“DESIGN OF A MULTISTOREY BUILDING IN AN EARTHQUAKE PRONE AREA USING STAAD.PRO V8i”

A PROJECT

Submitted in partial fulfilment of the requirements for the award of the degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

Prof. Ashok Kumar Gupta

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT SOLAN – 173 234

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CERTIFICATE

This is to certify that the work which is being presented in the project title **"Design of a Multistorey Building In An Earthquake Prone Area using STAAD.Pro"** in partial fulfilment of the requirements for the award of the degree of Bachelor of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by Roopak Jain(121601) and Saumya Joshi(121602) during a period from July 2015 to May 2016 under the supervision of Prof. Dr. Ashok Kumar Gupta, Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

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

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Content

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We would like to express our special thanks of gratitude to our teacher Dr. Ashok Kumar Gupta (Prof and Head of Department) who gave us the golden opportunity to do this wonderful project on the topic 'Design of a Multistorey building in an earthquake prone area using STAAD.Pro V8i', which also helped us in doing a lot of Research and we came to know about so many new things. We are also very thankful to Mr. Abhilash Shukla (Assistant Professor) who helped us in conceptualizing this project and during the entire process of completionof project.We would also like to thank our friends who helped us a lot in finalizing this project within the limited time frame.

Thanking you.

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List of Abbreviations and Symbols

Abbreviations and Symbols	Description
RCC	Reinforced Cement Concrete
RC	Reinforced Concrete
cum	Cubic metre
GUI	Graphical User Interface
FRC	Fibre Reinforced Concrete
ECC	Engineered Cementitious Composites
G.F	Ground Floor
DL	Dead Load
LL	Live Load
EL	Earthquake Load
T	Time Period
Z	Zone Factor
BIS	Bureau of Indian Standards
HYSD	High Yielding Strength Deformed Bars

ABSTRACT

The principle objective of this project is to analyse and design a multi-storeyed building [G + 5 (3 dimensional frame)] using STAAD Pro. The design involves load calculations manually and analyzing the whole structure by STAAD Pro. The design methods used in STAAD-Pro analysis are Limit State Design conforming to Indian Standard Code of Practice. STAAD.Pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.Pro is the professional's choice.

STAAD.Pro has a very interactive user interface which allows the users to draw the frame and input the load values and dimensions. Then according to the specified criteria assigned it analyses the structure and designs the members with reinforcement details for RCC frames.

We considered a 3-D RCC frame with the dimensions of 3 bays @7.5m in x-axis and 3 bays @7.5m in z-axis. The y-axis consisted of G + 5 floors.The building will be used for exhibitions, as an art gallery or show room, etc., so that there are no walls inside the building. Only external walls 230 mm thick with 12 mm plaster on both sides are considered.

CHAPTER 1: INTRODUCTION TO EARTHQUAKES AND EARTHQUAKE RESISTANT STRUCTURES

1.1 Earthquake

An **earthquake** (also known as a **quake**, **tremor** or **temblor**) is the perceptible shaking of the surface of the Earth, which can be violent enough to destroy major buildings and kill thousands of people. The severity of the shaking can range from barely felt to violent enough to toss people around. Earthquakes have destroyed whole cities. They result from the sudden release of energy in the Earth's crust that creates seismic waves. The **seismicity** or **seismic activity** of an area refers to the frequency, type and size of earthquakes experienced over a period of time.

Earthquakes are measured using observations from seismometers and seismographs. Earthquakes are measured using observations from seismometers. The moment magnitude is the most common scale on which earthquakes larger than approximately 5 are reported for the entire globe. The more numerous earthquakes smaller than magnitude 5 reported by national seismological observatories are measured mostly on the local magnitude scale, also referred to as the Richter magnitude scale. These two scales are numerically similar over their range of validity. Magnitude 3 or lower earthquakes are mostly almost imperceptible or weak and magnitude 7 and over, potentially cause serious damage over larger areas, depending on their depth. The largest earthquakes in historic times have been of magnitude slightly over 9, although there is no limit to the possible magnitude. The most recent large earthquake of magnitude 9.0 or larger was a 9.0 magnitude earthquake in Japan in 2011 (as of March 2014), and it was the largest Japanese earthquake since records began. Intensity of shaking is measured on the modified Mercalli scale. The shallower an earthquake, the more damage to structures it causes, all else being equal.

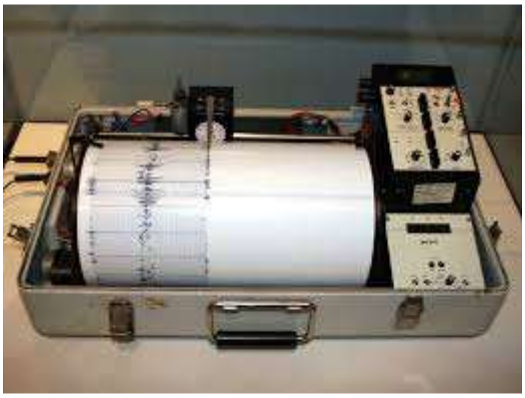


Fig 1: Modern Day Seismograph

At the Earth's surface, earthquakes manifest themselves by shaking and sometimes displacement of the ground. When the epicentre of a large earthquake is located offshore, the seabed may be displaced sufficiently to cause a tsunami. Earthquakes can also trigger landslides, and occasionally volcanic activity.

1.2 Tectonic Plate

It is a massive irregularly shaped mass of solid rock generally composed of both continental and oceanic lithosphere. The plate size can vary from a few hundred kilometres to thousands of kilometres across. The pacific and the Antarctic plates are among the largest. Their interaction causes continental drift, volcanoes and earthquakes etc.

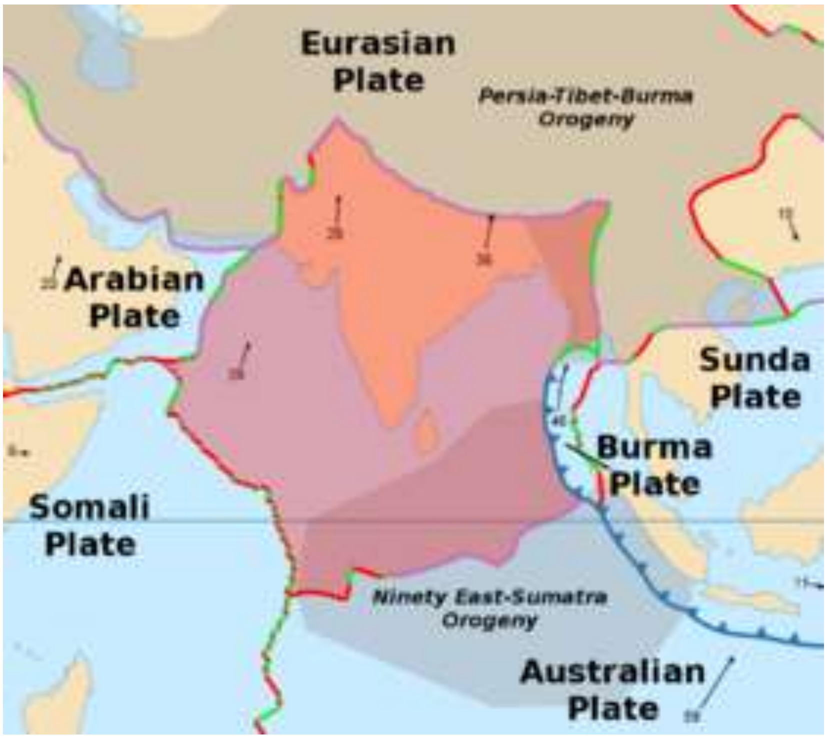


Fig 2: Tectonic Plates around India

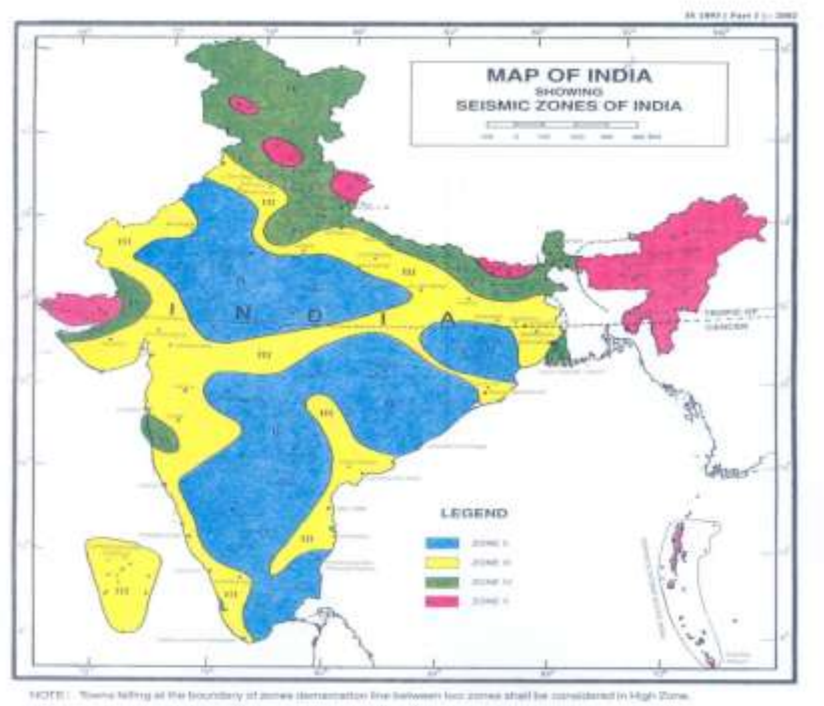


Fig 3: Seismic zones in India according to IS 1893:2002 (Part 1)

1.3 History of Major Earthquakes in India

Table 1: Major earthquakes in India

S. No.	Place	Date	Magnitude
1.	Arunachal Pradesh	August 15,1950	8.6
2.	Uttarakhand	October 20,1991	7.0
3.	Gujarat	January 26,2001	7.6/7.7
4.	Andaman Islands	December 26,2004	9.1
5.	Kashmir	October 8,2005	7.6
6.	Northern and North-East India	April 25,2015	7.8

1.4 Earthquake Resistant Structures

Earthquake-resistant structures are structures designed to withstand earthquakes. While no structure can be entirely immune to damage from earthquakes, the goal of earthquake-resistant construction is to erect structures that fare better during seismic activity than their conventional counterparts.

According to building codes, earthquake-resistant structures are intended to withstand the largest earthquake of a certain probability that is likely to occur at their location. This means the loss of life should be minimized by preventing collapse of the buildings for rare earthquakes while the loss of functionality should be limited for more frequent ones.

Currently, there are several design philosophies in earthquake engineering, making use of experimental results, computer simulations and observations from past earthquakes to offer the required performance for the seismic threat at the site of interest. These range from appropriately sizing the structure to be strong and ductile enough to survive the shaking with an acceptable damage, to equipping it with base isolation or using structural vibration control technologies to minimize any forces and deformations. While the former is the method typically applied in most earthquake-resistant structures, important facilities, landmarks and cultural heritage buildings use the more advanced (and expensive) techniques of isolation or control to survive strong shaking with minimal damage.

1.5WORKING WITH STAAD.Pro V8i

Our project involves analysis and design of multi-storeyed [G + 5] using a very popular designing software STAAD Pro. We have chosen STAAD Pro because of its following advantages:

- Easy to use interface,
-Conformation with the Indian Standard Codes,
-Versatile nature of solving any type of problem,
-Accuracy of the solution.

STAAD.Pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.Pro is the professional's choice for steel, concrete, timber, aluminium and cold-formed steel design of low and high-rise buildings, culverts, petrochemical plants, tunnels, bridges, piles and much more.

STAAD.Pro consists of the following:

- The STAAD.Pro Graphical User Interface: It is used to generate the model, which can then be analyzed using the STAAD engine. After analysis and design is completed, the GUI can also be used to view the results graphically.
- The STAAD analysis and design engine: It is a general-purpose calculation engine for structural analysis and integrated Steel, Concrete, Timber and Aluminium design.

To start with we have solved some sample problems using STAAD Pro and checked the accuracy of the results with manual calculations. The results were to satisfaction and were accurate. In the initial phase of our project we have done calculations regarding loadings on buildings and also considered seismic and wind loads. Structural analysis comprises the set of physical laws and mathematics required to study and predicts the behaviour of structures.

Structural analysis can be viewed more abstractly as a method to drive the engineering design process or prove the soundness of a design without a dependence on directly testing it. To perform an accurate analysis a structural engineer must determine such information as structural loads, geometry, support conditions, and materials properties. The results of such an analysis typically include support reactions, stresses and displacements. This information is then compared to criteria that indicate the conditions of failure.

Advanced structural analysis may examine dynamic response, stability and non-linear behaviour. The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance to the effects of seismic and wind. Structure and structural elements shall normally be designed by Limit State Method. Account should be taken of accepted theories, experiment and experience and the need to design for durability. Design, including design for durability, construction and use in service should be considered as a whole.

The realization of design objectives requires compliance with clearly defined standards for materials, production, workmanship and also maintenance and use of structure in service. The design of the building is dependent upon the minimum requirements as prescribed in the Indian Standard Codes. The minimum requirements pertaining to the structural safety of buildings are being covered by way of laying down minimum design loads which have to be assumed for dead loads, imposed loads, and other external loads, the structure would be required to bear. Strict conformity to loading standards recommended in this code, it is hoped, will not only ensure the structural safety of the buildings which are being designed

CHAPTER 2: LITERATURE REVIEW

- "**A Study on Earthquake Resistant Construction Techniques"**

  (Mohammad Adil Dar, Prof (Dr) A.R. Dar , Asim Qureshi , Jayalakshmi Raju

Apart from the modern techniques which are well documented in the codes of practice, there are some other old traditional earthquake resistant techniques which have proved to be effective for resisting earthquake loading and are also cost effective with easy constructability.

In addition to the main earthquake design code 1893 the BIS(Bureau of Indian Standards)has published other relevant earthquake design codes for earthquake resistant construction Masonry structures (IS-13828 1993)

- Horizontal bands should be provided at plinth ,lintel and roof levels as per code
- Providing vertical reinforcement at important locations such as corners, internal and external wall junctions as per code.
- Grade of mortar should be as per codes specified for different earthquake zones.
- Irregular shapes should be avoided both in plan and vertical configuration.
- Quality assurance and proper workmanship must be ensured at all cost without any compromise.

In RCC framed structures (IS-13920)

- In RCC framed structures the spacing of lateral ties should be kept closer as per the code
- The hook in the ties should be at 135 degree instead of 90 degree for better anchoragement.
- The arrangement of lateral ties in the columns should be as per code and must be continued through the joint as well.
- Whenever laps are to be provided, the lateral ties (stirrups for beams) should be at closer spacing as per code.

- **"Earthquake Analysis of High Rise Building with and Without In filled Walls"**

  (Wakchaure M.R, Ped S. P)

The effect of masonry infill panel on the response of RC frames subjected to seismic action is widely recognized and has been subject of numerous experimental investigations, while several attempts to model it analytically have been reported. In analytically analysis infill walls are modelled as equivalent strut approach there are various formulae derived by research scholars and scientist for width of strut and modelling. Infill behaves like compression strut between column and beam and compression forces are transferred from one node to another. In this study the effect of masonry walls on high rise building is studied. Linear dynamic analysis on high

rise building with different arrangement is carried out. For the analysis G+9 R.C.C. framed building is modelled. Earthquake time history is applied to the models. The width of strut is calculated by using equivalent strut method. Various cases of analysis are taken. All analysis is carried out by software ETABS. Base shear, storey displacement, story drift is calculated and compared for all models. The results show that infill walls reduce displacements, time period and increases base shear. So it is essential to consider the effect of masonry infill for the seismic evaluation of moment resisting reinforced concrete frame.
The response of RC frames subjected to seismic action is widely recognized and has been subject of numerous experimental investigations, while several attempts to model it analytically have been reported.

- **"Effect of foundation compliance on earthquake stresses in multistory buildings"-**(Merritt,R.G. and Housner, G. W. (1954 Bulletin of the Seismological Society of America, 44 (4). pp. 551-569. ISSN 0037)
  This paper shows the quantitative effect that foundation compliance has on the maximum base shear force and the fundamental period of vibration in typical tall buildings subjected to strong-motion earthquakes.

- **"Seismic Vulnerability Of Existing Rc Buildings In India"-2004**
  (Prathibha S and A Meher Prasad)
  The need for evaluating the seismic adequacy of the existing structures has come into focus following the damage and collapse of numerous concrete structures during recent earthquakes. In order to assess the vulnerability, a simplified procedure for evaluation is highly in need for a country like India which is prone to earthquakes. It is important to estimate the response of buildings under earthquakes from the viewpoint of life reservation and risk management.
  In a seismically active region like India, there is potential risk for existing RC buildings. The need for a simple yet reliable evaluation of existing buildings is of growing concern to the practicing community. While analytical tools for nonlinear static analysis exist, the real issue is whether the modelling of certain Non-ductile detailing is properly accounted for in the evaluations. The purpose of this study is to provide a simple rational procedure to analyze existing RC buildings that were designed for gravity loads. The procedure allows modeling of non ductile detailing in an implicit manner so that existing analytical tools can be used to carry out the required seismic evaluation. The analysis provides an insight into the behaviour of the components and the failure mechanism of the structure as a whole. The evaluation procedure is applied to typical four storey RC MRF building that reveals the inherent deficiencies as compared to current earthquake resistant design requirements in India.
  In this paper a rational procedure for seismic evaluation of Indian RC MRF buildings is presented with a detailed pushover analysis of a typical four storey building. The inadequacies in detailing are incorporated in the model in the form of moment

rotation properties for the structural elements. This procedure gives a quick estimate of the base shear and the desirable performance of the building in its existing condition. Also this methodology is efficient in determining the deficient members and the performance of the building as a whole. The performance of the building is finally checked for code compliance and for the probable failure mechanisms. This evaluation is a prerequisite for the retrofit of the existing RC MRF buildings in India.

- **“Seismic Response of RC Frame Buildings with Soft First Storeys”-1997**
  (Jaswant N. Arlekar, Sudhir K. Jain and C.V.R. Murty)
  Open first storey is a typical feature in the modern multistorey constructions in urban India. Such features are highly undesirable in buildings built in seismically active areas; this has been verified in numerous experiences of strong shaking during the past earthquakes. This paper highlights the importance of explicitly recognizing the presence of the open first storey in the analysis of the building. The error involved in modeling such buildings as complete bare frames, neglecting the presence of infills in the upper storeys, is brought out through the study of an example building with different analytical models.This paper argues for immediate measures to prevent the indiscriminate use of soft first storeys in buildings, which are designed without regard to the increased displacement, ductility and force demands in the first storey columns. Alternate measures, involving stiffness balance of the open first storey and the storey above, are proposed to reduce the irregularity introduced by the open first storey. The effect of soil flexibility on the above is also discussed in this paper.

- **“Multi-Objective Optimal Seismic Design Of Buildings Using Advanced Engineering Materials”-2011**
  (Bora Gencturk and Amr S. Elnashai)
  The goal of this study is to develop a framework that concurrently addresses the societal level objectives of safety, economy and sustainability using consistent tools at every component of the analysis. To this end, a high-performance material; namely, engineered cementitious composites (ECC) is utilized. ECC is classified under the general class of fiber-reinforced concrete (FRC); however, ECC is superior to conventional FRC in many aspects, but most importantly in its properties of energy absorption, shear resistance and damage tolerance, all of which are utilized in the proposed procedure. The behavior of ECC is characterized through an experimental program at the small-scale (scale factor equal to 1/8). ECC mixtures with different cost and sustainability indices are considered. It is seen that all ECC mixtures outperform concrete to different extents of stiffness, strength, ductility and energy absorption under cyclic loading conditions. Under simulated earthquake motion, ECC shows significant damage tolerance resulting from increased shear and spalling resistance and reduced inter-story drifts.

CHAPTER 3: ANALYSIS OF A G+5 STOREY BUILDING USING STAAD.Pro

3.1 General Details:

- A 3-D RCC frame with the dimensions of 3 bays @7.5m in x-axis and 3 bays @7.5m in z-axis. The y-axis consisted of G + 5 floors.
- The building will be used for exhibitions, as an art gallery or show room, etc., so that there are no walls inside the building. Only external walls 230 mm thick with 12 mm plaster on both sides are considered.

3.2 Design Data Considered:

Table 2:Design data considered

Live Load	4.0 kN/m^2 at typical floor 1.5 kN/m^2 at terrace
Floor Finish	1 kN/m^2
Water Proofing	2 kN/m^2
Terrace Finish	1 kN/m^2
Location	Vadodara City (Seismic zone III)
Wind Loads	As per IS 875-Not designed for wind loads as earthquake loads exceeds it
Earthquake load	As per IS-1893 (Part 1)-2002
Soil Type	Type II, Medium as per IS 1893
Allowable Bearing Pressure	300 kN/m^2
Storey height	Typical floor: 5m G.F :4.1m Plinth:1.1m
Floors	G+5 upper floors
Walls	230mm thick brick masonry walls only at periphery and 12mm plaster on both sides

3.3 Material Properties

Table 3:Material properties considered

Concrete: All components unless specified in design: M25 grade $Ec=5000(f_{ck})^{0.5}$ N/mm^2 $=5000(f_{ck})^{0.5}$MN/m^2 =25000MN/m^2

Steel: HYSD reinforcement of grade Fe 415 conforming to IS 1786 will be used throughout

3.4 Plan of the Project

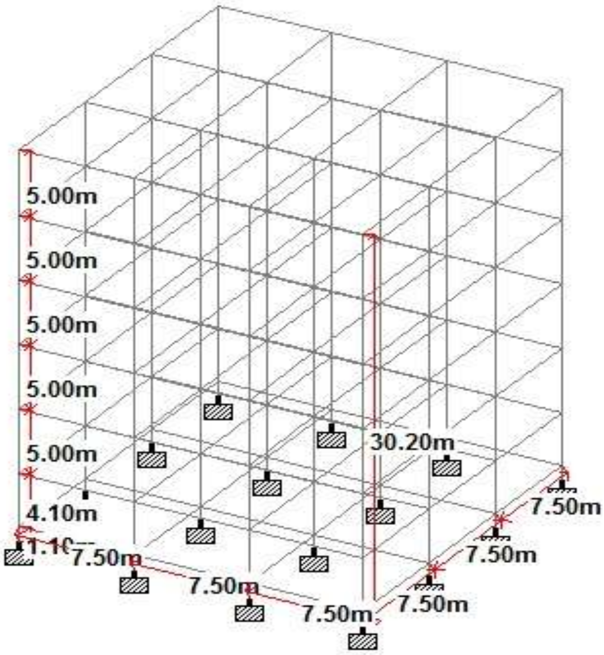


Fig 4:3-D View

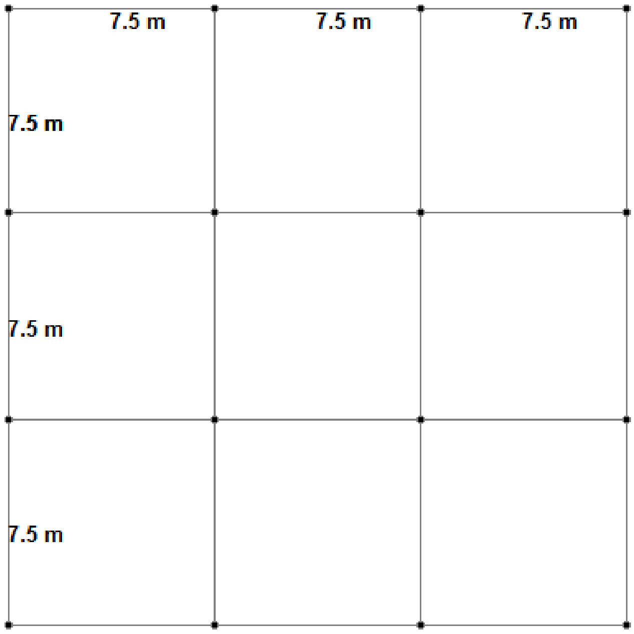


Fig 5:Plan

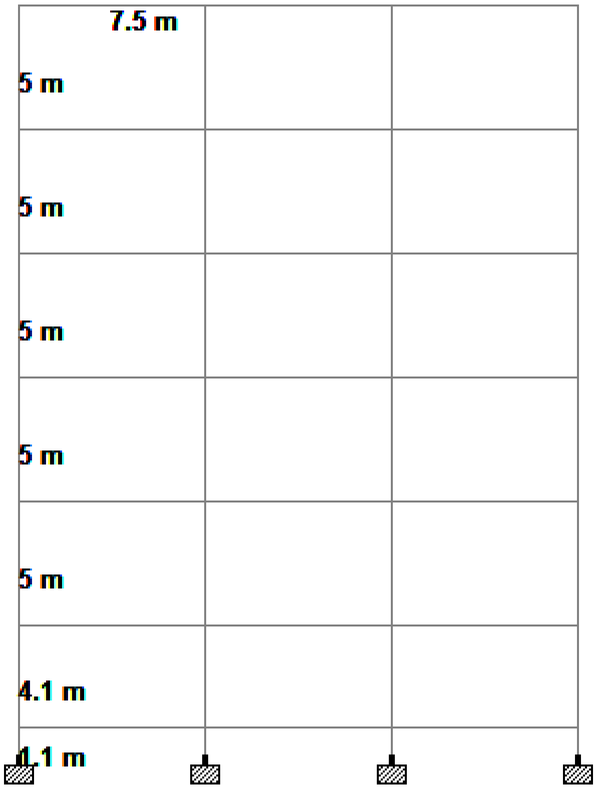


Fig 6:Elevation

Area: 22.5*22.5=506.25m^2

3.5 Loads Considered:

- **Dead Loads**: All permanent constructions of the structure form the dead loads. The dead load comprises of the weights of walls, partitions floor finishes, false ceilings, false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights. The unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m^3 and 25 kN/m^3 respectively.
  Dead load calculations will be done following IS 875(Part 1)-1987

- **Imposed Loads**: Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo. Load calculations will be done following IS 875(Part 2)-1987

- **Seismic Loads**: Seismic load calculations will be done following IS 1893(Part 1)-2000. The seismic weights are calculated in a manner similar to gravity loads. The weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. Following reduced live loads are used for analysis:
  Zero on terrace, and 50% on other floors [IS: 1893 (Part 1): 2002, Clause 7.4)

3.6 Loading Diagrams

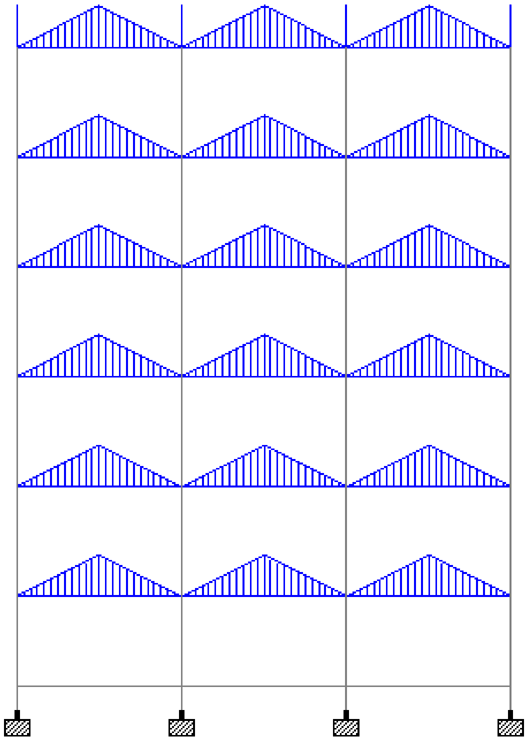


Fig 7: Loading diagram for Slab Self Weight

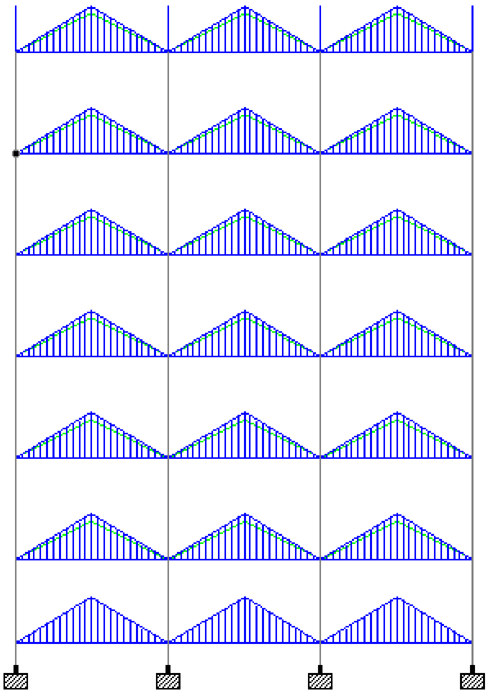


Fig 8: Loading diagram for Superimposed dead load

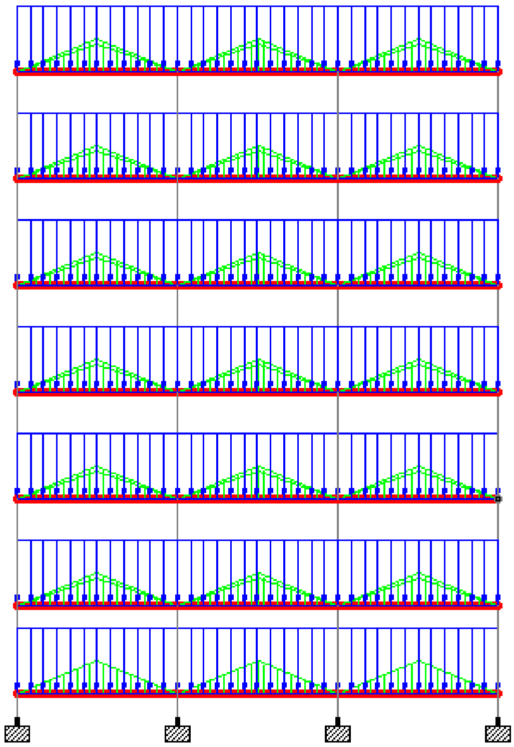


Fig 9:Loading Diagram for Total Dead Load

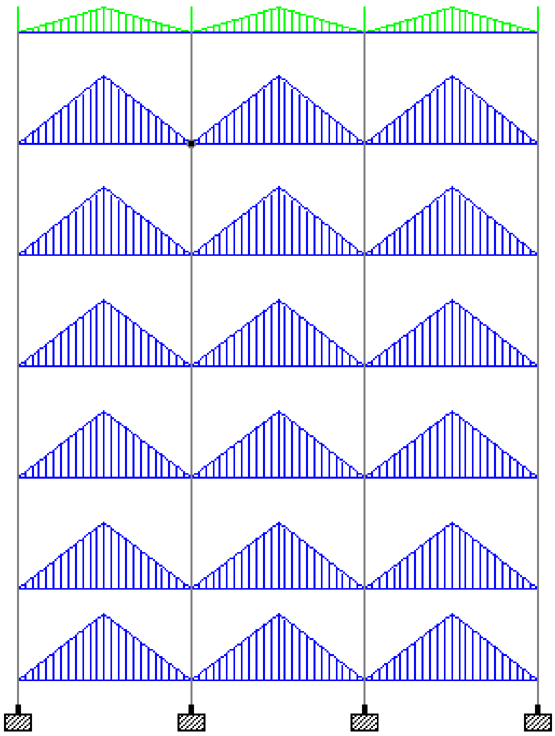


Fig 10:Loading Diagram for Live Load

3.7 Analysis of Frame:

Gravity Load calculations

Unit load calculations

Assumed sizes of beam and column sections are:

Columns: 500 x 500 at all typical floors
Area, $A = 0.25\ m^2$, $I = 0.005208\ m^4$

Columns: 600 x 600 below ground level
Area, $A = 0.36\ m^2$, $I = 0.0108\ m^4$

Main beams: 300 x 600 at all floors
Area, $A = 0.18\ m^2$, $I = 0.0054\ m^4$

Member self- weights:

Columns (500 x 500)
0.50 x 0.50 x 25 = 6.3 kN/m

Columns (600 x 600)
0.60 x 0.60 x 25 = 9.0 kN/m

Main beams (300 x 600)
0.30 x 0.60 x 25 = 4.5 kN/m

Slab (100 mm thick)
0.1 x 25 = 2.5 kN/m^2

Brick wall (230 mm thick)
0.23 x 19 (wall) +2 x 0.012 x 20 (plaster)= 4.9 kN/m^2

Floor wall (height 4.4 m)
4.4 x 4.9 = 21.6 kN/m

Ground floor wall (height 3.5 m)
3.5 x 4.9 = 17.2 kN/m

Ground floor wall (height 0.5 m)
0.5 x 4.9 = 2.45 kN/m

Terrace parapet (height 1.0 m)
1.0 x 4.9 = 4.9 kN/m

Slab load calculations

Table 4: Slab Load Calculations

Component	**Terrace (DL + LL)**	**Typical (DL + LL)**
Self (100 mm thick)	2.5 + 0.0	2.5 + 0.0
Water proofing	2.0 + 0.0	0.0 + 0.0
Floor finish	1.0 + 0.0	1.0 + 0.0
Live load	0.0 + 1.5	0.0 + 4.0

Seismic Weight Calculations:

The seismic weights are calculated in a manner similar to gravity loads. The weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. Following reduced live loads are used for analysis: Zero on terrace, and 50% on other floors [IS: 1893 (Part 1): 2002, Clause 7.4)

Table 5:Seismic Weight Calculation of Terrace

	(in kN)	DL + LL
From slab	22.5 x 22.5 (5.5+0)	2 784 + 0
Parapet	4 x 22.5 (4.9 + 0)	441 + 0
Walls	0.5 x 4 x 22.5 x (21.6 + 0)	972 + 0
Main Beams	8 x 22.5 x (4.5 + 0)	810+ 0
Columns	0.5 x 5 x 16 x (6.3 + 0)	252 + 0
Total	5259 + 0	= 5259 kN

Table 6:Seismic weight calculation of Middle Storeys

	(in kN)	DL + LL
From slab	22.5 x 22.5 x (3.5 + 0.5 x 4)	1772 + 1013
Walls	4 x 22.5 x (21.6 + 0)	1944 + 0
Main Beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	16 x 5 x (6.3 + 0)	504+0
Total	5030 + 1013	= 5030+1013=6043 kN

Table 7:Seismic Weight Calculation of Ground storey

	(in kN)	DL + LL
From slab	22.5 x 22.5 x (3.5 + 0.5 x 4)	1772 + 1013
Main Beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	16 x 0.5 x (5 + 4.1) x (6.3 + 0)	459 + 0
Total	3041 + 1013	=3041+1013=4054kN

Table 8:Seismic weight Calculation at Plinth

	(in kN)	DL + LL
Main Beams	8 x 22.5 x (4.5 + 0)	810 + 0
Columns	16 x 0.5 x 4.1 x (6.3 + 0) + 16 x .5 x 1.1 x (9 + 0)	285 + 0
Total	1095 + 0	= 1095+0=1095kN

Seismic weight of the entire building = 5259 + 4 x 6043 + 4054 + 1095 = 34580 kN

The seismic weight of the floor is the lumped weight, which acts at the respective floor level at the centre of mass of the floor.

Design Seismic Load

The infill walls in upper floors may contain large openings, although the solid walls are considered in load calculations. Therefore, fundamental time period T is obtained by using the following formula:

- $T\text{a} = 0.075\ h^{0.75}$ [IS 1893 (Part 1):2002, Clause 7.6.1]
  $= 0.075 \times (30.2)^{0.75}$
  $= 0.97$ sec.
- Zone factor, Z = 0.16 for Zone III
- IS: 1893 (Part 1):2002, Table 2
- Importance factor, I = 1.5 (public building)
- Medium soil site and 5% damping

Table 9: Distribution of Total Horizontal Load to Different Floor Levels

Storey	**W_i(kN)**	**hi(m)**	**$W_ih_i^2*10^{-3}$**	**$Qi=W_ih_i^2/\Sigma W_ih_i^2 *V_b$**	**Vi(kN)**
7	5259	30.2	4796.41836	421.6530582	421.65
6	6043	25.2	3837.54672	337.3595438	759.24
5	6043	20.2	2465.78572	216.7677442	976
4	6043	15.2	1396.17472	122.7380149	1098.73
3	6043	10.2	628.71372	55.27035611	1154
2	4054	5.2	109.62016	9.636731452	1163
1	1095	1.1	1.32495	0.116476635	1163.54
Total			**13235.58435**	**1163.541925**	

$S_a/g=1.36/.97=1.402$

$$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$$

$A_h=0.0336$

Base shear, $V_B = A_h\ W=1163.54kN$

VARIOUS LOAD COMBINATIONS:

- ❖ As per IS 1893 (Part 1): 2002 Clause no. 6.3.1.2, the following load cases have to be considered for analysis:
  1.5 (DL + IL)
  1.2 (DL + IL ± EL)
  1.5 (DL ± EL)
  0.9 DL ± 1.5 EL

- ❖ Thus, ±EL above implies 8 cases, and in all, 25 cases as per Table 1 must be considered.
  EXTP: EQ load in X direction with torsion positive
  EXTN: EQ load in X direction with torsion negative
  EZTP: EQ load in Z direction with torsion positive
  EZTN: EQ load in Z direction with torsion negative.

Table 10: Load Combinations used for design

No.	Load combination
1	1.5 (DL + IL)
2	1.2 (DL + IL + EXTP)
3	1.2 (DL + IL + EXTN)
4	1.2 (DL + IL – EXTP)
5	1.2 (DL + IL – EXTN)
6	1.2 (DL + IL + EZTP)
7	1.2 (DL + IL + EZTN)
8	1.2 (DL + IL – EZTP)
9	1.2 (DL + IL – EZTN)
10	1.5 (DL + EXTP)
11	1.5 (DL + EXTN)
12	1.5 (DL – EXTP)
13	1.5 (DL – EXTN)
14	1.5 (DL + EZTP)
15	1.5 (DL + EZTN)
16	1.5 (DL – EZTP)
17	1.5 (DL – EZTN)
18	0.9 DL + 1.5 EXTP
19	0.9 DL + 1.5 EXTN
20	0.9 DL - 1.5 EXTP
21	0.9 DL - 1.5 EXTN
22	0.9 DL + 1.5 EZTP
23	0.9 DL + 1.5 EZTN
24	0.9 DL - 1.5 EZTP
25	0.9 DL - 1.5 EZTN

- **Storey Drift:**

  - As per Clause no. 7.11.1 of IS 1893 (Part 1): 2002, the storey drift in any storey due to specified design lateral force with partial load factor of 1.0, shall not exceed 0.004 times the storey height.

Table 11:**Storey Drift Calculations**

Storey	Deflection(mm)	Storey Drift(mm)
7(Fifth Floor)	97.667	8.689
6(Fourth Floor)	85.978	14.971
5(Third Floor)	71.007	19.495
4(Second Floor)	51.512	19.963
3(First Floor)	31.549	19.769
2(Ground Floor)	11.78	11.287
1(Below Plinth)	0.49	0.49
0 (Footing top)	0	0

- **Stability Indices:**

It is necessary to check the stability indices as per Annex E of IS 456:2000 for all storeys to classify the columns in a given storey as non-sway or sway columns.

As per IS 456:2000, the column is classified as non-sway if Qsi ≤ 0.04, otherwise, it is a sway column. It may be noted that both sway and nonsway columns are unbraced columns.

$$Q_{si} = \frac{\sum P_u \Delta_u}{H_u h_s}$$

Where

Q_{si} = stability index of ith storey

$\sum P_u$ = sum of axial loads on all columns in the ith storey

Δ_u = elastically computed first order lateral deflection

H_u = total lateral force acting within the storey

h_s = height of the storey.

Table 12: Stability Indices of Different Storeys

Storey	Storey Seismic Weight W_i(kN)	Axial Load $\Sigma P_u = \Sigma W_i$	Storey Drift(mm)	Lateral Load $H_u = V_i$ (kN)	H_s (mm)	Q_{si}	Classification
7	5259	5259	8.69	421.65	5000	0.021677083	No Sway
6	6043	11302	14.971	759.24	5000	0.044571477	Sway
5	6043	17345	19.945	976	5000	0.070890579	Sway
4	6043	23388	19.963	1098.73	5000	0.084988058	Sway
3	6043	29431	19.769	1154	5000	0.100835605	Sway
2	4054	33485	11.287	1163	4100	0.079262042	Sway
1	1095	34580	0.49	1163.54	1100	0.013238753	No Sway

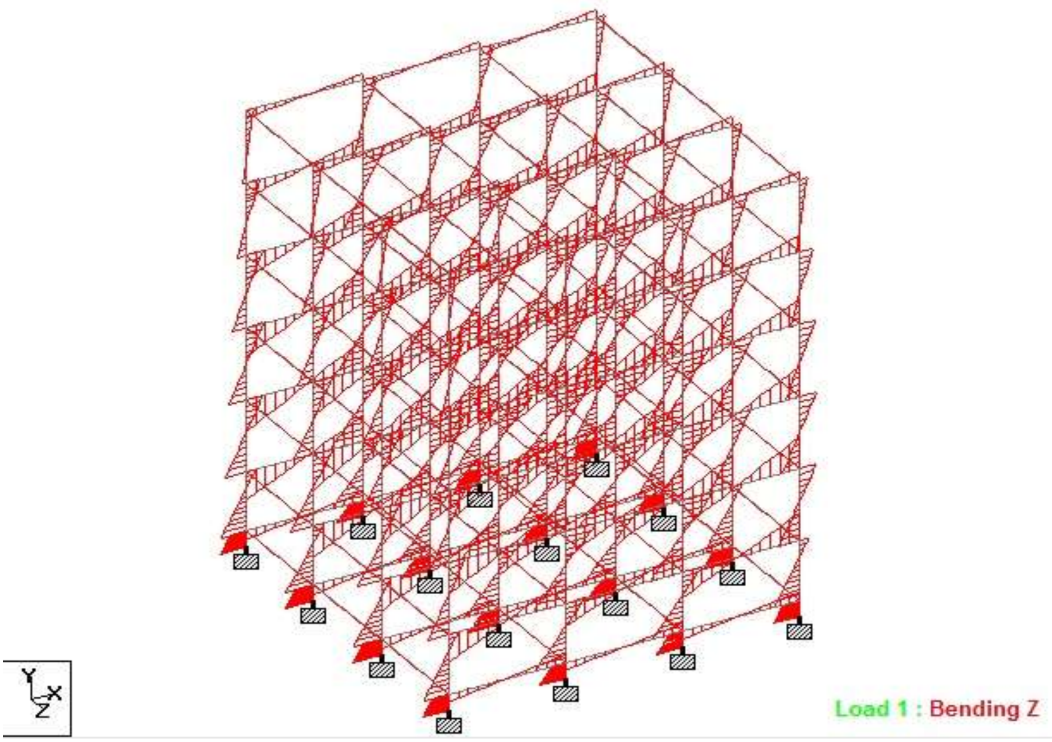


Fig 11:Bending along Z direction due to Seismic load in +X direction

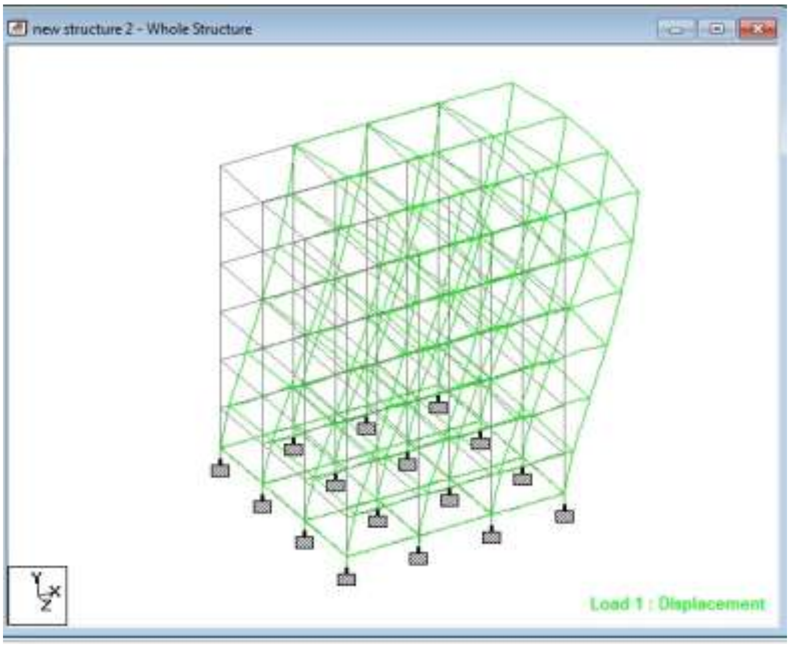


Fig 12:Deflection due to Seismic load in +X direction

CHAPTER 4: DESIGN RESULTS

DETAILING OF FRAME

4.1 Design of Beam

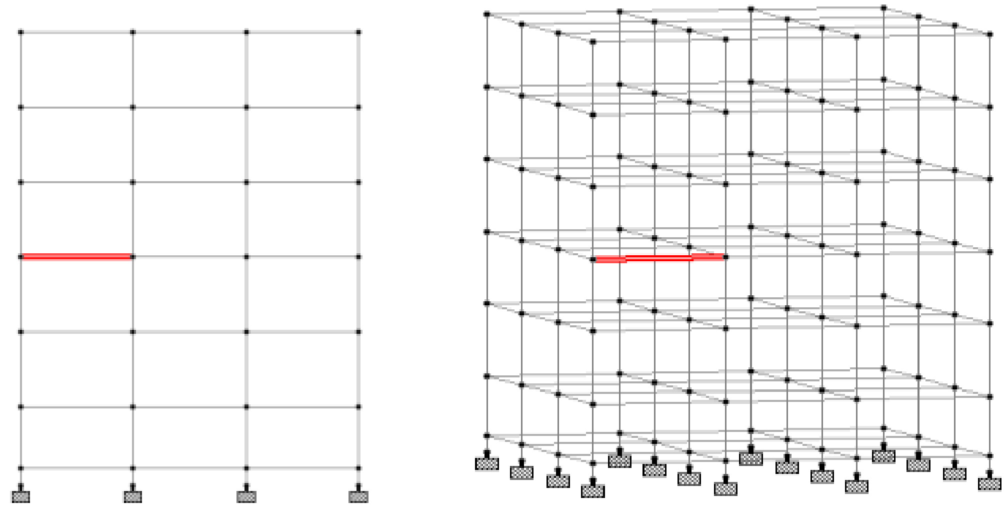


Fig 13:Beam View in Elevation

Left End (A)

A. Check For Axial Stress

- Factored Axial Force= 0.0 kN
- Factored Axial Stress=0.0 MPA
- $0.10f_{ck}$=0.10*25=2.5

Axial Stress< 2.5
Design as Flexural Member (Clause 6.1.1 IS 13920:1993)

B. Check For Member Size

- Width of the beam, B=300mm>200mm

Hence OK (Clause 6.1.3 IS13920:1993)

- Width/Depth= 300/600 =0.5>0.3
  Hence OK (Clause 6.1.2 IS13920:1993)
- Span,L=7.5m=7500mm
  L/D=7500/600=12.5>4
  Hence OK (Clause 6.1.4 IS13920:1993)

C. Check For Limiting Longitudnal Reinforcement

- Effective depth for Moderate Exposure Conditions with 20mm diameter bars in 2 layers on an average
  =600-30-20-(20/2) =532mm

- Minimum reinforcement required= $0.24(f_{ck}{}^{1/2})/f_y$ (Clause 6.2.1(b) IS13920:1993)
  $=(0.24*25^{1/2})/415=0.28\%$
  i.e min reinforcement=$(0.28/100)*300*252=446.8mm^2$
- Maximum reinforcement=2.5% (Clause 6.2.2 IS 13920:1993)
  $0.025*300*532=3990mm^2$

1.Design For Flexure

For Left End

A.Design for hogging moment

M_u =136.47 KN-m

$M_u/bd^2=(136.47*10^6)/(300*532^2)=1.61$

Referring to Table 51 of SP-16:

d'/d=68/532=0.13

A_{st} at top= 1.2%

=0.012*532*300

=1915.5mm^2 >minimum reinforcement

<maximum reinforcement

A_{sc} at bottom=0.003%

But A_{sc} must be atleast 50% of A_{st}

=0.6%

=0.06*300*532
=957.6mm^2

B.Design For Sagging Moment

Mu=574.47KN-m

Designing the beam as a T-Beam

Assuming $x_u<D_f$ and $x_u< x_{u\,max}$

Then, $Mu=0.87f_yA_{st}d(1-(A_{st}f_y/b_fdf_{ck}))$ …………(i)

Where, D_f is Depth of flange=125mm (assumed)

x_u=Depth of Neutral Axis

$x_{u\ max}$=Limiting Value of Neutral Axis

=0.48d

=0.48*532

=255mm

b_w=width of web=300mm

bf=width of flange

=$(L_0/6)+b_w+6d_f$

Or

c/c of beams

=(0.7*7500)/6+300+(6*125) Or 7500mm

=1925mm Or 7500mm

=1925mm (Least Value of Above) (Clause 23.1.2 of IS456:2000)

Substituting above values in the eqn(i) and finding the value of Ast

$574.47*10^6 = 0.87*415*532*A_{st}*(1-(A_{st}*415)/(1925*532*25))$

A_{st}=3151mm^2 at bottom >446mm^2 (minimum reinforcement)

<3990mm^2(maximum reinforcement)

Checking design assumptions:

$x_u=(0.87f_yA_{st})/(0.36f_{ck}b_f)$

=(0.87*415*3151)/(0.36*25*1925)

=65.66mm <D_f ……Hence Ok

<$X_{u\ max}$ …..Hence Ok

Providing 50% of A_{st} as bottom as A_{sc}=0.5*3151=1575mm^2 (Clause 6.2.3 IS13920:1993)

Top Reinforcement=max(1915,957.6)

A_{st}=3151mm^2

Bottom Reinforcement=max(3151,1575

A_{sc}=1575mm^2

For Centre of Beam AB

A.Design for Hogging Moment

Mu=189KN-m

M_u /bd^2=2.2

Referring to Table 51 of SP-16

d'/d=68/532=0.13

A_{st} at top= 1.2%

=0.012*532*300

=1915.5mm^2 >minimum reinforcement

<maximum reinforcement

A_{sc} at bottom=0.003%

But A_{sc} must be atleast 50% of A_{st}

=0.6%

=0.06*300*532

=957.6mm^2

B. No need to design for **sagging moment** as value is negligible

Top Reinforcement=A_{st}=1915mm

Bottom Reinforcement=A_{sc}=957mm^2

For Right End (B)

A.Design For Hogging Moment

M_u =204.27 KN-m

M_u /bd^2=(204.27*10^6)/(300*532^2)

=2.47

Referring to Table 51 of SP-16

d'/d=68/532=0.13

A_{st} at top= 1.2%

=0.012*532*300

=1915.5mm^2 >minimum reinforcement

<maximum reinforcement

A_{sc} at bottom=0.003%

But A_{sc} must be atleast 50% of A_{st}

=0.6%

=0.06*300*532

=957.6mm^2

B.Design For Sagging Moment

Mu=539.29KN-m

Designing the beam as a T-Beam

Assuming x_u<D_f and x_u< $x_{u\ max}$

Then, $Mu=0.87f_yA_{st}d(1-(A_{st}f_y/b_fdf_{ck}))$ …………(i)

Where, D_f is Depth of flange=125mm (assumed)

x_u=Depth of Neutral Axis

$x_{u\ max}$=Limiting Value of Neutral Axis

=0.48d

=0.48*532

=255mm

b_w=width of web=300mm

bf=width of flange

$=(L_0/6)+b_w+6d_f$

Or

c/c of beams

=(0.7*7500)/6+300+(6*125) Or 7500mm

=1925mm Or 7500mm

=1925mm (Least Value of Above) (Clause 23.1.2 of IS456:2000)

Substituting above values in the eqn(i) and finding the value of A_{st}

$539.29*10^6 = 0.87*415*532*A_{st}*(1-(A_{st}*415)/(1925*532*25))$

A_{st}=2948mm^2 at bottom >446mm^2 (minimum reinforcement)

<3990mm^2(maximum reinforcement)

Design Assumptions already checked above

Providing 50% of A_{st} as bottom as A_{sc}=0.5*2948=1474mm^2 (Clause 6.2.3IS13920:1993)

Top Reinforcement=max(1915,1474)

A_{st}=1915mm^2

Bottom Reinforcement=max(957.6,2948)

A_{sc}=2948mm^2

Table 13:Flexural Design of Beam AB(Beam No.-175)

Beam AB (no. 175)	**Top Reinforcement**		
	Left End	Centre	Right End
Hogging moment(kN-m)	-136.47KN-m	-189KN-m	-204.27KN-m
M_u /bd^2	1.61	2.2	2.41
A_{st} at top	1.2%=1915mm^2	1.2%	1.2%
A_{sc} at bottom	0.6%=957.6mm^2	0.6%	0.6%
Bottom Reinforcement			
Sagging Moment(KN-m)	574.47KN-m	-	53929KN-m
A_{st} at bottom	3151mm^2=1.97%	-	2948mm^2
A_{sc} at top	1575mm^2	-	1474mm^2
Summary of Required Reinforcement			
	Top=1915mm^2	Top=1915mm^2	Top=1915mm^2
	Bottom=3151mm^2	Bottom=957.6mm^2	Bottom=2948mm^2

Table 14:Details of Reinforcement

Beam AB(No.175)	**Longitudnal Reinforcement**		
	Left	Centre	Right
Top Reinforcement	3-16Øbars+4-20 Øbars +1-12 Øbars Steel Provided=1972mm^2	3-16Øbars+4-20 Øbars +1-12 Øbars Steel Provided=1972mm^2	3-16Øbars+4-20 Øbars +1-12 Øbars Steel Provided=1972mm^2
Bottom Reinforcement	3-16Ø bars+3-20Ø bars +1-12 Ø bars Steel Provided=3229mm^2	3-16 Ø bars+1-20 Øbars +1-8 Øbar Steel Provided=965mm^2	3-16 Øbars+7-20 Øbars +3-8 Øbars Steel Provided=2945mm^2

2.Design For Shear

Tensile steel provided at the left end=1.2%

Permissible design shear stress of Concrete=Ʈc= 100A/bd= (100*1915)/(300*532)=1.2

From table 19 IS456:2000 for M25 grade concrete

Ʈc=0.68MPa

Design shear strength= Ʈcbd=(0.68*532*300)/1000=108.528KN

Shear Force Due To Plastic Hinge Formation

As per clause 6.3.3 of IS13920:1993

$V_{sway\ to\ right}=\pm1.4(M_u^{As}+M_u^{Bh})/L$

$V_{sway\ to\ left}=\pm1.4(M_u^{Ah}+M_u^{Bs})/L$

At the Left End:

Actual Steel Provided: $A_{st}=3229mm^2=2.02\%$

$A_{sc}=1972mm^2=1.2\%$

$M_u/bd^2=min(6.0,6.4)=6.0$

Hogging moment capacity at left end(A)

$M_u^{Ah}=(6.0*300*532^2)/10^6=509.4KN\text{-}m$

$M_u^{As}=M_u=0.87f_yA_{st}d(1-(A_{st}f_y/b_fdf_{ck}))=307.3KN\text{-}m$

Hogging capacity at right end(B)

Ast=1.23%=pt

Asc=1.8%=pc

From table 51 SP-16: $M_u/bd^2=min(3.6,6.10)=3.6$

$M_u^{Bh}=(3.6*300*532^2)/10^6=305.3KN\text{-}m$

$M_u^{Bs}=M_u=0.87f_yA_{st}d(1-(A_{st}f_y/b_fdf_{ck}))=526KN\text{-}m$

$V_{sway\ to\ right}=\pm1.4(307.3+305.3)/7.5=114.3KN\text{-}m$

$V_{sway\ to\ left}=\pm1.4(509.4+526)/7.5=193.4KN\text{-}m$

Design Shear:

Dead Load=149.43KN

Live Load=51KN

Shear at Left end for sway at right=$V_{u,a}$

$V_{u,a}$=1.2(DL+LL)/2-1.4(M_u^{As}+M_u^{Bh})/L

=120-114.4=5.95KN

Shear at Left for sway to left= $V_{u,a}$

$V_{u,a}$=1.2(DL+LL)/2+1.4(M_u^{Ah}+M_u^{Bs})/L

=120+193.3=313KN

Shear at right for sway to right

$V_{u,b}$=1.2(DL+LL)/2+1.4(M_u^{As}+M_u^{Bh})/L

Shear at right for sway to left

$V_{u,b}$=1.2(DL+LL)/2-1.4(M_u^{Ah}+M_u^{Bs})/L

=-73.3KN

The design shear force shall be a maximum of:

(i)Calculated factored shear force as per analysis

(ii)Shear force due to formation of plastic hinge at both ends+factored gravity loads on span

Hence,

$V_{u\ left}$=313KN

$V_{u\ right}$=234KN

$(Vu-Vc)_{left}$=313-108.5=204.5KN

$(Vu-Vc)_{right}$=234-108.4=125.5KN Where Vc=$\tau_c$bd

We are taking 8Ø 2-legged stirrups

According to Table 62 of SP-16:spacing will be

Left=350mm

Right=500mm

The spacing at the rest of the beam member shall be limited to d/2=532/3=266mm

4.2 Interior Column Design

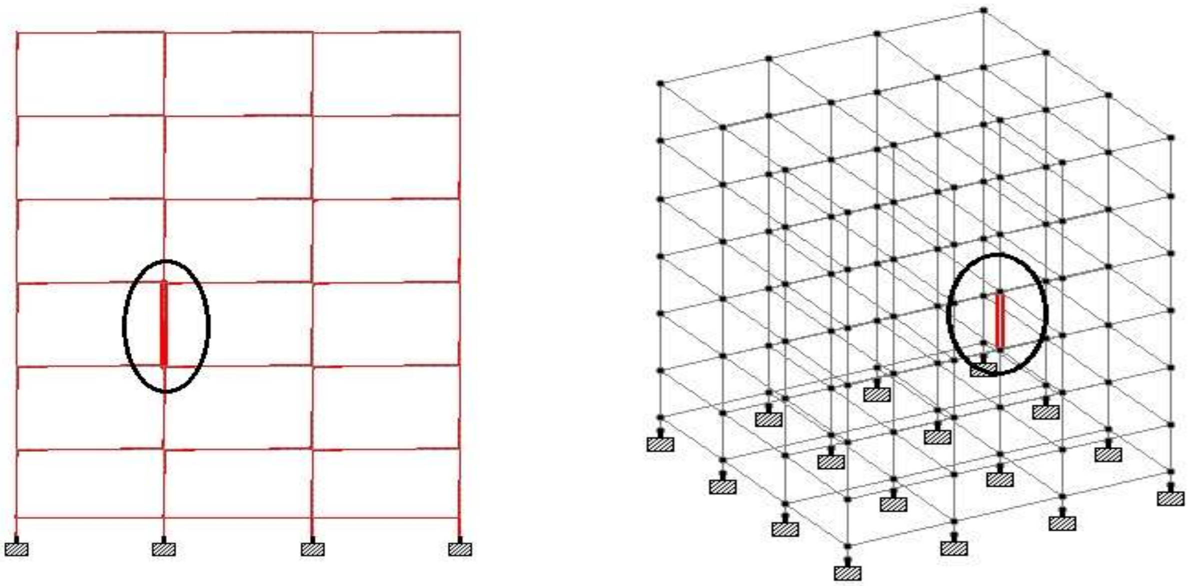


Fig 14: Column View In Elevation

For column AB, the force resultants for various load cases and load combinations are shown in Table ......

Design Checks

Check for Axial Stress

Factored axial force= 4993.081 kN (L/C 5)

Factored Axial Stress= 4993.081*1000/(500*500)=19.97> 0.1*fck*

Check for member size

Width of column, *B* = 500 mm > 300 mm

Hence, ok

(Clause 7.1.2; IS 13920:1993)

Depth of column, *D* = 500 mm

B/D=500/500=1 > 0.4, hence ok

(Clause 7.1.3; IS 13920:1993)

Span, *L* = 5000 mm

The effective length of column can be calculated using Annex E of IS 456: 2000. In this example as per Table 28 of IS 456: 2000, the effective length is taken as 0.85 times the unsupported length, which is in between that of fixed and hinged case.

L/D=7.48<12,

i.e., Short Column. Hence ok.

(Clause 25.1.2 of IS 456: 2000)

Check for Limiting Longitudinal Reinforcement
Minimum reinforcement,
= 0.8 %.
= 0.8 x 500 x 500/100
= 2,000 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

Maximum reinforcement = 4%
(Limited from practical considerations)
= 4 x 500 x 500/100
= 10,000 mm^2

(Clause 26.5.3.1 of IS 456: 2000)

Design of Column
Sample Calculation for Column Reinforcement at Bottom End (Node 58)
First approximate design is done and finally it is checked for all force combinations.
(a) Approximate Design
In this case, the moment about one axis dominates and hence the column is designed as an uniaxially loaded column.
Design for Earthquake in X-direction
P_u = 3846.38 kN
M_{u2} = -333.852 kN-m

$P_u/f_{ck}BD$=0.615
$M_{u2}/f_{ck}BD^2$ =0.106
d'/D=0.105

Referring to Charts of SP16
For d'/D=0.105, we get p/f_{ck}= 0.115

Design for Earthquake in Z direction
Pu = 3884.2 kN
Mu2 = -322.041 kN-m

$P_u/f_{ck}BD$=0.62
$Mu2/f_{ck}BD^2$ =0.103
d'/D=0.105

Referring to Charts of SP16
For *d'/D*=0.105, we get *p/fck*= 0.12

Longitudinal Steel
The required steel will be governed by the higher of the above two values and hence, take *p/fck* =0.12.
Required steel = (0.12 x 25) %
= 3%=7500 mm^2
Provide 12-32Φ bars with total *Asc* provided = 9650.97 mm^2
i.e., 9650.97 x100 /(500 x 500) = 3.85%.
Hence, *p/fck* provided = 3.85/25 = 0.154.

(b) Checking of Section

The column should be checked for bi-axial 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 with Earthquake in X Direction (Longitudinal direction)

Width = 500 mm; Depth = 500 mm
Pu = 3846.38 kN
Mu2 = -333.852 kN-m
Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((5000-600) / 500) + (500 / 30) = 25.467 mm > 20 mm
Hence, design eccentricity = 25.467 mm
Mu3 = 3846.38 x 0.025 = 97.95 kN-m
For $P_u/f_{ck}BD$ = 0.615 and p/f_{ck}=0.154
$Mu2/f_{ck}BD^2$=0.13

Mu21= *Mu31*=0.13*25*500*500*500= 406.25 kN-m

Puz = 0.45*fck Ac* + 0.75*fy Asc*
(Clause 39.6 of IS 456:2000)
= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 3846.38 / 5707.79 = 0.67

α*n* =1.78

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$

=0.784<1
Hence, ok

Checking for Critical Combination with Earthquake in Z Direction (Transverse direction)

Width = 500 mm; Depth = 500 mm
Pu = 3884.2 kN
Mu2 = -322.041 kN-m

Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((5000-600) / 500) + (500 / 30) = 25.467 mm > 20 mm
Hence, design eccentricity = 25.467 mm
Mu3 = 3884.2 x 0.025 = 98.91 kN-m
For $P_u/f_{ck}BD$ = 0.62 and p/f_{ck}=0.154
$Mu2/f_{ck}BD^2$=0.13

Mu21= *Mu31*=0.13*25*500*500*500= 406.25 kN-m

Puz = 0.45fck Ac + 0.75fy Asc

(Clause 39.6 of IS 456:2000)

= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 3884.2 / 5707.79 = 0.68

αn =1.8

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$

= 0.736<1

Hence, ok

Sample Calculation for Column Reinforcement at Top End (Node 74)

(a) Approximate Design

In this case, the moment about one axis dominates and hence the column is designed as an uniaxially loaded column.

Design for Earthquake in X-direction

P_u = -3846.38 kN

M_{u2} = -343.233 kN-m

$P_u/f_{ck}BD$=0.615

$M_{u2}/f_{ck}BD^2$ =0.109

d'/D=0.105

Referring to Charts of SP16

For d'/D=0.105, we get p/f_{ck}= 0.13

Design for Earthquake in Z direction

Pu = -3884.2 kN

$Mu2$ = -328.314 kN-m

$P_u/f_{ck}BD$=0.62

$Mu2/f_{ck}BD^2$ =0.105

d'/D=0.105

Referring to Charts of SP16

For d'/D=0.105, we get p/fck= 0.12

Longitudinal Steel

The required steel will be governed by the higher of the above two values and hence, take p/fck =0.13

Required steel = (0.13 x 25) %

= 3.25%=8125 mm^2

Provide 12-32Φ bars with total Asc provided = 9650.97 mm^2

i.e., 9650.97 x100 /(500 x 500) = 3.85%.
Hence, p/fck provided = 3.85/25 = 0.154.

(b) Checking of Section
The column should be checked for bi-axial 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 with Earthquake in X Direction (Longitudinal direction)
Width = 500 mm; Depth = 500 mm
Pu = -3846.38 kN
Mu2 = -343.233 kN-m
Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((5000-600) / 500) + (500 / 30) = 25.467 mm > 20 mm
Hence, design eccentricity = 25.467 mm
Mu3 = 3846.38 x 0.025 = 97.95 kN-m
For $P_u/f_{ck}BD$ = 0.615 and p/f_{ck}=0.154
$Mu2/f_{ck}BD^2$=0.13

Mu21= *Mu31*=0.13*25*500*500*500= 406.25 kN-m

Puz = 0.45*fck Ac* + 0.75*fy Asc*
(Clause 39.6 of IS 456:2000)
= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 3846.38 / 5707.79 = 0.67

α*n* =1.78

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$
=0.82<1
Hence, ok

Similar check is also performed when earthquake is in z-direction and is found to be satisfied.

Design for Shear
Shear Capacity of Column
Assuming 50% steel provided as tensile steel to be on conservative side, Ast = 3.86% / 2 = 1.93%
Permissible shear stress τ c = 0.81 Mpa
(Table 19 of IS 456: 2000)
Considering lowest Pu = 12.607 kN, we get
Multiplying factor = δ= 1+3Pu/(fck*Ag)= 1.006<1.5
(Clause 40.2.2 of IS 456: 2000)
τ c = 0.81 x 1.011 = 0.814 MPa

Effective depth in both direction = 500-40-25/2 = 447.5 mm
Vc = 0.814 x 500 x 447.5 /1,000 = 182.31 kN

Shear As Per Analysis
As per Table .... , the maximum factored shear force in X and Z direction is 135.417 and 130.071 kN, respectively.

Shear Force Due to Plastic Hinge Formation at Ends of Beam
Earthquake in X-Direction

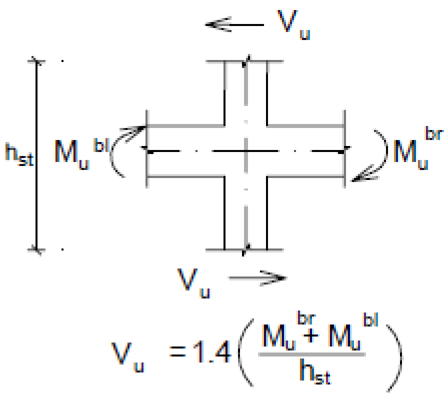


Fig 15: Column shear due to plastic hinge formation in beams

V_u = 1.4*(637.428-1.054)/5=178.184 kN (Values chosen against L/C 14)

Earthquake in Z-Direction

Vu = 1.4* (637.793-35.47/5)= 168.65 kN

Design Shear
The design shear force for the column shall be the higher of the calculated factored shear force as per analysis and the shear force due to plastic hinge formation in either of the transverseor longitudinal beams.
(Clause7.3.4; IS 13920: 1993)
From above, the design shear in X direction is 178.18 kN which is the higher of 135.417 kN and 178.18 kN . Similarly the design shear in Z direction is 168.65 kN, which is the higher of 130.07 kN and 168.65 kN.

Details of Transverse Reinforcement

Design of Links in X Direction
Vs = 178.18 – 182.31 < 0 (no need of transverse reinforcement)
But to be on conservative side we provide 8 Φ links @ 300 c/c i.e. maximum spacing.

Design of Links in Z Direction

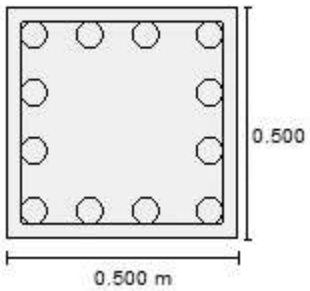
Vs = 168.65 – 1182.31 < 0 (no need of transverse reinforcement)
But to be on conservative side we provide 8 Φ links @ 300 c/c i.e. maximum spacing.

Nominal Links
The spacing of hoops shall not exceed half the least lateral dimension of the column, i.e., 300/ 2= 150 mm.

(Clause 7.3.3 of IS 13920: 1993)

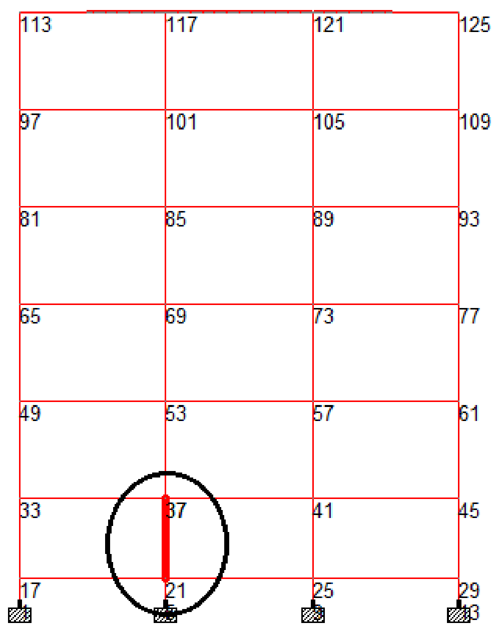
Provide 8 Φ links @ 150 c/c in mid-height portion of column.

Summary

<table>
<tr><td>Column 154</td><td>Longitudinal Reinforcement</td><td>Reinforcement Details</td></tr>
<tr><td>Reinforcement
At Bottom</td><td>12-32Φ bars with total Asc provided = 9650.97 mm^2</td><td rowspan="2">

Same at Both ends.</td></tr>
<tr><td>Reinforcement
at Top</td><td>12-32Φ bars with total Asc provided = 9650.97 mm^2</td></tr>
</table>

4.3 Exterior Column Design



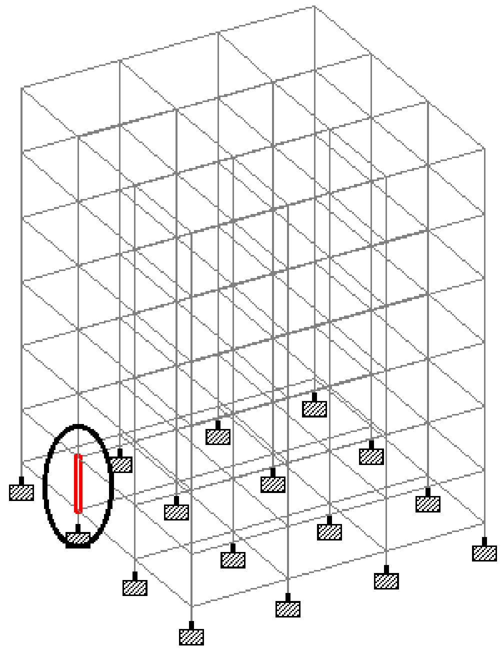


Fig 16: Column 69 view in Elevation

For column AB, the force resultants for various load cases and load combinations are shown in Table ......

Design Checks

Check for Axial Stress

Factored axial force= 4562.51kN (L/C 5)

Factored Axial Stress= 4562.51*1000/(500*500)=18.25> 0.1*fck*

Check for member size

Width of column, *B* = 500 mm > 300 mm

Hence, ok

(Clause 7.1.2; IS 13920:1993)

Depth of column, *D* = 500 mm

B/D=500/500=1 > 0.4, hence ok

(Clause 7.1.3; IS 13920:1993)

Span, *L* = 4,100 mm

The effective length of column can be calculated using Annex E of IS 456: 2000. In this example as per Table 28 of IS 456: 2000, the effective length is taken as 0.85 times the unsupported length, which is in between that of fixed and hinged case.

L/D=5.95<12,

i.e., Short Column. Hence ok.

(Clause 25.1.2 of IS 456: 2000)

Check for Limiting Longitudinal Reinforcement
Minimum reinforcement,
= 0.8 %.
= 0.8 x 500 x 500/100
= 2,000 mm^2
(Clause 26.5.3.1 of IS 456: 2000)
Maximum reinforcement = 4%
(Limited from practical considerations)
= 4 x 500 x 500/100
= 10,000 mm^2
(Clause 26.5.3.1 of IS 456: 2000)

Design of Column
Sample Calculation for Column Reinforcement at Bottom End (Node 21)
First approximate design is done and finally it is checked for all force combinations.
(a) Approximate Design
In this case, the moment about one axis dominates and hence the column is designed as an uniaxially loaded column.
Design for Earthquake in X-direction
P_u = 4065.61 kN
M_{u2} = -310.592 kN-m

$P_u/f_{ck}BD$=0.65
$M_{u2}/f_{ck}BD_2$ =0.099
d'/D=0.105

Referring to Charts of SP16
For d'/D=0.105, we get p/f_{ck}= 0.13

Design for Earthquake in Z direction
Pu = 3619.148 kN
Mu2 = 285.419 kN-m

$P_u/f_{ck}BD$=0.579
$Mu2/f_{ck}BD^2$ =0.0913
d'/D=0.105

Referring to Charts of SP16
For *d'/D*=0.105, we get *p/fck*= 0.1

Longitudinal Steel
The required steel will be governed by the higher of the above two values and hence, take *p/fck* =0.13.
Required steel = (0.13 x 25) %
= 3.25%=8125mm^2
Provide 12-32Φ bars with total *Asc* provided = 9650.97 mm^2
i.e., 9650.97 x100 /(500 x 500) = 3.85%.
Hence, *p/fck* provided = 3.85/25 = 0.154.

(b) Checking of Section

The column should be checked for bi-axial 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 with Earthquake in X Direction (Longitudinal direction)

Width = 500 mm; Depth = 500 mm
Pu = 4065.51 kN
Mu2 = -310.592 kN-m
Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((4100-600) / 500) + (500 / 30) = 23.67 mm > 20 mm
Hence, design eccentricity = 23.67 mm
Mu3 = 4065.91 x 0.023 = 96.22 kN-m
For $P_u/f_{ck}BD$ = 0.65 and p/f_{ck}=0.154
$Mu2/f_{ck}BD^2$=0.12

Mu21= *Mu31*=0.12*25*500*500*500= 375kN-m

Puz = 0.45*fck Ac* + 0.75*fy Asc*
(Clause 39.6 of IS 456:2000)
= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 4065.61 / 5707.79 = 0.71

α*n* =1.85

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$

= 0.786<1
Hence, ok

Checking for Critical Combination with Earthquake in Z Direction (Transverse direction)

Width = 500 mm; Depth = 500 mm
Pu = 3619.148 kN
Mu2 = 285.419 kN-m

Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((4100-600) / 500) + (500 / 30) = 23.67 mm > 20 mm
Hence, design eccentricity = 23.67 mm
Mu3 = 3619.148 x 0.023 = 85.65 kN-m
For $P_u/f_{ck}BD$ = 0.579 and p/f_{ck}=0.154
$Mu2/f_{ck}BD^2$=0.145

Mu21= *Mu31*=0.145*25*500*500*500= 453.125 kN-m

$Puz = 0.45 fck\ Ac + 0.75 fy\ Asc$

(Clause 39.6 of IS 456:2000)

= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 3619.148 / 5707.79 = 0.634

αn =1.723

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$

= 0.508<1

Hence, ok

Sample Calculation for Column Reinforcement at Top End (Node 37)

(a) Approximate Design

In this case, the moment about one axis dominates and hence the column is designed as an uniaxially loaded column.

Design for Earthquake in X-direction

Pu = -4065.61 kN

Mu2 = -231.658 kN-m

Pu/fckBD=0.65

Mu2/fckBD2 =0.074

d'/D=0.105

Referring to Charts of SP16

For d'/D=0.105, we get p/fck= 0.12

Design for Earthquake in Z direction

Pu = -3619.148 kN

Mu2 = -186.91 kN-m

Pu/fckBD=0.58

Mu2/fckBD2 =0.06

d'/D=0.105

Referring to Charts of SP16

For d'/D=0.105, we get p/fck= 0.04

Longitudinal Steel

The required steel will be governed by the higher of the above two values and hence, take p/fck =0.12.

Required steel = (0.12 x 25) %

= 3%=7500 mm^2

Provide 12-32Φ bars with total Asc provided = 9650.97 mm^2

i.e., 9650.97 x100 /(500 x 500) = 3.85%.
Hence, p/fck provided = 3.85/25 = 0.154.

(b) Checking of Section
The column should be checked for bi-axial 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 with Earthquake in X Direction (Longitudinal direction)
Width = 500 mm; Depth = 500 mm
P_u = 4065.51 kN
M_{u2} = -231.658 kN-m
Eccentricity = Clear height of column/500 +lateral dimension / 30
(Clause 25.4 of IS 456:2000)
= ((4100-600) / 500) + (500 / 30) = 23.67 mm > 20 mm
Hence, design eccentricity = 23.67 mm
M_{u3} = 4065.91 x 0.023 = 96.22 kN-m

For $P_u/f_{ck}BD$ = 0.65 and p/f_{ck}=0.154

$M_{u2}/f_{ck}BD^2$=0.12

M_{u21}= M_{u31}=0.12*25*500*500*500= 375kN-m

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$
(Clause 39.6 of IS 456:2000)
= 0.45 x 25 x(500 x 500 -9650.97)+ (0.75 x 415 x 9650.97) = 5707.79 kN

Pu/Puz = 4065.61 / 5707.79 = 0.71

αn =1.85

(Using the interaction formula of clause 39.6 of IS 456: 2000)

$$\left[\frac{M_{u2}}{M_{u2,1}}\right]^{\alpha_n} + \left[\frac{M_{u3}}{M_{u3,1}}\right]^{\alpha_n}$$
=0.786<1
Hence, ok

Similar check is also performed when earthquake is in z-direction and is found to be satisfied.

Design for Shear
Shear Capacity of Column
Assuming 50% steel provided as tensile steel to be on conservative side, Ast = 3.86% / 2 = 1.93%
Permissible shear stress τ c = 0.81 Mpa
(Table 19 of IS 456: 2000)
Considering lowest Pu = 23.524 kN, we get

Multiplying factor = δ= 1+3Pu/(fck*Ag)= 1.011<1.5

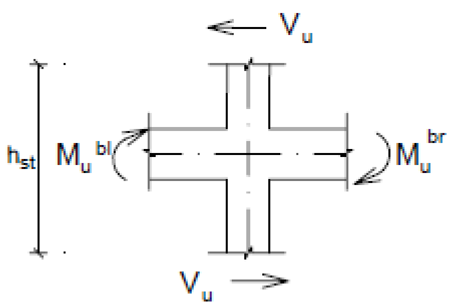
(Clause 40.2.2 of IS 456: 2000)

τ c = 0.81 x 1.011 = 0.819 MPa

Effective depth in both direction = 500-40-25/2 = 447.5 mm
Vc = 0.819 x 500 x 447.5 /1,000 = 183.25 kN

Shear As Per Analysis
As per Table , the maximum factored shear force in X and Z direction is 132.256 and 115.202 kN respectively.

Shear Force Due to Plastic Hinge Formation at Ends of Beam
Earthquake in X-Direction

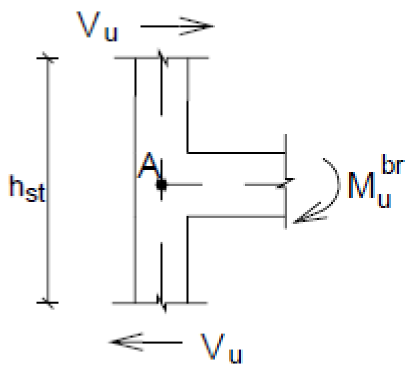


$$V_u = 1.4\left(\frac{M_u^{br} + M_u^{bl}}{h_{st}}\right)$$

Fig17: Column shear due to plastic hinge formation in beams

V_u = 1.4*(256.627+249.120)/4.1=172.63 kN (Values chosen against L/C 14)

Earthquake in Z-Direction



$$V_u = 1.4\left(\frac{M_u^{br}}{h_{st}}\right)$$

Fig18: Column shear due to plastic hinge formation in transverse beams

Vu = 1.4* (286/4.1)= 97.65 kN
Design Shear

The design shear force for the column shall be the higher of the calculated factored shear force as per analysis and the shear force due to plastic hinge formation in either of the transverseor longitudinal beams.
(Clause7.3.4; IS 13920: 1993)
From above, the design shear in X direction is 172.63 kN which is the higher of 132.56 kN and 172.63 kN . Similarly the design shear in Z direction is 115.202 kN, which is the higher of 115.202 kN and 97.65 kN.

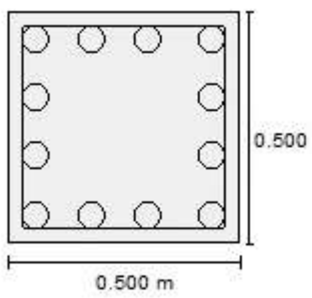
Details of Transverse Reinforcement

Design of Links in X Direction
Vs = 172.63 – 183.25 < 0 (no need of transverse reinforcement)
But to be on conservative side we provide 8 Φ links @ 300 c/c i.e. maximum spacing.

Design of Links in Z Direction
Vs = 115.202 – 183.25 < 0 (no need of transverse reinforcement)
But to be on conservative side we provide 8 Φ links @ 300 c/c i.e. maximum spacing.

Nominal Links
The spacing of hoops shall not exceed half the least lateral dimension of the column, i.e., 300/ 2= 150 mm.
(Clause 7.3.3 of IS 13920: 1993)
Provide 8 Φ links @ 150 c/c in mid-height portion of column.

Summary

<table>
<tr><td>Column 69</td><td>Longitudinal Reinforcement</td><td>Reinforcement Details</td></tr>
<tr><td>Reinforcement
At Bottom</td><td>12-32Φ bars with total Asc provided = 9650.97 mm^2</td><td rowspan="2">

Same at Both ends.</td></tr>
<tr><td>Reinforcement
at Top</td><td>12-32Φ bars with total Asc provided = 9650.97 mm^2</td></tr>
</table>

4.4 Footing Design(Isolated)

Design Of Footing No. 16

a=b=600 mm (width and length of column)
Safe Bearing capacity(q_o) = 300kN/m^2

Size Of Foundation:

Load from column= 2639.169 kN (L/C 5)
Weight of Foundation(10 %)= 263.91 kN

Total P_t= 2903.085 kN

Area Of Footing= P_t / q_o = 9.67 m^2
Designing a square footing , L=B=3.1 m

Net soil pressure(w_o)= 1.5*2639.16/9.67 = 409.385 kN/m^2

Check For Bending Moment:

Calculate moment for 1m strip,

M_x=M_y = w_o* $(B-b)^2$ /8 = 322.39 kN-m (as this is symmetrical footing)

Depth required:
d=$(M_x/Qb)^{0.5}$ = 305.37 mm ~ 310 mm

Eff. Cover= 80 mm
D=310+80= 390 mm

Check For One-Way Shear:

O_x = O_y = ((B-b)/2-d) = 0.945 m
Maximum Shear Force:
V_{uy} = 409.385*1*0.945 = 386.86 kN
τ_v = (386.86*1000)/(1000*310) = 1.247 > τ_{cmin} (failed)

Depth required:
d= (386.86*1000)/(1000*0.28) = 1381.64 mm

Check for d=750 mm
V_{uy} = w_o * ((L-a)/2 – d)= 204.69 kN
τ_{uy}= 0.2729< 0.28 (OK)

d= 750mm (OK)

Check For Punching Shear:

For d= 750mm

Punching Shear Developed = (Net Punching Shear)/(Resisting Area) = -0.18 N/mm^2

Punching Shear Permissible = k_s* 0.25$(f_{ck})^{0.5}$
where k_s = 0.5+b/a = 1.5 ≤ 1
= 1

Punching Shear permissible= 1.25 N/mm^2

Punching shear developed<Punching Shear Permissible (Hence Ok).

Area Of Steel:

Since M_x=M_y , Area of steel in both directions is equal.

$A_{st} = \frac{.5fck}{fy} \times \left[1 - \sqrt{1 - \frac{4.6\times Mu}{fck\times B\times d\times d}}\right]$ = 1224.33 mm^2

for total 'L=3.1m' width = 3.1* 1224.33 mm^2= 3795.44 mm^2

Total number of 10mm ∅ bars = 48.32 ~ 49

Number Of bars in Central Band $n_c = \frac{2}{1+\frac{L}{B}}$ *32 = 32 bars

4.5 Design Results from STAAD.Pro V8i

Some of the sample analysis and design results have been shown below for beam number 52 which is at the roof level of 1st floor.

B E A M N O. 57 D E S I G N R E S U L T S
M25 Fe415 (Main) Fe415 (Sec.)

LENGTH: 7500.0 mm SIZE: 300.0 mm X 600.0 mm COVER: 40.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	1875.0 mm	3750.0 mm	5625.0 mm	7500.0 mm
TOP REINF.	341.02 (Sq. mm)	341.02 (Sq. mm)	341.02 (Sq. mm)	341.02 (Sq. mm)	404.35 (Sq. mm)
BOTTOM REINF.	376.05 (Sq. mm)	341.02 (Sq. mm)	0.00 (Sq. mm)	341.02 (Sq. mm)	341.02 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

```
SECTION      0.0 mm     1875.0 mm     3750.0 mm     5625.0 mm     7500.0 mm
----------------------------------------------------------------------------
TOP        4-12í         4-12í         4-12í         4-12í         4-12í
REINF.   1 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)

BOTTOM     5-10í         5-10í         2-10í         5-10í         5-10í
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  8í
REINF.  @ 180 mm c/c  @ 180 mm c/c  @ 180 mm c/c  @ 180 mm c/c  @ 180 mm c/c
----------------------------------------------------------------------------
```

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

SHEAR DESIGN RESULTS AT   850.0 mm AWAY FROM START SUPPORT
VY =   -19.88 MX =   -0.26 LD=   10
Provide 2 Legged 8í @ 180 mm c/c

SHEAR DESIGN RESULTS AT   850.0 mm AWAY FROM END SUPPORT
VY =   -19.88 MX =   -0.26 LD=   10
Provide 2 Legged 8í @ 180 mm c/c

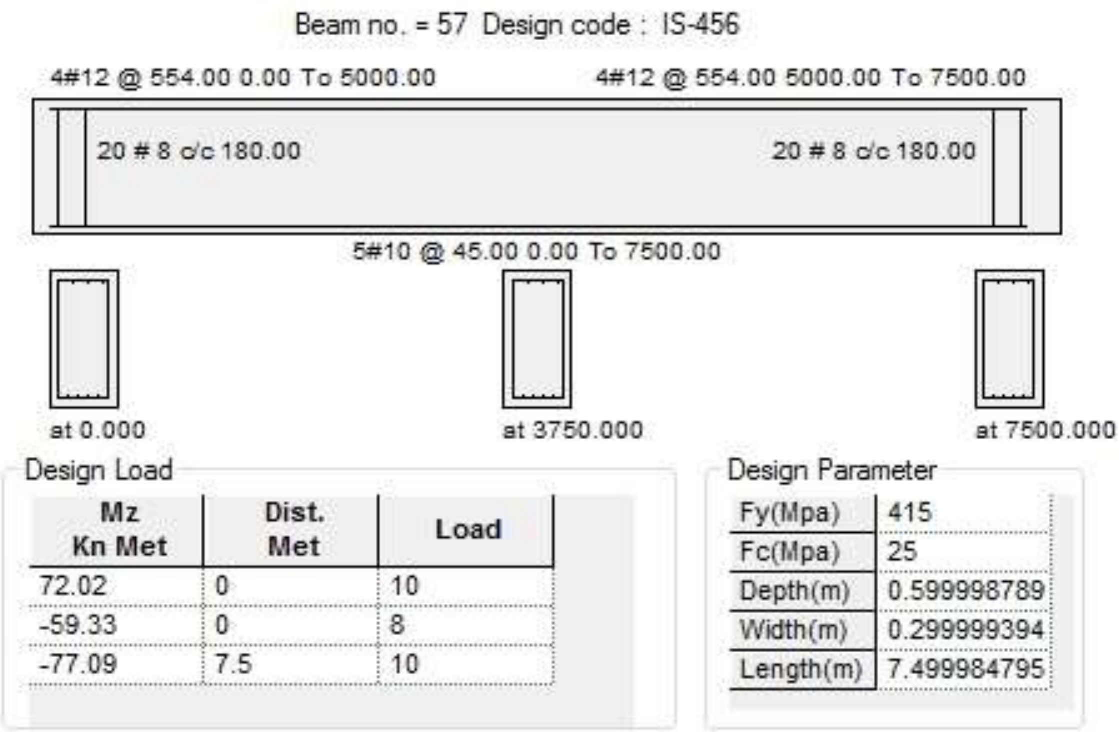


Fig. 19- Reinforcement Detailing of Beam

C O L U M N N O.   69 D E S I G N R E S U L T S

M25 Fe415 (Main) Fe415 (Sec.)

LENGTH: 4100.0 mm CROSS SECTION: 500.0 mm X 500.0 mm COVER: 40.0 mm

** GUIDING LOAD CASE: 5 END JOINT: 37 SHORT COLUMN

REQD. STEEL AREA : 8833.38 Sq.mm.
REQD. CONCRETE AREA: 241166.62 Sq.mm.
MAIN REINFORCEMENT : Provide 12 - 32 dia. (3.86%, 9650.97 Sq.mm.)
(Equally distributed)
TIE REINFORCEMENT : Provide 8 mm dia. rectangular ties @ 300 mm c/c

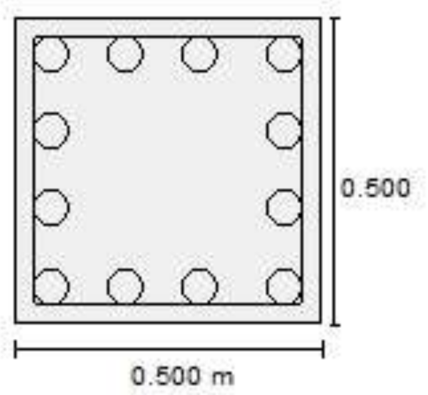
SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)

Puz : 5462.51 Muz1 : 210.87 Muy1 : 210.87

INTERACTION RATIO: 0.97 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)

Beam no. = 69 Design code : IS-456



Design Load

Load	5
Location	End 2
Pu(Kns)	4562.51
Mz(Kns-Mt)	173.95
My(Kns-Mt)	5.59

Design Parameter

Fy(Mpa)	415
Fc(Mpa)	25
As Reqd(mm²)	8833
As (%)	3.86
Bar Size	32
Bar No	12

Fig. 20- Reinforcement Detailing of Column

Design of Footing

Isolated Footing 16

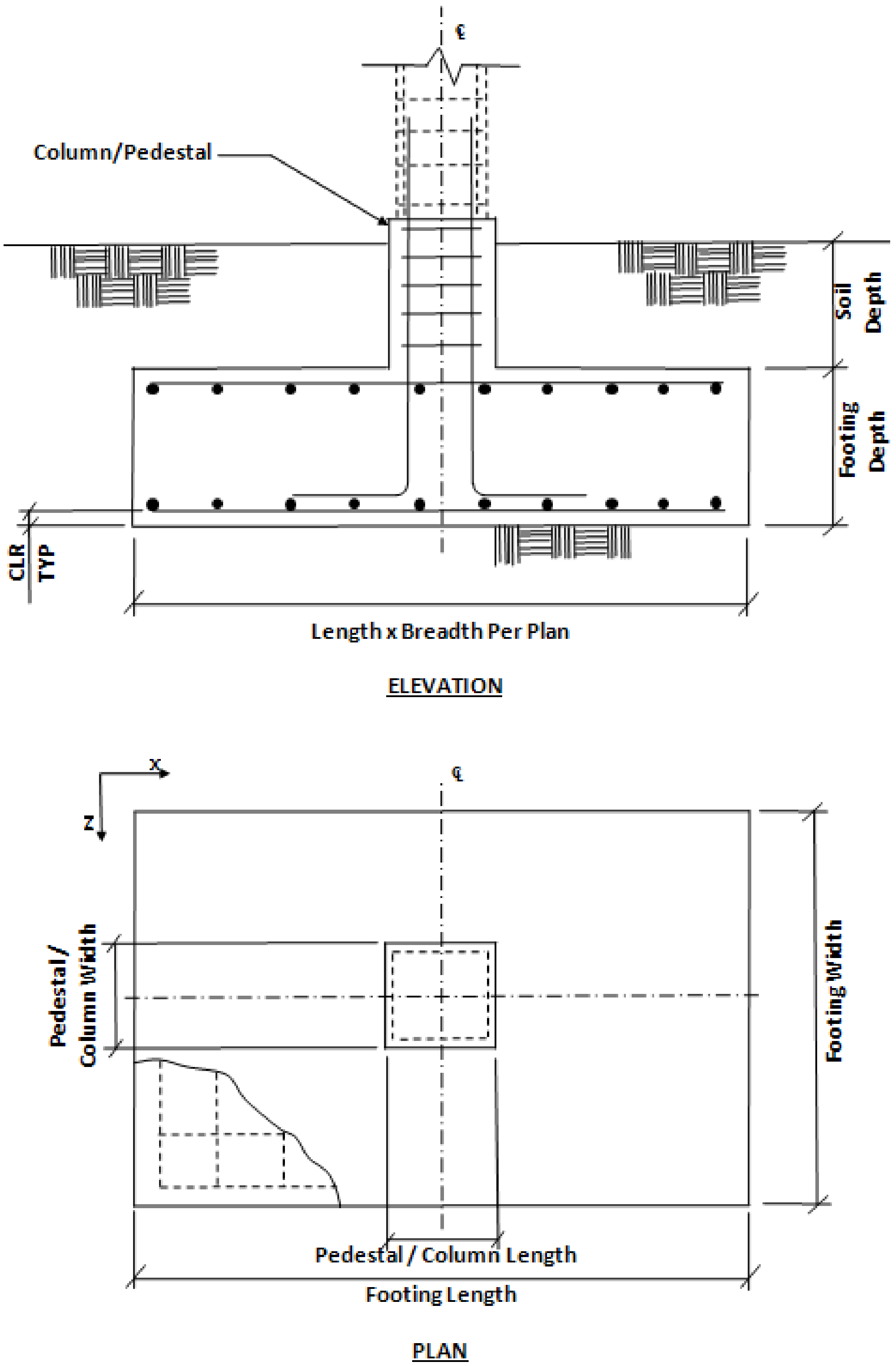


Fig 21: Plan and Elevation of Footing

Input Values

Unit Weight of Concrete :	25.000 kN/m^3
Strength of Concrete :	25.000 N/mm^2
Yield Strength of Steel :	415.000 N/mm^2
Minimum Bar Size :	Ø6
Maximum Bar Size :	Ø32
Minimum Bar Spacing :	50.000 mm
Maximum Bar Spacing :	500.000 mm
Pedestal Clear Cover (P, CL) :	50.000 mm
Footing Clear Cover (F, CL) :	50.000 mm

Footing Geomtery

Footing Thickness (Ft) :	305.000 mm
Footing Length - X (Fl) :	1000.000 mm
Footing Width - Z (Fw) :	1000.000 mm

Design Parameters

Concrete and Rebar Properties

Soil Properties

Soil Type :	Drained
Unit Weight :	22.000 kN/m^3
Soil Bearing Capacity :	300.000 kN/m^2
Soil Surcharge :	0.000 kN/m^2
Depth of Soil above Footing :	0.000 mm
Cohesion :	0.000 kN/m^2
Min Percentage of Slab :	0.000

Sliding and Overturning

Coefficient of Friction :	0.500
Factor of Safety Against Sliding :	1.500
Factor of Safety Against Overturning :	1.500

Load Combination/s- Service Stress Level	
Load Combination Number	**Load Combination Title**
1	LOAD CASE 1 SIESMIC +X
2	LOAD CASE 2 SIESMIC +Z
3	LOAD CASE 3 DEAD LOAD
4	LOAD CASE 4 LIVE LOAD
5	GENERATED INDIAN CODE GENRAL_STRUCTURES 1
6	GENERATED INDIAN CODE GENRAL_STRUCTURES 2
7	GENERATED INDIAN CODE GENRAL_STRUCTURES 3
8	GENERATED INDIAN CODE GENRAL_STRUCTURES 4
9	GENERATED INDIAN CODE GENRAL_STRUCTURES 5
10	GENERATED INDIAN CODE GENRAL_STRUCTURES 6
11	GENERATED INDIAN CODE GENRAL_STRUCTURES 7
12	GENERATED INDIAN CODE GENRAL_STRUCTURES 8
13	GENERATED INDIAN CODE GENRAL_STRUCTURES 9
14	GENERATED INDIAN CODE GENRAL_STRUCTURES 10
15	GENERATED INDIAN CODE GENRAL_STRUCTURES 11
16	GENERATED INDIAN CODE GENRAL_STRUCTURES 12
17	GENERATED INDIAN CODE GENRAL_STRUCTURES 13
18	GENERATED INDIAN CODE GENRAL_STRUCTURES 14
19	GENERATED INDIAN CODE GENRAL_STRUCTURES 15

Load Combination/s- Strength Level	
Load Combination Number	**Load Combination Title**
1	LOAD CASE 1 SIESMIC +X
2	LOAD CASE 2 SIESMIC +Z
3	LOAD CASE 3 DEAD LOAD
4	LOAD CASE 4 LIVE LOAD
5	GENERATED INDIAN CODE GENRAL_STRUCTURES 1
6	GENERATED INDIAN CODE GENRAL_STRUCTURES 2
7	GENERATED INDIAN CODE GENRAL_STRUCTURES 3
8	GENERATED INDIAN CODE GENRAL_STRUCTURES 4

9	GENERATED INDIAN CODE GENRAL_STRUCTURES 5
10	GENERATED INDIAN CODE GENRAL_STRUCTURES 6
11	GENERATED INDIAN CODE GENRAL_STRUCTURES 7
12	GENERATED INDIAN CODE GENRAL_STRUCTURES 8
13	GENERATED INDIAN CODE GENRAL_STRUCTURES 9
14	GENERATED INDIAN CODE GENRAL_STRUCTURES 10
15	GENERATED INDIAN CODE GENRAL_STRUCTURES 11
16	GENERATED INDIAN CODE GENRAL_STRUCTURES 12
17	GENERATED INDIAN CODE GENRAL_STRUCTURES 13
18	GENERATED INDIAN CODE GENRAL_STRUCTURES 14
19	GENERATED INDIAN CODE GENRAL_STRUCTURES 15

Applied Loads - Service Stress Level

LC	Axial (kN)	Shear X (kN)	Shear Z (kN)	Moment X (kNm)	Moment Z (kNm)
1	256.296	54.655	0.055	0.065	-162.419
2	256.296	0.055	54.655	162.419	-0.065
3	1455.522	1.267	1.267	17.794	-17.794
4	303.924	0.353	0.353	4.833	-4.833
5	2639.169	2.429	2.429	33.941	-33.941
6	2111.335	1.943	1.943	27.153	-27.153
7	2418.891	67.529	2.009	27.230	-222.056
8	2418.891	2.009	67.529	222.056	-27.230
9	1803.780	-63.642	1.877	27.075	167.750
10	1803.780	1.877	-63.642	-167.750	-27.075
11	2183.284	1.900	1.900	26.691	-26.691
12	2567.728	83.882	1.982	26.788	-270.320
13	2567.728	1.983	83.882	270.320	-26.788
14	1798.839	-80.082	1.818	26.594	216.938
15	1798.839	1.818	-80.082	-216.938	-26.594
16	1694.415	83.122	1.222	16.111	-259.644
17	1694.415	1.222	83.122	259.644	-16.111
18	925.526	-80.842	1.058	15.918	227.614
19	925.526	1.058	-80.842	-227.614	-15.918

Applied Loads - Strength Level

LC	Axial (kN)	Shear X (kN)	Shear Z (kN)	Moment X (kNm)	Moment Z (kNm)
1	256.296	54.655	0.055	0.065	-162.419
2	256.296	0.055	54.655	162.419	-0.065
3	1455.522	1.267	1.267	17.794	-17.794
4	303.924	0.353	0.353	4.833	-4.833
5	2639.169	2.429	2.429	33.941	-33.941
6	2111.335	1.943	1.943	27.153	-27.153
7	2418.891	67.529	2.009	27.230	-222.056
8	2418.891	2.009	67.529	222.056	-27.230
9	1803.780	-63.642	1.877	27.075	167.750
10	1803.780	1.877	-63.642	-167.750	-27.075

11	2183.284	1.900	1.900	26.691	-26.691
12	2567.728	83.882	1.982	26.788	-270.320
13	2567.728	1.983	83.882	270.320	-26.788
14	1798.839	-80.082	1.818	26.594	216.938
15	1798.839	1.818	-80.082	-216.938	-26.594
16	1694.415	83.122	1.222	16.111	-259.644
17	1694.415	1.222	83.122	259.644	-16.111
18	925.526	-80.842	1.058	15.918	227.614
19	925.526	1.058	-80.842	-227.614	-15.918

Design Calculations

Footing Size

Initial Length (L_o) = 1.000 m

Initial Width (W_o) = 1.000 m

Uplift force due to buoyancy = 0.000 kN

Effect due to adhesion = 0.000 kN

Area from initial length and width, A_o = L_o X W_o = 1.000 m^2

Min. area required from bearing pressure, A_{min} = P / q_{max} = 8.823 m^2

Note: A_{min} is an initial estimation.
P = Critical Factored Axial Load(without self weight/buoyancy/soil).
q_{max} = Respective Factored Bearing Capacity.

Final Footing Size

Length (L_2) =	3.300	m	Governing Load Case :	# 12
Width (W_2) =	3.300	m	Governing Load Case :	# 12
Depth (D_2) =	0.756	m	Governing Load Case :	# 12
Area (A_2) =	10.890	m^2		

Check For Stability Against Overturning And Sliding

-	Factor of safety against sliding		Factor of safety against overturning	
Load Case No.	**Along X-Direction**	**Along Z-Direction**	**About X-Direction**	**About Z-Direction**
1	3.104	3092.060	6891.398	3.126
2	3092.051	3.104	3.126	6891.353
3	607.264	607.264	139.632	139.632
4	548.857	548.858	129.226	129.226
5	560.364	560.363	129.507	129.507
6	564.635	564.637	130.495	130.495
7	18.525	622.678	148.264	17.012
8	622.673	18.525	17.012	148.264
9	14.824	502.526	112.601	16.634
10	502.527	14.824	16.634	112.601
11	596.339	596.339	137.120	137.120
12	15.801	668.544	159.667	14.781
13	668.540	15.801	14.781	159.666
14	11.750	517.597	114.371	12.865
15	517.600	11.750	12.865	114.371
16	10.692	727.025	177.911	10.290
17	727.018	10.692	10.290	177.911
18	6.238	476.718	102.465	6.596
19	476.722	6.238	6.596	102.465

Critical Load Case And The Governing Factor Of Safety For Overturning and Sliding X Direction

Critical Load Case for Sliding along X-Direction :	1
Governing Disturbing Force :	54.655 kN
Governing Restoring Force :	169.666 kN
Minimum Sliding Ratio for the Critical Load Case :	3.104
Critical Load Case for Overturning about X-Direction :	2
Governing Overturning Moment :	179.089 kN-m
Governing Resisting Moment :	559.889 kN-m

Minimum Overturning Ratio for the Critical Load Case : 3.126

Critical Load Case And The Governing Factor Of Safety For Overturning and Sliding Z Direction

Critical Load Case for Sliding along Z-Direction :	2
Governing Disturbing Force :	54.655 kN
Governing Restoring Force :	169.666 kN
Minimum Sliding Ratio for the Critical Load Case :	3.104
Critical Load Case for Overturning about Z-Direction :	1
Governing Overturning Moment :	-179.089 kN-m
Governing Resisting Moment :	559.889 kN-m
Minimum Overturning Ratio for the Critical Load Case :	3.126

Moment Calculation

Check Trial Depth against moment (w.r.t. X Axis)

Critical Load Case **= #19**

Effective Depth = $D - (cc + 0.5 \times d_b)$ = 0.402 m

Governing moment (M_u) = 347.653 kN-m

As Per IS 456 2000 ANNEX G G-1.1C

Limiting Factor1 (K_{umax}) = $\frac{700}{(1100 + 0.87 \times f_y)}$ = 0.479107

Limiting Factor2 (R_{umax}) = $0.36 \times f_{ck} \times k_{umax} \times (1 - 0.42 \times kumax)$ = 3444.291146 kN/m^2

Limit Moment Of Resistance (M_{umax}) = $R_{umax} \times B \times d_e^2$ = 1836.783511 kNm

M_u <= M_{umax} hence, safe

Check Trial Depth against moment (w.r.t. Z Axis)

Critical Load Case **= #13**

Effective Depth = $D - (cc + 0.5 \times d_b)$ = 0.702 m

Governing moment (M_u) = 719.042 kN-m

As Per IS 456 2000 ANNEX G G-1.1C

Limiting Factor1 (K_{umax}) = $\frac{700}{(1100 + 0.87 \times f_y)}$ = 0.479107

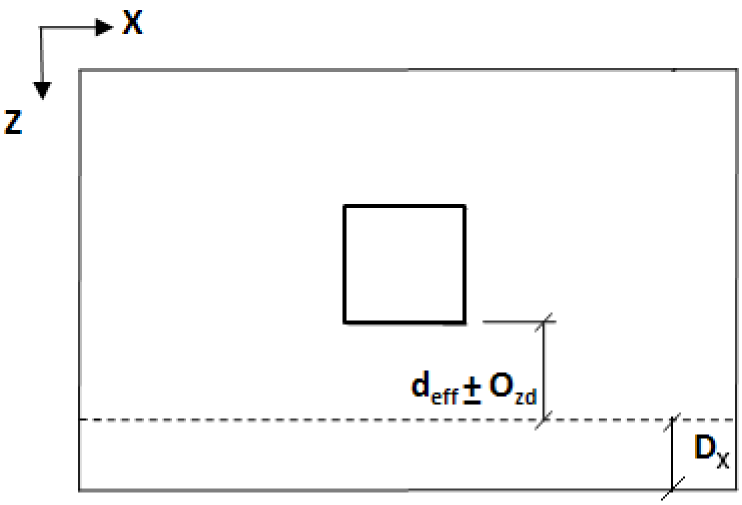
Limiting Factor2 (R_{umax}) = $0.36 \times f_{ck} \times k_{umax} \times (1 - 0.42 \times kumax)$ = 3444.291146 kN/m^2

Limit Moment Of Resistance (M_{umax}) = $R_{umax} \times B \times d_e^2$ = 5601.187159 kN-m

M_u <= M_{umax} hence, safe

Shear Calculation

Check Trial Depth for one way shear (Along X Axis) (Shear Plane Parallel to X Axis)



PLAN

Critical Load Case **= #13**

D_X = 0.702 m

Shear Force(S) = 589.111 kN

Shear Stress(T_v) = 254.299693 kN/m^2

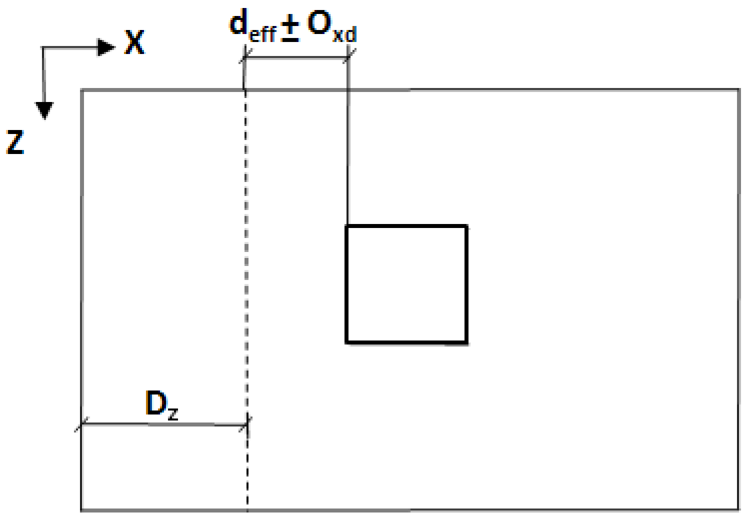
Percentage Of Steel(P_t) = 0.1292

As Per IS 456 2000 Clause 40 Table 19

Shear Strength Of Concrete(T_c) = 271.872 kN/m^2

T_v< T_c hence, safe

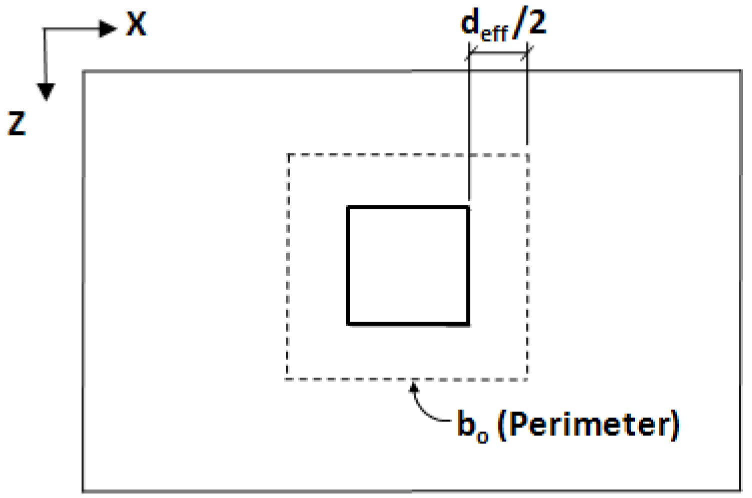
Check Trial Depth for one way shear (Along Z Axis)
(Shear Plane Parallel to Z Axis)



PLAN

Critical Load Case	**= #12**
D_Z =	0.652 m
Shear Force(S)	= 632.842 kN
Shear Stress(T_v)	= 294.126461 kN/m^2
Percentage Of Steel(P_t)	= 0.1455
As Per IS 456 2000 Clause 40 Table 19	
Shear Strength Of Concrete(T_c)	= 918.328 kN/m^2
	$T_v < T_c$ hence, safe

Check Trial Depth for two way shear



PLAN

Critical Load Case	**= #5**
Shear Force(S)	= 2289.025 kN
Shear Stress(T_v)	= 790.842 kN/m^2

As Per IS 456 2000 Clause 31.6.3.1

K_s = $\min[(0.5 + \beta), 1]$= 1.000

Shear Strength(T_c)= $0.25 \times \sqrt{f_{ck}}$= 1250.0000 kN/$m^2$

K_s x T_c = 1250.0000 kN/m^2

T_v<= K_s x T_c hence, safe

Reinforcement Calculation

Calculation of Maximum Bar Size

Along X Axis

Bar diameter corresponding to max bar size (d_b) = 32 mm

As Per IS 456 2000 Clause 26.2.1

Development Length(l_d) = $\frac{d_b \times 0.87 \times f_y}{4 \times \tau_{bd}}$ = 1.289 m

Allowable Length(l_{db}) = $\left[\frac{(B-b)}{2} - cc\right]$= 1.300 m

l_{db} >=l_d hence, safe

Along Z Axis

Bar diameter corresponding to max bar size(d_b) = 32 mm

As Per IS 456 2000 Clause 26.2.1

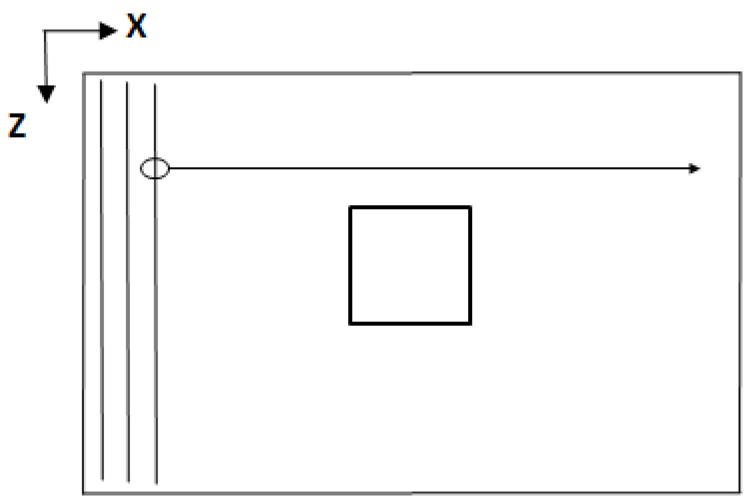
Development Length(l_d) = $\frac{d_b \times 0.87 \times f_y}{4 \times \tau_{bd}}$ = 1.289 m

Allowable Length(l_{db}) = $\left[\frac{(H-h)}{2} - cc\right]$= 1.300 m

l_{db} >=l_d hence, safe

Bottom Reinforcement Design

Along Z Axis



PLAN

For moment w.r.t. X Axis (M_x)

As Per IS 456 2000 Clause 26.5.2.1

Critical Load Case **= #19**

Minimum Area of Steel (A_{stmin}) = 2997.720 mm^2

Calculated Area of Steel (A_{st}) = 3302.317 mm^2

Provided Area of Steel ($A_{st,Provided}$) = 3302.317 mm^2

A_{stmin}<= $A_{st,Provided}$ Steel area is accepted

Selected bar Size (d_b) = Ø10

Minimum spacing allowed (S_{min}) = 50.000 mm

Selected spacing (S) = 75.952 mm

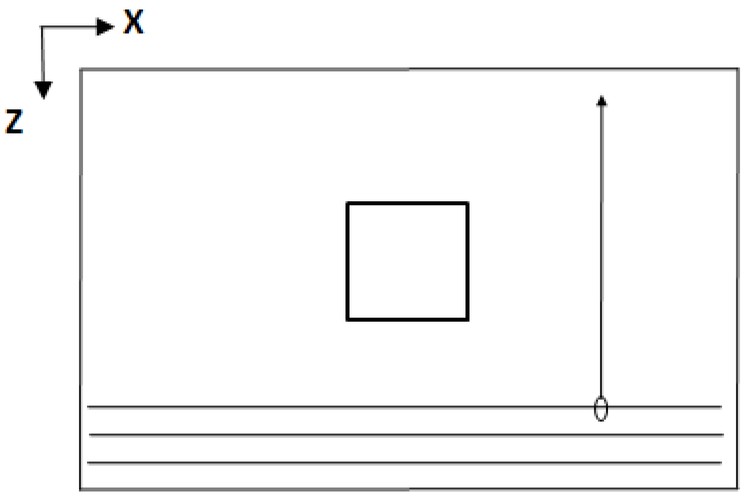
S_{min} <= S <= S_{max} and selected bar size < selected maximum bar size...

The reinforcement is accepted.

Based on spacing reinforcement increment; provided reinforcement is

Ø10 @ 75.000 mm o.c.

Along X Axis



PLAN

For moment w.r.t. Z Axis (M_z)

As Per IS 456 2000 Clause 26.5.2.1

Critical Load Case **= #13**

Minimum Area of Steel (A_{stmin}) = 1805.760 mm^2

Calculated Area of Steel (A_{st}) = 1844.294 mm^2

Provided Area of Steel ($A_{st,Provided}$) = 1844.294 mm^2

A_{stmin}<= $A_{st,Provided}$ Steel area is accepted

Selected bar Size (d_b) = Ø8

Minimum spacing allowed (S_{min}) = = 50.000 mm

Selected spacing (S) = 88.667 mm

S_{min} <= S <= S_{max} and selected bar size < selected maximum bar size...

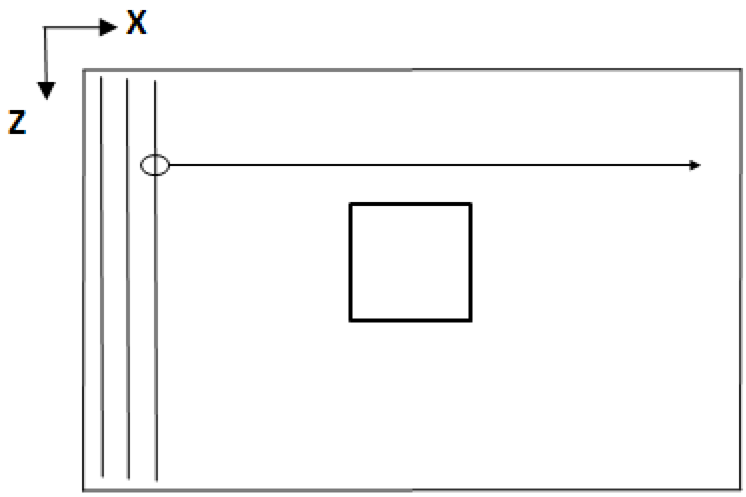
The reinforcement is accepted.

Based on spacing reinforcement increment; provided reinforcement is

Ø8 @ 85.000 mm o.c.

Top Reinforcement Design

Along Z Axis



PLAN

Minimum Area of Steel (A_{stmin}) = 2997.720 mm^2

Calculated Area of Steel (A_{st}) = 1801.800 mm^2

Provided Area of Steel ($A_{st,Provided}$) = 2997.720 mm^2

$A_{stmin} <= A_{st,Provided}$ Steel area is accepted

Governing Moment = 25.477 kN-m

Selected bar Size (d_b) = Ø6

Minimum spacing allowed (S_{min}) = 50.000 mm

Selected spacing (S) = 50.698 mm

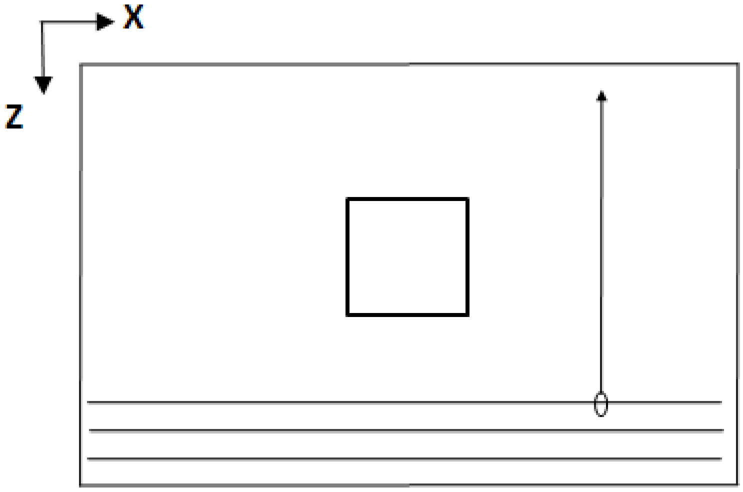
$S_{min} <= S <= S_{max}$ and selected bar size < selected maximum bar size...

The reinforcement is accepted.

Based on spacing reinforcement increment; provided reinforcement is

Ø6 @ 50 mm o.c.

Along X Axis



PLAN

Minimum Area of Steel (A_{stmin}) = 1805.760 mm^2

Calculated Area of Steel (A_{st}) = 1844.294 mm^2

Provided Area of Steel ($A_{st,Provided}$) = 1844.294 mm^2

$A_{stmin} <= A_{st,Provided}$ Steel area is accepted

Governing Moment = 25.477 kN-m

Selected bar Size (d_b) = Ø8

Minimum spacing allowed (S_{min}) = = 50.000 mm

Selected spacing (S) = 88.667 mm

S_{min} <= S <= S_{max} and selected bar size < selected maximum bar size...

The reinforcement is accepted.

Based on spacing reinforcement increment; provided reinforcement is

Ø8 @ 85 mm o.c.

CHAPTER 5: CONCLUSION

STAAD PRO has the capability to calculate the reinforcement needed for any concrete section. The program contains a number of parameters which are designed as per IS:456 (2000). Also the results obtained from STAAD.Pro are comparable with the manual design results.
The frame selected is also found to be safe in storey drift calculations.

Beams are designed for flexure, shear and torsion.

Design for Flexure:
Maximum sagging (creating tensile stress at the bottom face of the beam) and hogging (creating tensile stress at the top face) moments are calculated for all active load cases at each of the above mentioned sections. Each of these sections are designed to resist both of these critical sagging and hogging moments. Where ever the rectangular section is inadequate as Singly reinforced section, doubly reinforced section is tried.
Design for Shear*:*
Shear reinforcement is calculated to resist both shear forces and torsional moments. Shear capacity calculation at different sections without the shear reinforcement is based on the actual tensile reinforcement provided by STAAD program. Two-legged stirrups are provided to take care of the balance shear forces acting on these sections.
Beam Design Output:
The default design output of the beam contains flexural and shear reinforcement provided along the length of the beam.
Column Design:
Columns are designed for axial forces and biaxial moments at the ends. All active load cases are tested to calculate reinforcement. The loading which yield maximum reinforcement is called the critical load. Column design is done for square section. Square columns are designed with reinforcement distributed on each side equally for the sections under biaxial moments and with reinforcement distributed equally in two faces for sections under uni-axial moment. All major criteria for selecting longitudinal and transverse reinforcement as stipulated by IS: 456 have been taken care of in the column design of STAAD.

ANNEXURE A- Maximum & Minimum Support Reactions

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kN)	FY (kN)	FZ (kN)	MX (kN·m)	MY (kN·m)	MZ (kN·m)
Max FX	5	14:GENERATEI	**224.527**	5.45E+3	-20.572	-13.873	1.799	-295.587
Min FX	8	12:GENERATEI	**-224.527**	5.45E+3	-20.572	-13.873	-1.799	295.587
Max FY	6	5:GENERATED	-35.784	**9.67E+3**	-35.784	-24.230	-0.000	24.230
Min FY	5	1:LOAD CASE	-71.848	**-343.922**	-0.046	-0.041	-1.215	212.823
Max FZ	3	15:GENERATEI	20.572	5.45E+3	**224.527**	295.587	1.799	-13.873
Min FZ	14	13:GENERATEI	-20.572	5.45E+3	**-224.527**	-295.587	1.799	13.873
Max MX	10	15:GENERATEI	-23.013	7.57E+3	175.026	**345.569**	-1.071	15.771
Min MX	6	13:GENERATEI	-23.013	7.57E+3	-175.026	**-345.569**	1.071	15.771
Max MY	1	17:GENERATEI	57.733	1.4E+3	-38.841	-300.647	**2.613**	13.539
Min MY	1	12:GENERATEI	-0.287	2.64E+3	96.286	-22.489	**-2.613**	309.597
Max MZ	6	12:GENERATEI	-175.026	7.57E+3	-23.013	-15.771	-1.071	**345.569**
Min MZ	11	14:GENERATEI	175.026	7.57E+3	23.013	15.771	-1.071	**-345.569**

ANNEXURE B- Maximum & Minimum Node Displacement

Node Displacement Summary

	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	117	12:GENERATE	**156.228**	-12.554	0.085	156.732	-0.000	0.001	-0.004
Min X	120	14:GENERATE	**-156.228**	-12.554	0.085	156.732	-0.000	-0.001	0.004
Max Y	117	1:LOAD CASE	103.959	**0.879**	0.008	103.963	-0.000	0.001	-0.001
Min Y	118	5:GENERATED	0.125	**-26.592**	0.125	26.593	0.000	0.000	-0.000
Max Z	114	13:GENERATE	0.085	-12.554	**156.228**	156.732	0.004	-0.001	0.000
Min Z	126	15:GENERATE	0.085	-12.554	**-156.228**	156.732	-0.004	0.001	0.000
Max rX	50	13:GENERATE	-0.000	-6.477	52.140	52.541	**0.007**	-0.000	-0.000
Min rX	62	15:GENERATE	-0.000	-6.477	-52.140	52.541	**-0.007**	0.000	-0.000
Max rY	113	12:GENERATE	136.212	-7.514	0.242	136.419	0.002	**0.001**	-0.003
Min rY	116	18:GENERATE	-136.124	-4.049	0.155	136.185	0.001	**-0.001**	0.003
Max rZ	56	14:GENERATE	-52.140	-6.477	-0.000	52.541	0.000	-0.000	**0.007**
Min rZ	53	12:GENERATE	52.140	-6.477	-0.000	52.541	0.000	0.000	**-0.007**
Max Rst	118	12:GENERATE	156.040	-21.387	0.096	**157.499**	0.000	0.001	-0.001

ANNEXURE C- Maximum & Minimum Beam end forces

Beam End Force Summary

The signs of the forces at end B of each beam have been reversed. For example: this means that the Min Fx entry gives the largest tension value for an beam.

				Axial	Shear		Torsion	Bending	
	Beam	Node	L/C	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)
Max Fx	30	6	5:GENERATED	**9.67E+3**	35.784	-35.784	-0.000	24.230	24.230
Min Fx	29	5	1:LOAD CASE	**-343.922**	71.848	-0.046	-1.215	0.041	212.823
Max Fy	127	54	12:GENERATEI	-5.759	**380.857**	-1.649	-0.078	5.745	761.339
Min Fy	142	54	13:GENERATEI	-5.759	**-380.857**	1.649	0.078	5.745	761.339
Max Fz	27	3	15:GENERATEI	5.45E+3	-20.572	**224.527**	1.799	-295.587	-13.873
Min Fz	38	14	13:GENERATEI	5.45E+3	20.572	**-224.527**	1.799	295.587	13.873
Max Mx	188	68	18:GENERATEI	673.357	-36.448	31.933	**4.916**	-79.054	-75.019
Min Mx	185	65	16:GENERATEI	673.357	36.448	31.933	**-4.916**	-79.054	75.019
Max My	78	30	13:GENERATEI	4.83E+3	0.606	-171.860	2.237	**422.815**	-0.179
Min My	66	18	15:GENERATEI	4.83E+3	0.606	171.860	-2.237	**-422.815**	-0.179
Max Mz	127	53	14:GENERATEI	0.663	-374.306	1.652	-0.050	6.634	**777.380**
Min Mz	69	21	14:GENERATEI	4.83E+3	-171.860	-0.606	2.237	-0.179	**-422.815**

REFERENCES

1. **IS 1893(Part 1)(2002)-**Criteria For Earthquake Resistant Design Of Structures Part 1 General Provisions And Buildings (*Fifth Revision)*
2. **IS 456(2000)-**Plain And reinforced Concrete-Code of Practise(Fourth revision)
3. **IS875 (Part1)**-Dead Loads
   (Part2)-Imposed Loads
4. **IS 13920(1993)**-Ductile Detailing Of Reinforced Concrete Structures subjected to Seismic forces-Code Of Practise
5. **"Design Example of a Multistorey Building"**
   Dr. H.J Shah Department of Applied Mechanics M. S. University of Baroda,Vadodara
   Dr. Sudhir K Jain,Department of Civil Engineering Indian Institute of Technology Kanpur (Document No. :: IITK-GSDMA-EQ26-V3.0)
6. **Reinforced concrete Design** (Third Edition) -2009
   S.Unnikrishna Pillai, Devdas Menon : McGraw Hill Education (India) Private Limited
7. **Strength of Materials** (Third Edition)-2008
   S.S Bhavikatti : Vikas Publishing House Pvt Ltd
8. **Mohammad Adil Dar, Prof (Dr) A.R. Dar , Asim Qureshi , Jayalakshmi Raju-**
   "A Study on Earthquake Resistant Construction Techniques"
   American Journal of Engineering Research (AJER)**-2013**
   Volume-02, Issue-12, pp-258-264
9. **Wakchaure M.R, Ped S. P-**"Earthquake Analysis of High Rise Building with and Without In filled Walls"
   International Journal of Engineering and Innovative Technology (IJEIT) Volume 2, Issue 2, August 2012
10. **Merritt, R. G. and Housner, G. W. (1954)-** "Effect of foundation compliance on earthquake stresses in multistory buildings"
    Bulletin of The Seismological Society of America, 44 (4).
11. **Prathibha S and A Meher Prasa**d **(2004)**"Seismic Vulnerability Of Existing Rc Buildings In India"
    13th World Conference on Earthquake Engineering,Vancouver, B.C., Canada,August 1-6, 2004,Paper No. 1207
12. **Jaswant N. Arlekar,Sudhir K Jain and C.V.R Murty (1997**)-"Siesmic Response of RC Frame Building with Soft First Storeys"
    International Journal of Civil, Structural,Environmental and Infrastructure Engineering Research and Development (IJCSEIERD)
    ISSN(P): 2249-6866; ISSN(E): 2249-7978
    Vol. 4, Issue 3, Jun 2014, 35-44
13. **Bora Gencturk and Amr S. Elnashai (2011**)-"Multi-Objective Optimal Seismic Design Of Buildings Using Advanced Engineering Materials"