be sufficient against shear, although shear is usually not a problem in two-way slabs subjected to uniformly distributed loads.

The data in the above module is filled and the working of this module is shown in the figure below.

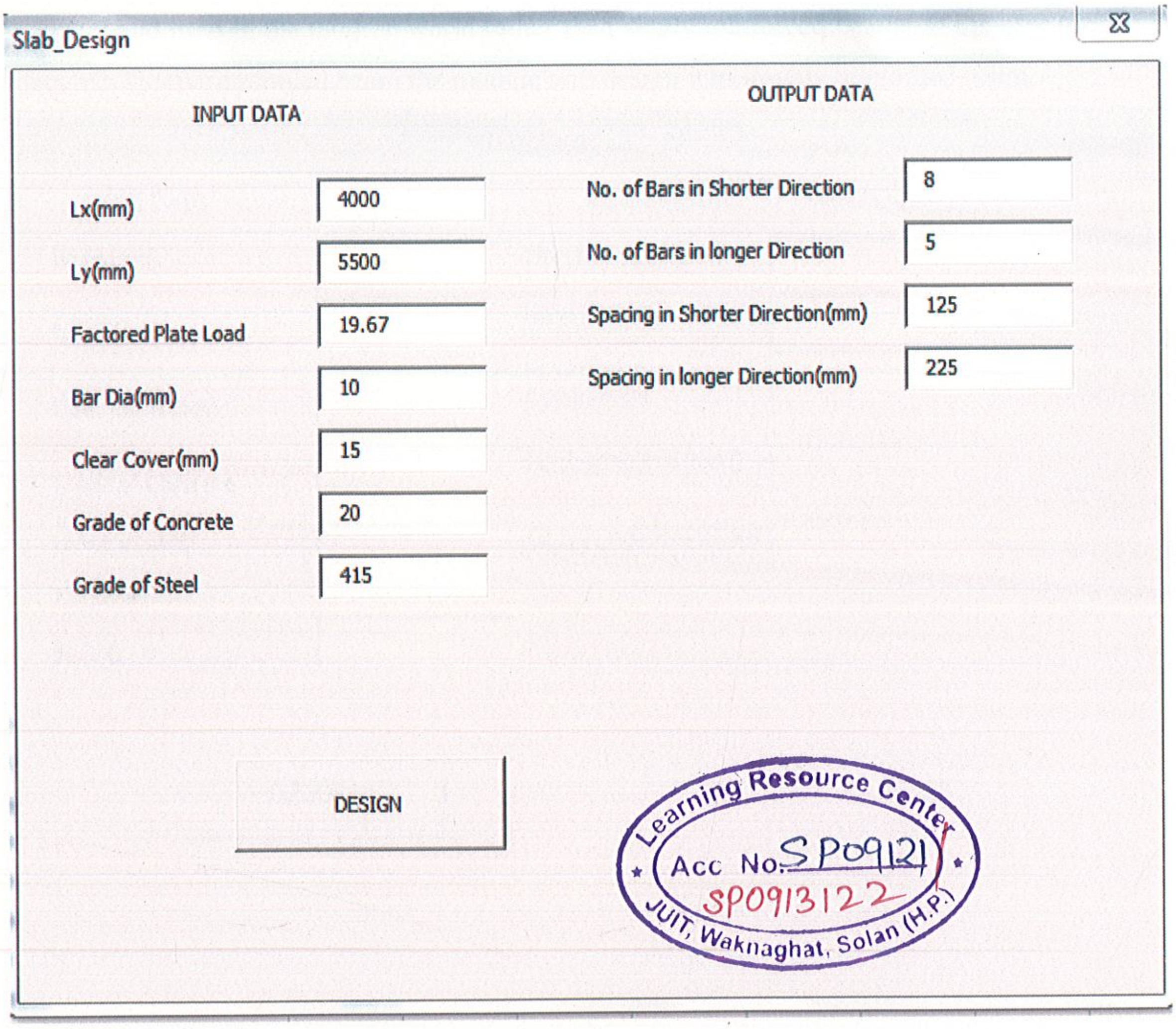

Fig. 4.3, AN EXAMPLE SHOWING THE WORKING OF MODULE DEVELOPED FOR SLAB DESIGN

4.4 *Beam Design using excel*

Doubly reinforced sections are generally resorted to in situations where the cross-sectional dimensions of the beam are restricted (by architectural or other considerations) and where singly reinforced sections (with $p_t = p_{t,lim}$) are not adequate in terms of moment-resisting capacity. Doubly reinforced beams are also used in situations where reversal of moments is likely (as in multi-storeyed frames subjected to lateral loads). The presence of compression reinforcement reduces long-term deflections due to shrinkage. All compression reinforcement must be enclosed by *closed* stirrups, in order to prevent their possible buckling and to provide some ductility by confinement of concrete. The beams in our problem are designed as doubly reinforced beams because these beams are more economical and used in the modern world rather than singly reinforced beams. If the user tries to input the data of a singly reinforced beam the module will design it as doubly reinforced beam.

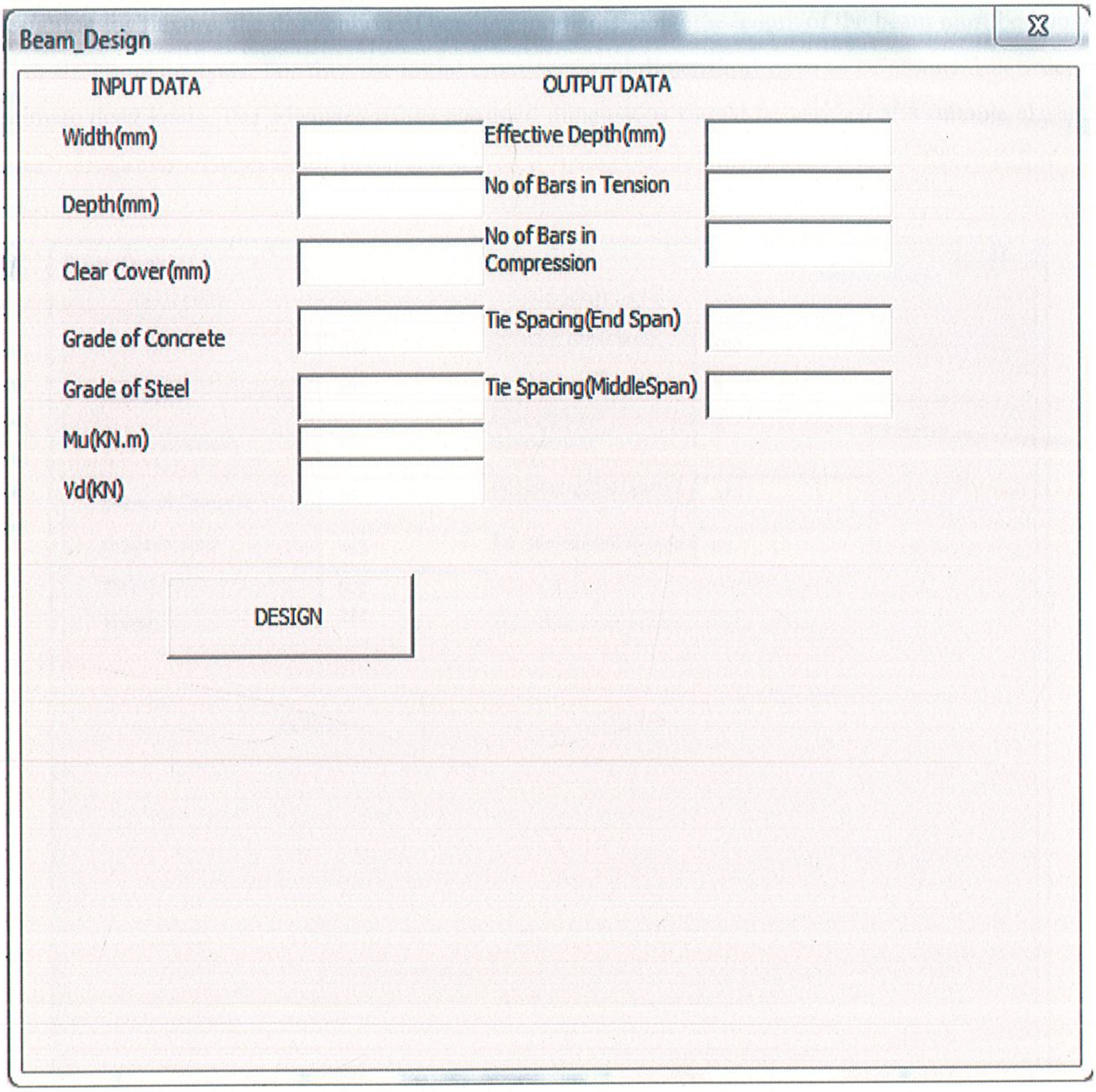

Fig. 4.4, MODULE FOR BEAM DESIGN

In design of a Beam section, the external loads (or load effects), material properties and the skeletal dimensions of the beam are known, and it is required to arrive at suitable cross-sectional dimensions and details of the reinforcing steel, which would give adequate *safety* and *serviceability*. In designing for flexure, the distribution of bending moments along the length of the beam must be known from structural analysis. For this, the initial cross-sectional dimensions have to be assumed in order to estimate dead loads. The adequacy of the assumed dimensions should be verified and suitable changes made, if required. The example taken and solved by using our module is shown in the figure below.

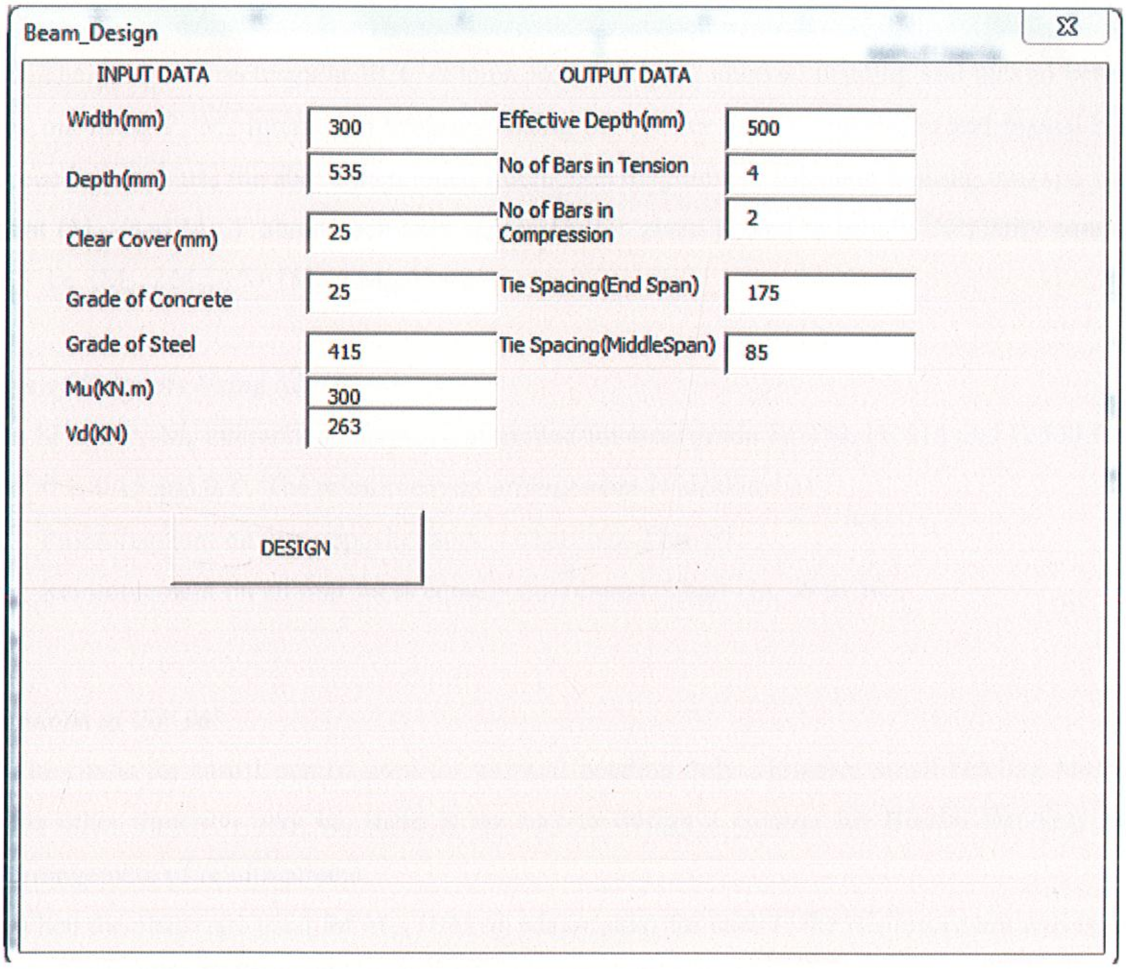

Fig. 4.5, AN EXAMPLE SHOWING THEWORKING OF MODULE DEVELOPED FOR BEAM DESIGN

4.5 *Column Design using Excel*

A 'compression member' is a structural element which is subjected (predominantly) to axial compressive forces. Compression members are most commonly encountered in reinforced concrete buildings as *columns*.

The 'column' is representative of all types of compression members, and hence, sometimes, the terms 'column' and 'compression member' are used interchangeably. The Code (Cl. 25.1.1) defines the column as a *compression member*, the 'effective length†' of which exceeds three times the least lateral dimension. The term 'pedestal' is used to describe a vertical compression member whose 'effective length' is less than three times its least lateral dimension [Cl. 26.5.3.1(h) of the Code].

The design of rectangular RCC column for axial load, uniaxial bending and biaxial bending is carried out using P_u-M_u Interaction Diagrams using SP 16. For axial compression and biaxial bending the procedure is to use the above mentioned Interaction diagrams to calculate limiting uniaxial bending moment (M_{uxl} and M_{uyl}) about each axes separately for given P_u and to satisfy inequality equation of IS-456 i.e. $(M_{ux} / M_{uxl})^{\alpha} + (M_{uy} / M_{uyl})^{\alpha} < 1$.

Design of Columns Using SP 16

In SP 16, P_u-M_u interaction charts are presented for steel grade Fe 250, Fe 415 and Fe500 for d'/D = 0.05, 0.1, 0.15 and 0.2 . The reinforcement arrangement is idealized as

1. Reinforcement on two opposite faces : Chart No. 27 to 38
2. Reinforcement on all four faces equally distributed: Chart No. 39 to 50.

Limitations of 'SP 16'

1. The charts for case 1 can be used for uniaxial bending only. However small bending Moment in the other direction may be, there is no way to design a column for Biaxial Bending for this arrangement of reinforcement.
2. When the charts are used for M_{ux} (BM @ Major axis) for case 1, the reinforcement will be on the smaller (width- b) faces. These charts do not accommodate reinforcement to be provided on larger (Depth-D) faces from detailing aspects. If a column with D = 900mm, minimum four bars are required on each D-face, the charts cannot account for these internal bars which may be of smaller diameter.

3. It is evident that larger diameter bars on the highly stressed face makes efficient use of reinforcement, even larger diameter bars in corner are used with advantage for biaxial bending, it cannot be accommodated by use of SP16.
4. There would be confusion in arranging the reinforcement as equally distributed for rectangular section with large D/b ratio.

An effort is made to develop a spreadsheet for design of RCC columns to overcome the above mentioned limitations.

The module developed for input of data to design the column is shown below. The module developed was to design the biaxial columns only as our problem only included only biaxial columns as the result of STAAD.Pro was analyzed. The module was developed by having all the checks that are specified in the code IS 456: 2000 for the design of compression member. We have tried to remove the drawbacks of SP-16 by plotting P_u-M_u Interaction Diagrams for 200 different depths of neutral axes.

The module or user-interface is shown below for the column design

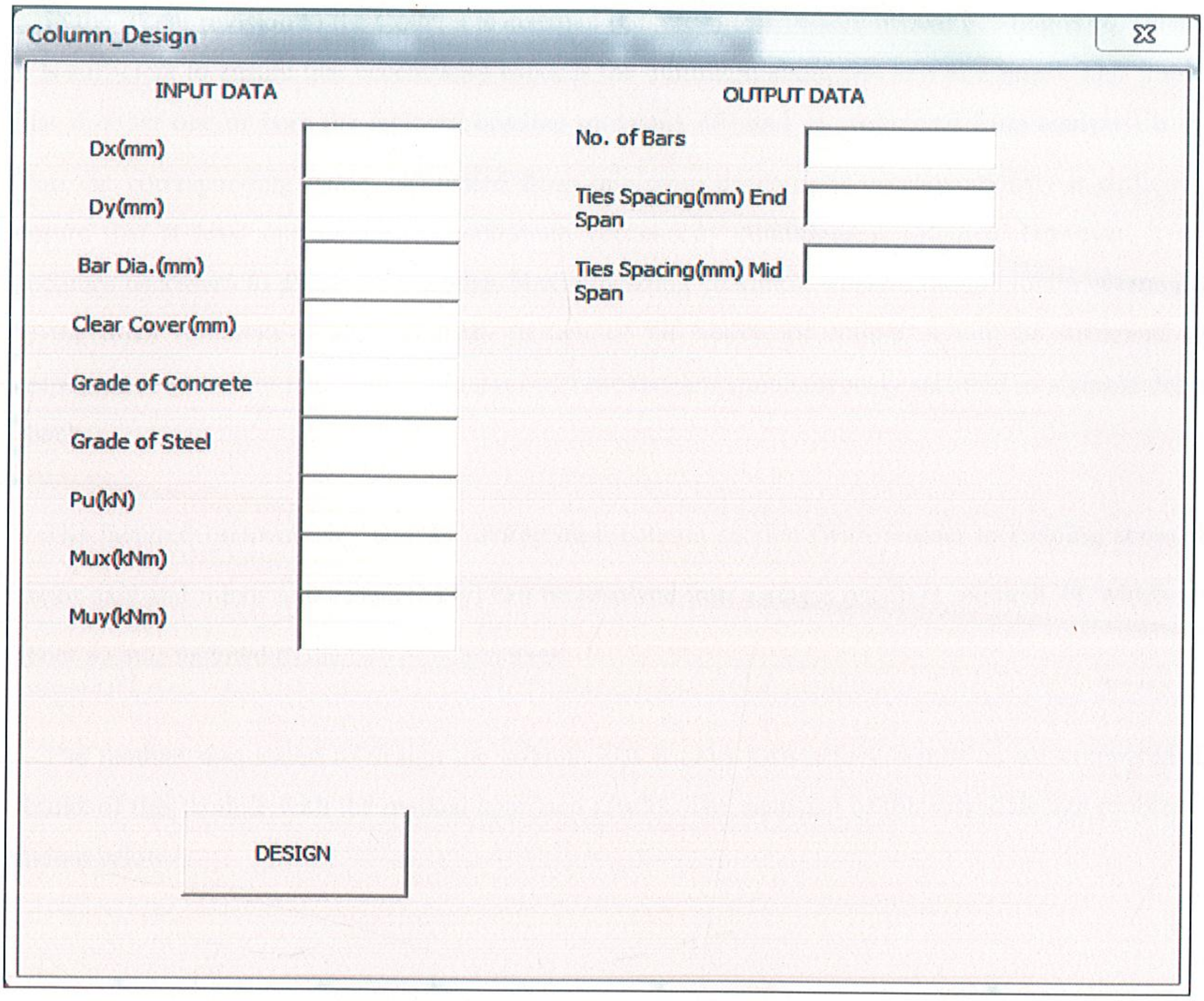

Fig. 4.6, THE MODULE DEVELOPED FOR COLUMN DESIGN

All columns are (in a strict sense) to be treated as being subject to axial compression combined with *biaxial* bending, as the design must account for possible eccentricities in loading (e_{min} at least) with respect to both major and minor principal axes of the column section. *Uniaxial loading* is an idealised approximation which can be made when the *e*/*D* ratio with respect to one of the two principal axes can be considered to be negligible. Also, if the *e*/*D* ratios are negligible with respect to both principal axes, conditions of *axial loading* may be assumed, as a further approximation.

In the recent revision to the Code, it is clarified (Cl. 25.4) that "where biaxial bending is considered, it is sufficient to ensure that eccentricity exceeds the minimum about one axis at a time". This implies that if either one or both the factored bending moments M_{ux} and M_{uy} (obtained from analysis) is less than the corresponding value, calculated from minimum eccentricity considerations , it suffices to ensure that at least one of the two minimum eccentricity conditions is satisfied. However, it also becomes necessary to check for the other biaxial bending condition wherein the minimum eccentricity in the other direction is also satisfied. In lieu of the above, of course, it will be sufficient and conservative to ensure that both minimum eccentricities are simultaneously satisfied in a single design check.

The factored moments M_{ux} and M_{uy} acting on a column section (with respect to bending about the major axis and minor axis respectively) can be resolved into a single resultant moment M_u which acts about an axis inclined to the two principal axes.

The module was tested to design the column and it gave satisfactory results as we compared the results of this module with the manual approach results. The snapshot of the solved design problem is shown below.

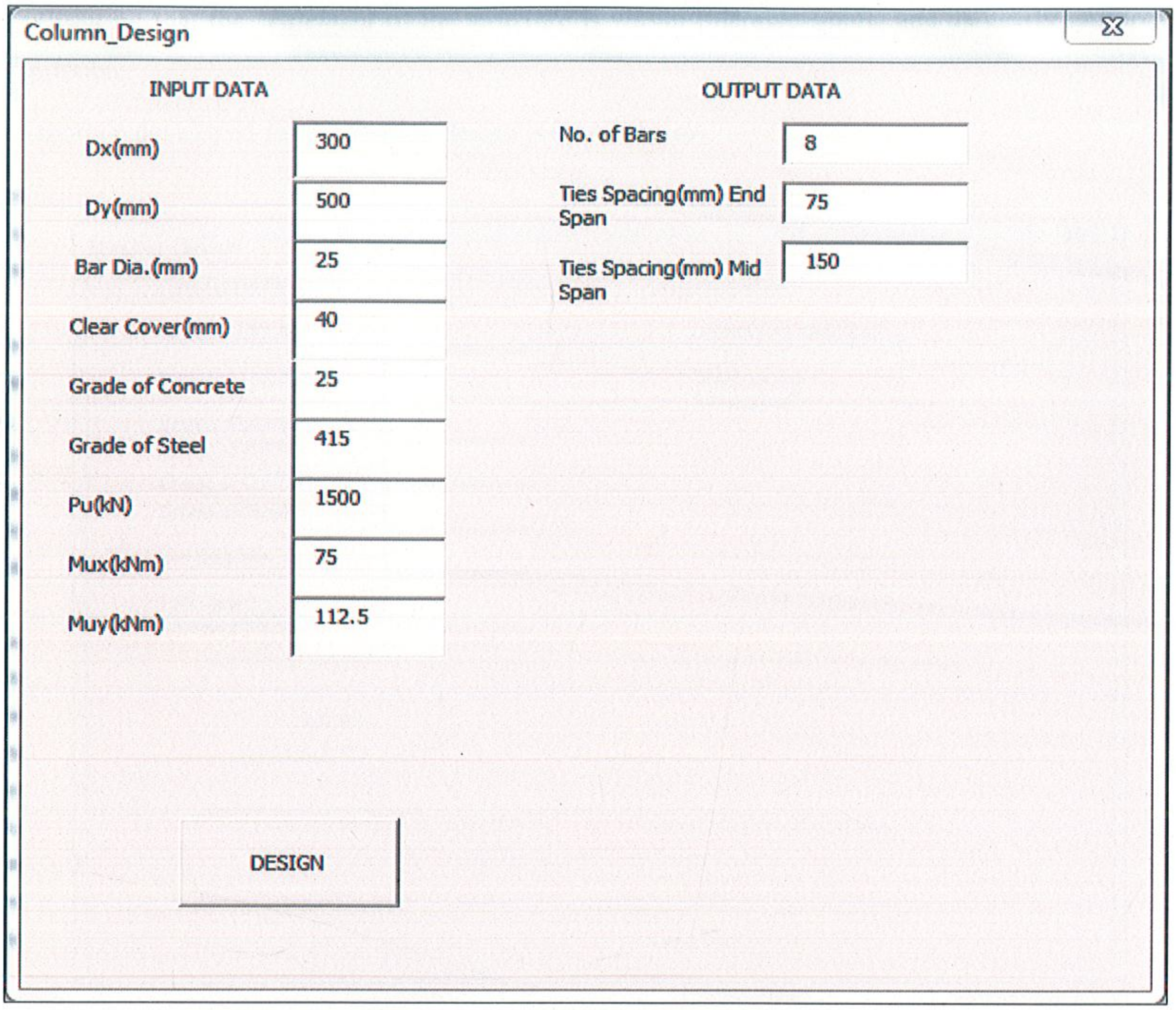

Fig. 4.7, AN EXAMPLE SHOWING THE WORKING OF MODULE DEVELOPED FOR COLUMN DESIGN

4.6 *Design of isolated square footing using excel*

Isolated footing Transfers individual column loads directly to the soil. If a single spread footing interferes with another spread footing, the two can be combined to form a combined footing. In a typical structure built on ground, that part of the structure which is located above ground is generally referred to as the *superstructure*, and the part which lies below ground is referred to as the *substructure* or the 'foundation structure' (or simply, foundation). The purpose of the foundation is to effectively support the superstructure by

- Transmitting the applied load effects (reactions in the form of vertical and horizontal forces and moments) to the soil below, without exceeding the 'safe bearing capacity' of the soil, and

➢ Ensuring that the *settlement* of the structure is within tolerable limits, and as nearly uniform† as possible.

The module developed for the footing design is shown below.

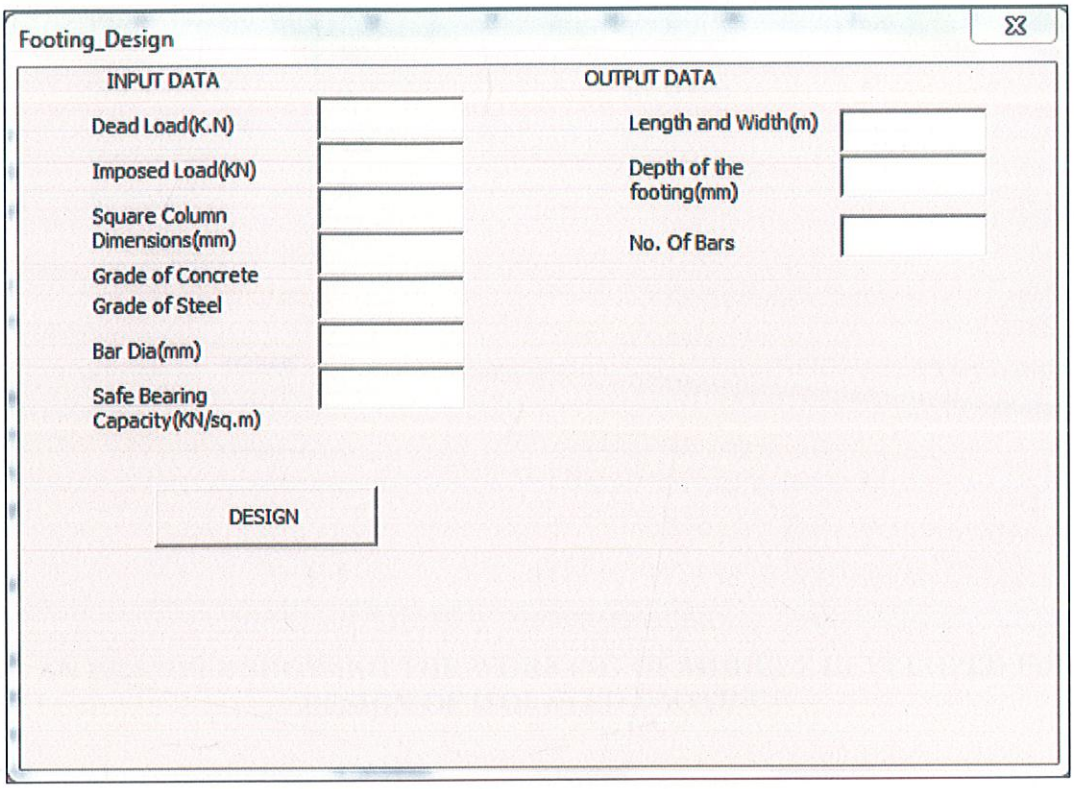

Fig. 4.8, MODULE FOR ISOLATED FOOTING DESIGN

Design

The major design considerations in the structural design of a footing relate to *flexure*, *shear* (both one-way and two-way action), *bearing* and *bond* (development length). In these aspects, the design procedures are similar to those for beams and two-way slabs supported on columns. Additional considerations involve the transfer of force from the column/pedestal to the footing, and in cases where horizontal forces are involved, safety against sliding and overturning.

Deflection control is not a consideration in the design of footings which are buried underground (and hence not visible). However, control of crack-width and protection of reinforcement by adequate cover are important serviceability considerations, particularly in aggressive environments. It is considered sufficient to limit the crack-width to 0.3 mm in a majority of footings, and for this the general detailing requirements will serve the purpose of crack-width control.

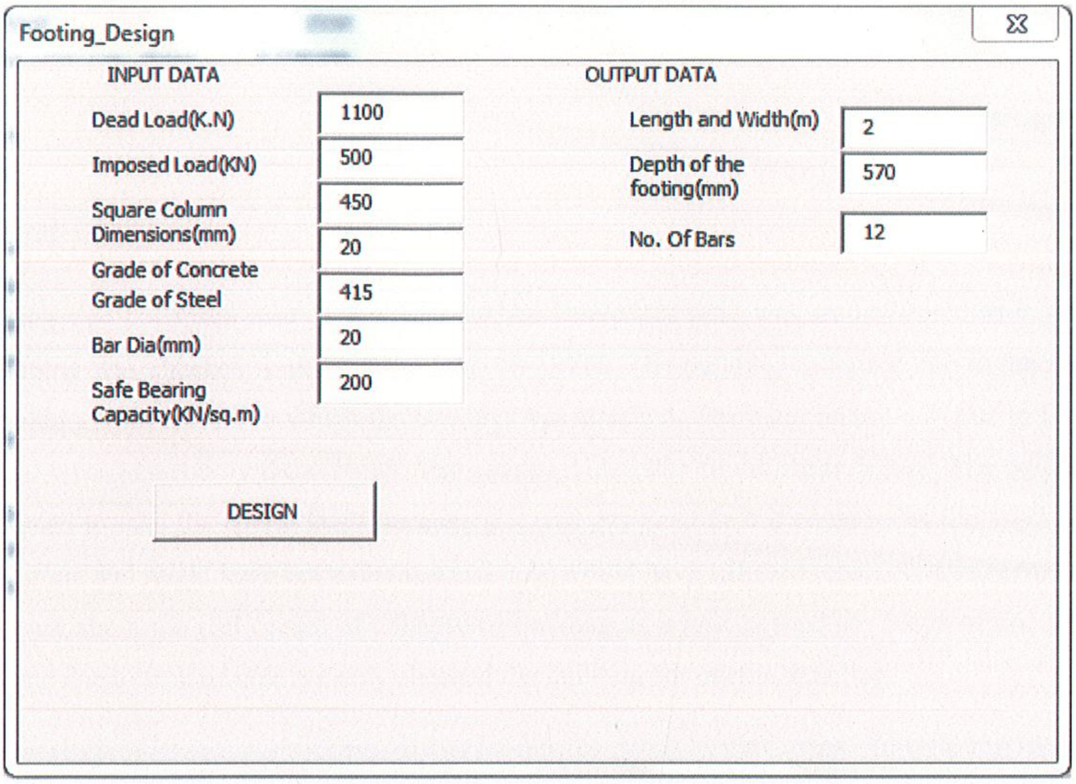

Fig. 4.9, AN EXAMPLE SHOWING THE WORKING OF MODULE DEVELOPED FOR THE DESIGN OF ISOLATED FOOTING

CHAPTER 5

CASE STUDY

5.1*General*

In building that collapsed near Waknaghat-Jaypee University road was studied for ductile detailing. The detailing was checked in accordance with IS-13920. The building collapsed due to the falling of rock boulders from the hill to which the building was attached. The main failure was due to the lateral loads that were induced by these rocks. The column failed due to the shear failure. The building was not designed to take the lateral loads because if it was designed then it could resist the loads imposed by rock strata and might have not collapsed and only would have suffered reparable damages. Since we are not sure about the real causes of collapse of building as it was beyond the scope of our final year project and due to lack of time we only checked the building for ductile detailing.

Fig. 5.1, COLLAPSED BUILDING NEAR WAKNAGHAT

- The beam-column joints were not provided with the anchorage length as they de-attached from the frame structure without even taking the load and the failure of column was not due to shear and neither it was buckling failure. This shows that the external beam-column joint was not designed to take the lateral forces and thus it contributed to the collapse of building. The fallen columns have been shown in the photo and beam column joint recommended by IS code is also shown.

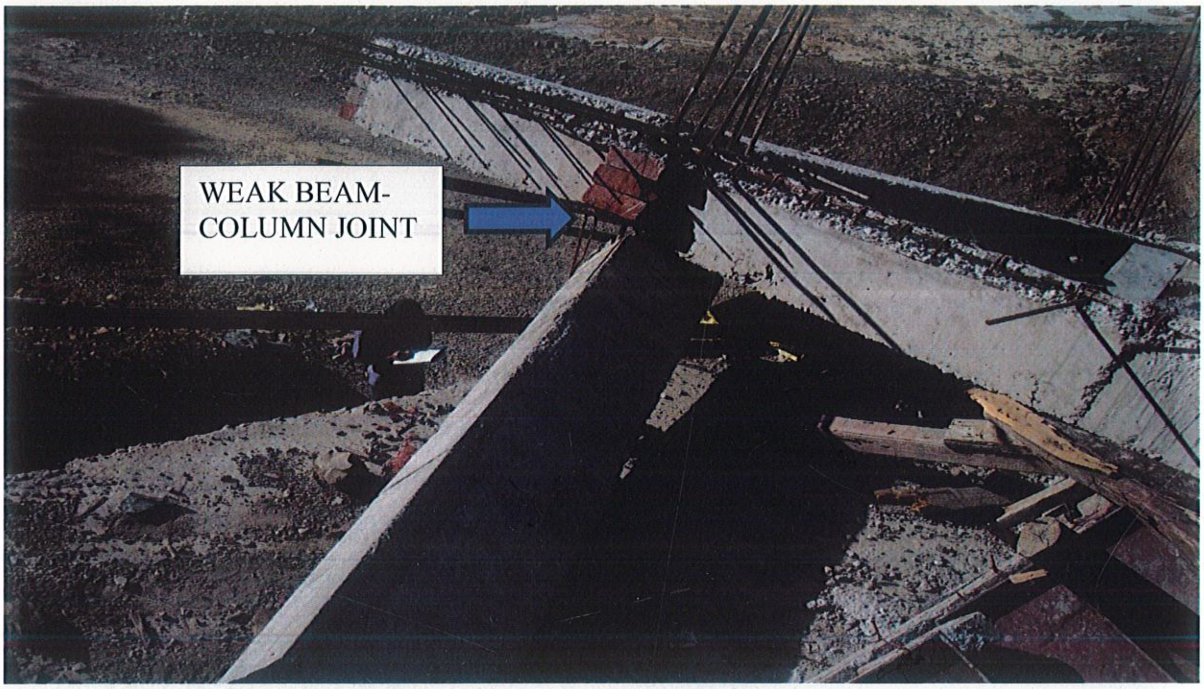

Fig. 5.2, PHOTOGRAPH SHOWING WEAK EXTERNAL BEM-COLUMN JOINT

- The span length between two columns is more than 6m and the minimum dimension of the column was 250 mm which is less than the recommended value.
- The spacing of the shear reinforcement was not in accordance with the IS 13920:1993. The spacing at the centre of beam and at the ends of beam is same.

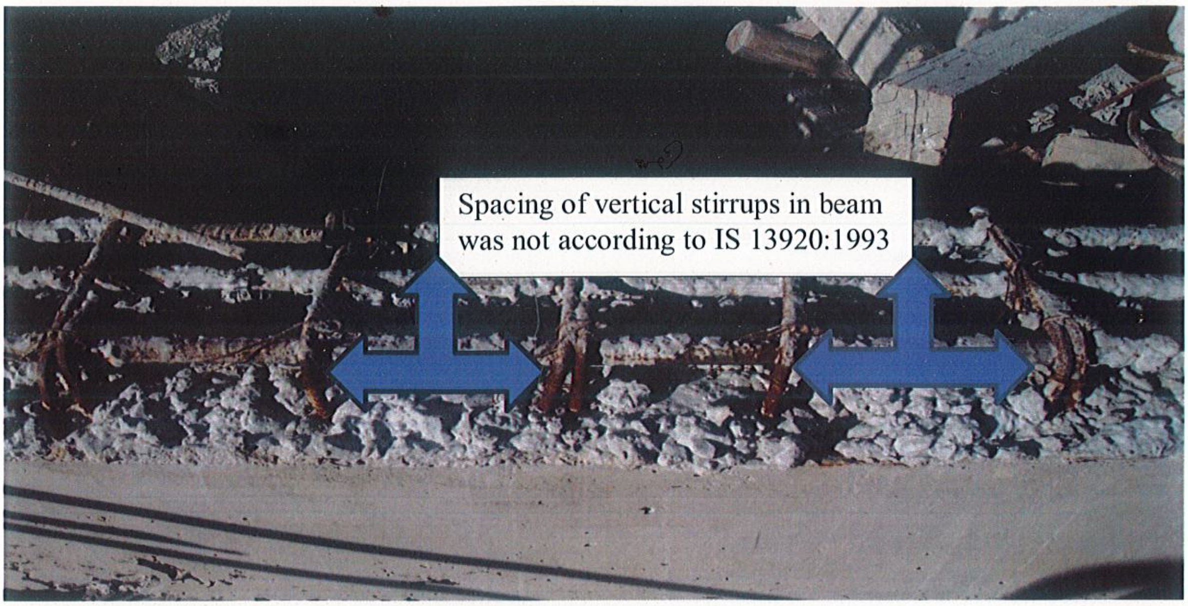

Fig. 5.3, IMPROPER SPACING

- The presence of very short column at the ground floor lead to the sudden failure of the building as they were not able to handle the relative displacement caused by the above floors due to the lateral force applied by the falling rock strata. The steel of the column yielded and might have led to the collapse of building. The short columns are stiffer and therefore fail early as compared to the long columns.

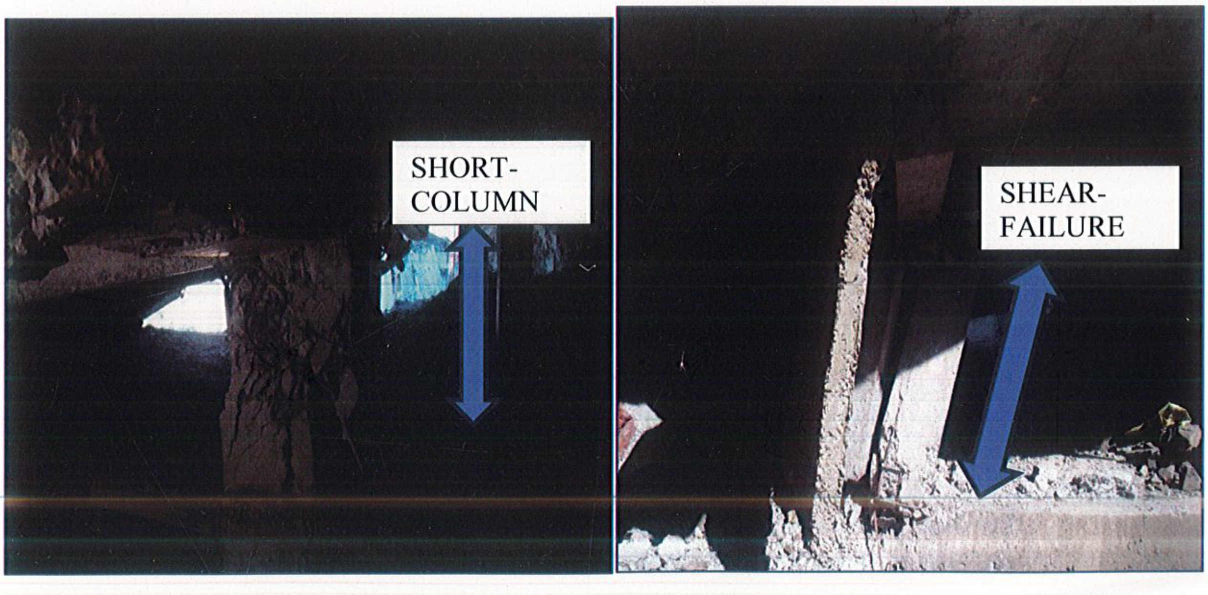

Fig.5.4, THE FAILURE OF SHORT COLUMNS IN GROUND STOREY

- The ultimate reason for the failure of building was the absence of retaining wall which can have saved the building from collapse. But the performance of building during earthquake might have been very bad as the building was stiff and it was not designed to resist the lateral forces.

CONCLUSIONS

- There is a lack of awareness in the earthquake disaster mitigations. Avoiding non-engineered structures with unskilled labour even in unimportant temporary constructions can help a great way.
- Proper Layout and distribution of stiffness of structural elements for the entire building are essential to improve the dynamic responses.
- State-wide awareness programmes have to be conducted by fully exploiting the advancement in the information technology.
- Urgent steps are required to be taken to make the codal provisions regarding earthquake resistant construction undebatable.
- The builders and constructors should adopt the codal provisions in all the future construction, as prevention is better than cure. On the light of avoiding the risk, this may not be an impossible task as earthquake resistant measures in building involves only 2%-6% additional cost depending on the type of building.

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