



Sesmic Analysis and Design of a Shopping Complex Using BIM: Replacement of Different Column Sections

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ABSTRACT

The Sesmic Analysis and Design of the G+7 Shopping Complex Using BIM: Replacement of Various Column Cross Sections is described in this article. The rigidity, strength, and stability of the structure are investigated using structural design. By substituting the Square Cross Section Columns with the Different Cross Section Columns L, T, and PLUS, the primary goal of structural analysis and design is to build a structure in Zone-III capable of withstanding all applied loads without failure for the duration of its planned life. To raise Drift so that Structures Can Survive in Zone III. In the twenty-first century, there is a need for many complicated and irregular structures and designs that can withstand earthquakes and winds. These structures must be designed and analysed using a variety of software programmes, including REVIT and STAAD pro. This job involves the structural analysis and design of a seven-story, G+7 commercial RCC building in seismic zone III. The building's final analysis and design were completed using STAAD Pro, and all of the structure's members were created using the limit state technique with reference to IS: 456-2000. Theoretical calculations were completed using the IS 1893-2016 code. The results of our design and analysis of different building column cross sections in this project are as follows. Based on the soil's secure bearing capacity (SBC), footings are created. In addition, a two-way continuous slab was designed for three instances. finding the reinforcement percentage for the important part. To determine the Time period rebounding phase. Additionally, storey Drift is computed. The structure can be maintained in Zone-III, according to this paper, by replacing the column cross sections with L, T and PLUS to raise drift values.

KEYWORDS: SBC, Rigidity, %of reinforcement, Critical section, Storey drift, Time period, Rebounding.

1. INTRODUCTION

1.1 SIEMIC ZONES

Seismic Zoning Map of a country is not only guiding to the seismic status and susceptibility of a region but also zoning indicates the direction of future work aimed at designing (Krishna, 1992). This review paper discusses the progressive modifications of the national seismic zonation map of India officially by BIS, other individual studies and by the international program like GSHAP. This study also analyzes the systematic development of zonation maps and various methods adopted. Since the first official release of the national seismic zonation map by BIS in 1962, it has been subsequently modified in 1966, 1970, 1984 and 2002 with the occurrences of major devastating earthquakes and availability of new datasets in terms of geological geophysical and tectonic maps.

In IS 1893-1962 and IS 1893-1966		In IS 1893-1970 IS 1893-1975 and IS 1893-1984		In IS 1893-2002	
Seismic zone	Mapped to MMI scale	Seismic zone	Mapped to MMI scale with CIS-64 Scale	Seismic zone	Mapped to Modified scale with CIS-64 Scale
0	Below V				
I	V	I	V and above		
II	VI	II	VI	II	VI and above
III	VII	III	VII	III	VII
IV	VIII	IV	VIII	IV	VIII
V VI	IX	V	IX and above	V	IX and above

	X and above				
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Besides the zoning map of India by the BIS, other non-official seismic hazard maps have been available in literature by various workers (Auden, 1959; Guha, 1962; Kaila & Rao, 1979; Khattri et al., 1984; Bhatia et al., 1999 and Parvez et al., 2003 ;) based on the statistical or probabilistic models.

BACKGROUND OF EARTHQUAKES

India has had a number of the world's greatest earthquakes in the last century. More than 50% area in the country is considered prone to damaging earthquakes. The north-eastern region of the country as well as the entire Himalayan belt is susceptible to great earthquakes of magnitude more than 8.0. The main cause of earthquakes in these regions is due to the movement of the Indian plate towards the Eurasian plate at the rate of about 50 mm per year.

When reviewing the past earthquakes it is important to have the correct perspective on earthquake magnitude and earthquake intensity. Earthquake magnitude is a measure of the size of the earthquake reflecting the elastic energy released by the earthquake. It is referred by a certain real number on the Richter scale (e.g., magnitude 6.5 earthquake). On the other hand, earthquake intensity indicates the extent of shaking experienced at a given location due to a particular earthquake. It is referred by a Roman numeral (e.g., VIII on MSK scale).

Intensity of shaking at a location depends not only on the magnitude of the earthquake, but also on the distance of the site from the earthquake source and the geology / geography of the area. Isoseismals are the contours of equal earthquake intensity. The area that suffers strong shaking and significant damage during an earthquake is termed as meizoseismal region.

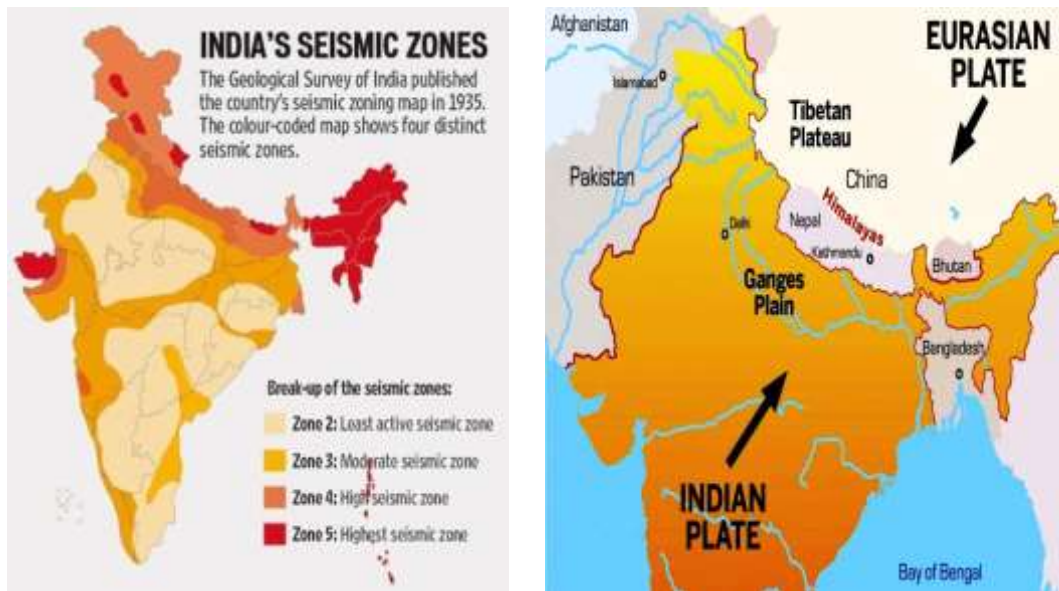


Figure 1.1 RELATIONSHIP BETWEEN SOIL TYPE AND EARTHQUAKE DAMAGE

2. LITERATURE REVIEW

Tiecheng Wang. (2014) In this paper Experimental Study of the Seismic Performance of L-Shaped Columns with 500 MPa Steel Bars, Experimental Study of the Seismic Performance of L-Shaped Columns with 500 MPa Steel Bars, Based on tests on six L-shaped RC columns with 500 MPa steel bars, the effect of axial compression ratios and stirrup spacing on failure mode, bearing capacity, displacement, and curvature ductility of the specimens is investigated. [1].

Tiecheng Wang (2014), In this exploratory study, Based on tests on six L-shaped RC columns with 500 MPa steel bars, the effect of axial compression ratios and stirrup spacing on failure mode, bearing capacity, displacement, and curvature ductility of the specimens is investigated. Test results show that specimens with lower axial load and large stirrup characteristic value (larger than about 0.35) are better at ductility and seismic performance, while specimens under high axial load or with a small stirrup characteristic value (less than about 0.35) are poorer at ductility; L-shaped columns with 500 MPa steel bars show better bearing capacity and ductility in comparison with specimens with HRB400 steel bars. [2].

Ali Hameed Naser Almamoori (2020), The paper presents the results of an experimental investigation to study the behaviour of 19 light weight aggregate concrete-filled steel tubular (CFST) columns subjected to concentric loads. The intermediate CFST columns consisted of 19 different cross-sections. In order to study the most effective section in terms of confinement and stability, All sections were designed to have approximately the same outer perimeter (P), and thus approximately the same cross sectional area, since all sections were manufactured using a mild steel plates with constant thickness. During the experimental tests, the ultimate strength, column shortening, lateral displacement and failure modes were recorded [3].

WANG Tie-cheng (2010), This study proposes A 1/3-scale reinforced concrete (RC) frame of unequal storey height with specially shaped columns was tested under low frequency cyclic loading. The damage characteristic, bearing capacity, deformation capacity and ductility were analyzed. The restoring force model of the frame was obtained based on the study of the hysteresis curve measured in experiment, and the stiffness degeneration characteristics of every storey of the frame were analyzed. [4].

Yu-ye Xu (2009), The purpose of this study is Four full-scale reinforced concrete (RC) columns with L-shaped cross-sections, four full-scale RC columns with T-shaped cross-sections, three full-scale RC columns with +-shaped cross-sections, and one full-scale RC column with a square cross-section were experimentally investigated for fire resistance following the ISO834 standard heating process. The effects of axial load ratio and fire exposure condition on failure mode, axial deformation and fire resistance of the columns were analyzed. The experimental results showed that: (a) when the axial load ratio is 0.55, the fire resistances of the columns with L-, T-, and +-shaped cross-sections subjected to fire on all sides were 60–73% that of the column with the square cross-section. (b) In the case of samples subjected to fire on all sides, the fire resistance of columns with differently-shaped cross-sections increased in the following order: L-shaped cross-section oT-shaped cross-section o+-shaped cross-section. A computer program RCSSCF was developed to calculate temperature, deformation, and fire resistance of the loaded columns with L-, T-, and +-shaped cross-sections. The results of the numerical simulation were compared with those of the full-scale fire resistance tests. [5].

3. METHODOLOGY

IDENTIFICATION OF PROBLEM

There is a remedy for every issue, but first we must recognise the issue. In our situation, we must create an earthquake-resistant building to prevent collapse in the event of an earthquake. However, in this case, the issue is how much weight that zone can support and on what kind of soil. Therefore, we need to determine what zone it is and what kind of dirt is present there.

SELECTION OF PLOT SIZE

After issue identification, plot size should be taken into account. The most important factor is choosing the right size plot because if we develop outside the chosen plot, the relevant corporation department for the zone will cause us a great deal of trouble.

DESIGN OF LAYOUT

We should plan the overall structure's layout after choosing the plot measurement. Because the layout itself inspires creative ideas about how the structure will look after construction. Additionally, it gives the builders a general notion of where to put various rooms.

CREATION OF SECTIONAL VIEW

Draw a section line along the x-axis after the arrangement has been planned. This section line, which provides a sectional view along the axis, aids in viewing the building's different sections.

ANALYSIS OF BUILDING

After the sectional image has been created. We should evaluate the structure using the various weights. Static and dynamic analysis are the two techniques we use to conduct the analysis in our undertaking.

GEOMETRY OF BUILDING

Figure3.2 depicts the building's basic layout. The ground beams are offered 110 millimetres below the surface of the ground.

STOREY NUMBER

The area of the building between two successive beam grids is assigned a storey number. The storey numbers for the example structure are specified as follows:

3.1 OBJECTIVES

- To determine the seismic analysis and design of Shopping Complex structures in zones 3.
- To determine the maximum values of X and Y-direction drift.
- To create and replace various column cross sections based on the position and maximum drift parameters.
- To lower the drift value by offering various column cross sections.

Table 3.1 Number of storey of building

<i>Portion of the Building</i>	<i>Storey number</i>
Ground Floor Beams~ First Floor	1

First Floor ~ Second Floor	2
Second Floor ~ Third Floor	3
Third Floor~ Fourth Floor	4
Fourth Floor~ Fifth Floor	5
Fifth Floor ~ Sixth Floor	6
Six Floor ~ Seventh Floor	7

STATIC ANALYSIS

The equivalent static analysis approach is another name for this technique. In this method, the effects of earthquake ground motion are represented by a number of forces acting on a structure. It is frequently described by a seismic reaction spectrum. It is assumed that the structure will react in its default state. The building must be low-rise and not significantly twist when the ground moves for this to be accurate. Given the natural frequency of the building, a design response spectrum is used to read the reaction (either calculated or defined by the building code). By including factors to take higher buildings with some higher modes and low levels of twisting into consideration, the applicability of this technique is expanded in many building codes. Many codes use modification factors that lower the design forces to take into consideration effects brought on by the structure's "yielding" (e.g. force reduction factors).

PROCEDURE AND CALCULATIONS

Statement/Assumptions

- The building is located in seismic zone III.
- The building consists of main block and service block (stair corner and lifts) connected by a exposure joint and is structurally separated.
- The building will be used for shops (10mx9m) and thickness of brick wall is 230mm

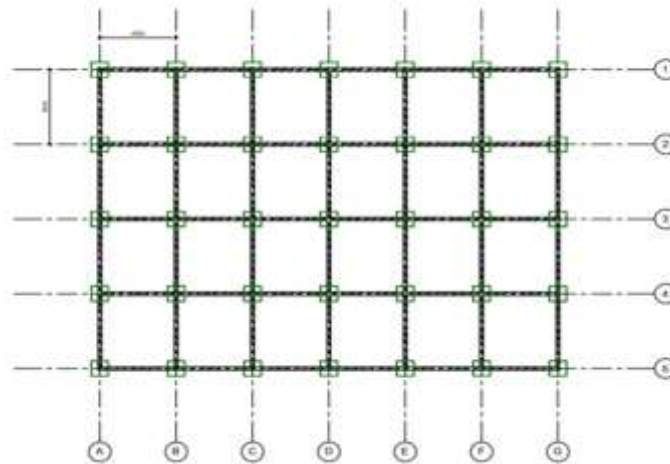


Figure 3.1 Layout of the commercial building

SELF WEIGHTS CALCULATION

Size of column – 1200x1200 mm

Self weight of column = $1.2 \times 1.2 \times 25 = 36$ kN/m Size of beam – 300x450 mm

Self weight of beam = $0.3 \times 0.45 \times 25 = 3.375$ kN/m Thickness of slab – 150mm

Self weight of slab = $0.15 \times 25 = 3.75$ kN/m² Thickness of brick wall – 230mm

Self weight of brick wall = $0.23 \times 20 = 4.6$ kN/m

Floor wall height = $3.1 - 0.45$ (beam depth) = 2.65m

Self weight = $4.6 \times 2.65 = 12.19$ kN/m

Ground floor wall (plinth beam to ground floor roof beam) = $0.7 + (3.1 - 0.45) = 3.35$ m

Self weight = $4.6 \times 3.35 = 15.41 \text{ kN/m}$

Terrace parapet wall (1m height) = $4.6 \times 1 = 4.6 \text{ kN/m}$

SLAB LOAD CALCULATION

Live load = 4 kN/m^2 Terrace = 1.5 kN/m^2 Floor finish = 1 kN/m^2

Water proofing = 2 kN/m^2

Self weight of slab = $0.15 \times 25 = 3.75 \text{ kN/m}^2$

TERRACE SLAB

Total dead load = $3.75 + 2 + 1 = 6.75 \text{ kN/m}^2$ Total live load = 0 kN/m^2 (As per IS 1893-2016)

Total load = $6.75 + 0 = 8.25 \text{ kN/m}^2$

OTHER FLOORS

Dead load = $3.35 + 1 = 4.35 \text{ kN/m}^2$ Live load = 4 kN/m^2

Total load = $4.35 + 4 = 8.75 \text{ kN/m}^2$

BEAM LOAD CALCULATION

LOAD = $0.3 \times 0.45 \times 25 = 3.375 \text{ kN/m}$

SEISMIC WEIGHT CALCULATION

Following reduced line loads are used for analysis. 0% on terrace, 50% on other floors (IS1893 part I: 2002, clause 7.4)

TERRACE

Slab = $6.75 \times 27 \times 20 = 3645 \text{ kN}$

Parapet = $4.6 \times [27 + 27 + 20 + 20] = 432.4 \text{ kN}$ Beams = $3.375 \times [(5 \times 27) + (7 \times 20)] = 928.125 \text{ kN}$

Walls = $\frac{1}{2} \times 12.19 \times [(3 \times 27) + (4 \times 20)] = 981.295 \text{ kN}$

Columns = $\frac{1}{2} \times 3.5 \times 36 \times 35 = 2205 \text{ kN}$

Total = $3645 + 432.4 + 928.125 + 981.295 + 2205 = 8191.82 \text{ kN}$

STOREY-2

Slab = 3645 kN

Walls = $\frac{1}{2} \times 12.19 \times 161 = 981.295 \text{ kN}$ Walls = $\frac{1}{2} \times 4.6 \times 3.35 \times 161 = 1240.50 \text{ kN}$ Beam = 928.125 kN

Column = $\frac{1}{2} \times 36 \times 35 \times [3.1 + 3.8] = 4347 \text{ kN}$

Total load = 11141.92 kN

STOREY-1

Walls = $\frac{1}{2} \times 4.6 \times 170 \times 3.75 = 1466.25 \text{ kN}$ Beams = 978.75 kN

Columns = $\frac{1}{2} \times 36 \times 35 \times [3.8 + 1.1] = 579.7 \text{ kN}$ Total load = 3087 kN

OTHER STOREYES (EXCEPT 1,2 FLOOR AND TERRACE)

From slab = $3.75 + 1 + (0.5 \times 4) = 6.75 \text{ kN/m}^2$ Slab = $6.75 \times 27 \times 20 = 3645 \text{ kN}$

Walls = $14.03 \times [(3 \times 27) + (4 \times 20)] = 2385 \text{ kN}$

Beams = 928.125 kN

Columns = $3.1 \times 36 \times 35 = 3906 \text{ kN}$

Total load = $4050 + 978.75 + 2385 + 3906 = 11319.75 \text{ kN}$

DESIGN OF SEISMIC LOAD

$T_a = 0.075h^{0.75}$ [IS 1893 PART 1 : 2016, clause 7.6.2]

$T_a = 0.075(26.6+1)^{0.75} = 0.878\text{sec}$

Zone factor $Z = 0.16$ for zone III [IS 1893 PART 1;2016, ANNEXE E]

$R = 5$ (for SMRF)

Importance factor $I = 1.5$ (public building) [IS 1893 PART 1;2016, clause 7.2.3 Table-8] Medium soil site and 5% damping [IS 1893 PART 1;2016, clause 7.2.6 Table-9] $S_a/g = 1.36/T$ (for $0.35 \leq T \leq 4$)

Medium soil site and 5% damping

$$S_a/g = 1.36/T \text{ (for } 0.35 \leq T \leq 4)$$

$$S_a/g = 1.36/0.878 = 1.548$$

$$A_h = \frac{Z(S_a/2)}{R/I}$$

$$A_h = \frac{(0.16)(1.78)}{5/1.5} = 0.1238/3.333 = 0.0371$$

CALCULATION OF BASE SHEAR

$V_B = A_h \times W$ [IS 1893 PART 1;2016, clause 7.6.1 Table-9]

W = seismic weight of entire building

$W = 8191.82 + 11141.92 + 3087 + (11319.75 \times 6) = 90339.24\text{kN}$ $V_B = 0.0371 \times 90339.24 = 3351.585\text{KN}$

DYNAMIC ANALYSIS

One of the efficient methods for assessing the building's earthquake performance is dynamic analysis. Software like ETABS and STAAD Pro, two of the most popular programmes currently used by businesses and structural engineers for their projects, can perform dynamic analysis. Because of its ease of use, we are considering the staad pro software for our endeavour. Linear and nonlinear dynamic analysis are the two subtypes of dynamic analysis.

LINEAR DYNAMIC ANALYSIS

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsion irregularities, or non-orthogonal systems, a dynamic procedure is required. In the linear dynamic procedure, the building is modeled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix.

BUILDING DATA:

Table 3.3 Building data parameters

PARAMETERS	MODEL
Soil type	Medium
Zone	III
No. of storey	G+7
Response reduction factor	5
Importance factor	1
Height of building	26.6
Outer beam size	450x300 mm
Inner beam size	450x300 mm
Column size	1200x1200 mm

Grade of concrete for beams	M20
Grade of concrete for footings	M25
Grade of concrete for columns	M30
Grade of steel	Fe 415

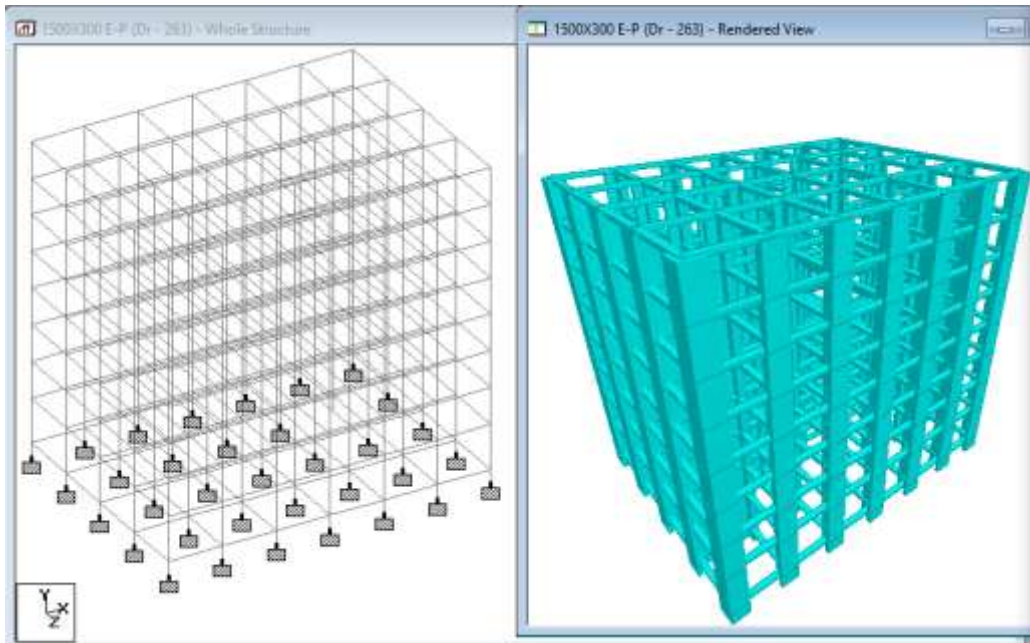


Figure 3.3 Isometric view of the G+7 And 3D Rendered commercial building

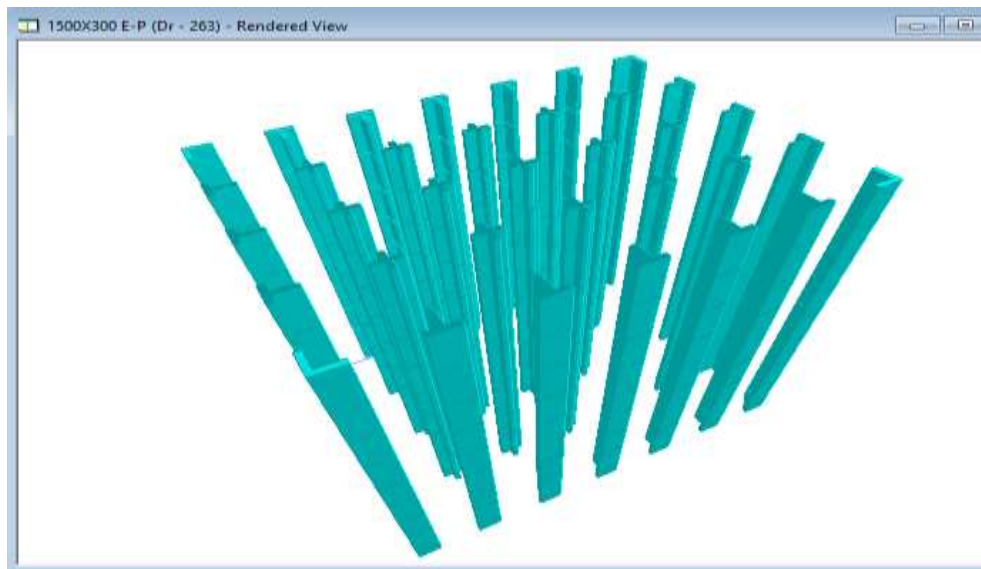


Figure 3.4 3D Rendered view of Columns L,P, PLUS the G+5 commercial building

DESIGN OF TRAPEZOIDAL FOOTING (SLOPED FOOTING):

Taking the column, no -18, the critical column Size of the column = 600*400 mm

Factored axial load $P_u = 3280.17 \text{ KN/m}^2$

M25 & Fe 415

• DESIGN OF SIZE OF FOOTING

$$P_u = 3280.17 \text{ KN}$$

Assume self-weight of footing = 10% of super imposed load

$$W_D = 10\% \text{ of } 3280.17 \text{ KN } W_D = 328 \text{ KN}$$

Total load, $P = P_u + W_D$

$$P = 3608.18 \text{ KN}$$

$$\text{Area of footing} = A = \frac{P}{\text{SBC}} = \frac{3608.18}{250}$$

$$A = 14.43 \text{ m}^2$$

Column size = 600*400mm Difference in dimension = 600 – 400

= 200 mm $b = \text{width}$

$$L = b + 0.2,$$

• Area of Rectangle(A) = L * B

$$(b) * (b+0.2) = \text{Area } b^2 + 0.2 b - 14.43 = 0 \quad b = 3.9$$

$$L = b + 0.2 = 4.1 \text{ m}$$

Length = 4.1 m Breadth = 3.9 m

• UPWARD PRESSURE

Net upward pressure = $\frac{\text{factored axial load}}{\text{area of footing}}$

$$= \frac{3608.2}{14.43}$$

$$= 250 \text{ KN/m}^2$$

BENDING MOMENT

UDL along X – direction

$$M_x = \text{net upward pressure} * 3.9$$

$$= 250 * 3.9$$

$$= 975 \text{ KN/m}$$

$$= 1.5 * 975 \text{ KN/m}$$

$$M_x = 1462.5 \text{ KN/m}$$

UDL along Y – direction $M_y = \text{Net upward pressure} * 4.1$

$$= 250 * 4.1$$

$$= 1025 \text{ KN/m}$$

$$L = 4100/2 - 600/2$$

$$L = 1750 \text{ mm} = 1.75 \text{ m}$$

$$M_y = \frac{wL^2}{2} = \frac{975 * (1.75)^2}{2}$$

$$= 1493 \text{ KN/m}$$

$$M_x = \frac{wL^2}{2} = \frac{1025 * (1.75)^2}{2}$$

$$= 1569.5 \text{ KN/m}$$

- **DEPTH OF FOOTING**

Depth along X- direction

$$M_{ux} = 0.138 f_{ck} b d_x^2 \quad b_x = \text{Width of resisting section}$$

$$1462.5 * 10^6 = 0.138 * 25 * 750 * d_x^2 \quad = 600 + 150 \quad d_x^2 = 565217.39 \text{ mm}$$

$$d_x = 751.8$$

$d_x = 752 \text{ mm}$ Depth along Y- direction

$$M_{uy} = 0.138 f_{ck} b d_y^2 \quad b_y = 400 + 75 + 75$$

$$1537.5 * 10^6 = 0.138 * 25 * 550 * d_y^2 \quad = 550 \text{ mm} \quad d_y^2 = 810276.6798 \text{ mm}$$

$$d_y = 900 \text{ mm}$$

For overall depth = 1000 mm

$$d_y = 1000 - 50 - \frac{12}{2} = 944 \text{ mm}$$

$$d_x = 1000 - 50 - \phi/2 - \phi$$

$$= 932 \text{ mm}$$

$$\text{Average depth} = \frac{944 + 932}{2} = 938 \text{ mm}$$

- **REINFORCEMENT**

Along X – direction

$$M_{uy} = 1537.5 * 10^6 \text{ N-mm and } b_y = 550 \text{ mm, } d_y = 944 \text{ mm } P_t = 0.85 \%$$

$$A_{st1} = \frac{0.85}{100} * 550 * 944$$

$$= 4413.2 \text{ mm}^2$$

$$\text{Diameter of the bars} = 12 \text{ mm of bars } A_{st} = \frac{\pi}{4} * (12)^2 = 113.09 \text{ mm}^2$$

$$\text{No of bars} = \frac{A_{st1}}{A_{st}} = \frac{4413.2}{113.1} = 39 \text{ Bars} \quad b_y = 750 \text{ mm}$$

Reinforcement along Y- direction $d_x = 932 \text{ mm } M_{ux} = 1462.5 * 10^6 \text{ N.mm}$

$$P_t = 0.47 \%$$

$$A_{st2} = \frac{0.47}{100} * 750 * 932$$

$$A_{st2} = 3285.3 \text{ mm}^2$$

$$A_{st} = 113.1 \text{ mm}^2$$

$$\text{No of bars} = \frac{A_{st2}}{A_{st}} = \frac{3285.3}{113.1}$$

$$= 29 \text{ bars}$$

$$A_{st} \text{ provided} = 29 * 113.1$$

$$= 3279.9 \text{ mm}^2$$

$$= 3280 \text{ mm}^2$$

- **Check for spacing**

Spacing along Y- direction

From *IS 456, Pg 46 -T- 15 (cl 26.3.3)*

$$\text{Clear spacing} = 4100 - 50 - 50 - \frac{1}{2} \phi - \frac{1}{2} \phi$$

$$= 4100 - 50 - 50 - \frac{1}{2} \cdot 12 - \frac{1}{2} \cdot 12$$

$$\frac{3988}{\text{no of bars}} = \frac{3988}{29-1}$$

$$= 142.4 \text{ mm}$$

$$\text{Clear spacing} = 142.4 - \frac{12}{2} - \frac{12}{2}$$

$$= 130.4 \text{ mm} \leq 180 \text{ mm}$$

Spacing along X- direction

Width = 3900mm & 39 bars

$$\text{C-C spacing} = \frac{3900 - 50 - 50 - 6 - 6}{\text{no of bars} - 1}$$

$$= \frac{3788}{39-1}$$

$$= 99.6 \text{ mm}$$

C-C spacing = 100 mm

$$\text{clear spacing} = 100 - \frac{12}{2} - \frac{12}{2}$$

$$88 \text{ mm} \leq 180 \text{ mm}$$

• CHECK FOR ONE WAY SHEAR

Along Y – direction

At distance “d” from force of column Total depth = 1000 mm

Edge depth = 230 mm

$$= 1000 - 230 \text{ mm}$$

$$= 770 \text{ mm}$$

y1 = depth @ critical section

$$1675 \longrightarrow 770$$

$$731 \longrightarrow ? (y1)$$

$$y1 = 336 \text{ mm}$$

d1 = effective depth

$$d1 = [336 - 230] - [50 - 12/2]$$

$$= 522 \text{ mm}$$

$$b1 = 400 + 2d = 400 + 2(944)$$

$$b1 = 2288 \text{ mm}$$

VU = SHEAR @ CRITICAL SECTION

= Upward pressure * (Area of highlight)

$$= 250 * (0.73 * 3.9)$$

$$= 712.72 \text{ KN}$$

RELATION BETWEEN $\tau_v < \tau_c$

IS 456:2000, P-72

$$\tau_v = \frac{\tau_u - \mu \tan \beta}{d_1}$$

$$\tan \beta = \frac{770}{1675} = 0.45$$

M_u = at critical section

$$= \frac{1462.5 * 0.731 * 0.731}{2}$$

$$= 390.75 \text{ KN/m}$$

$$\tau_v = \frac{[712.8 - \frac{390.8}{0.522} * 0.45] * 10^3}{2288 * 522}$$

$$\tau_v = 0.314 \text{ N/mm}^2$$

$$100 * 3280$$

$$\tau_c = \tau_{pt} = 2288 * 522 \longrightarrow \text{Ast provided} \quad \therefore [\tau_{pt} = \frac{\quad}{bd}]$$

$$\tau_c = 0.274 \%$$

As per table – 19 IS 456

$$\tau_c = 0.25 \text{ } 0.36$$

$$0.27 \dots \dots \dots ?$$

$$0.50 \dots \dots \dots 0.49$$

$$\tau_c = 0.3704 \text{ N/mm}^2$$

$$\tau_v < \tau_c$$

CHECK FOR TWO WAY SHEAR

At 1675 mm 770 mm

$$1206 \text{ mm } ? (y_1)$$

$$y^1 = 554.4 \text{ mm}$$

$$d^1 = (554.4 + 230) - 50 = 12 - \phi / 2$$

$$d^1 = 716.4 \text{ mm}$$

LENGTH OF PERIPHERY,

$$(600 + 469 + 469) + (400 + 469 + 469)$$

$$b_o = 2876 * 2 = 5752 \text{ mm } V_u = @ \text{ critical section}$$

= Upward pressure * Area of highlighted section

$$= 250 * [4.1 * 3.9 - (1.53 * 1.33)]$$

$$= 3488.7 \text{ KN}$$

$$\tau_v = \frac{V_u}{b_o d_1} = \frac{3488.7 * 10^3}{5752 * 716.4}$$

$$= 0.84 \text{ N/mm}^2$$

$$\tau_v = \tau_c = K_s * \tau_c$$

$$K_s = 0.5 + \beta_c \quad (\text{IS 456:2000 Pg 58, 59})$$

$$K_s = 0.5 + 400 = 1.16$$

$$600$$

$$\tau_c = 0.25 \sqrt{f_{ck}} \quad \tau_c = 0.25 \sqrt{25} \quad \tau_c = 1.25$$

$$\tau_1 = 1.16 * 1.25$$

$$\tau_1 = 1.45 \text{ N/mm}^2$$

$$\tau_v < \tau_1 \quad (\text{Two way shear is Ok})$$

Therefore, the design of the foundation satisfies both the one way shear and two way shear, so it could be safe for taking into consideration.

4. Results and Discussions

BASE SHEAR

Base shear is an estimate of the maximum expected lateral force on the base of the structure due to seismic activity. It is calculated using the seismic zone, soil material, and building code lateral force equations. We should always consider the value obtained by the analysis through staad pro.

Table 4.3 Base shear values

Type of analysis	Base shear (KN)
Static analysis	3351.588
Dynamic analysis	2750

TIME PERIOD

Time Period is the amount of time needed for a building's displacement to recover after an earthquake. Since the longer it takes, the greater the likelihood of the structure collapsing, this period of time should be as brief as possible. Its units should always be in seconds. The time period's number is obtained. Table 4.4 Values for the time period

Time Period	0.878 sec
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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. From the frame analysis the displacements of the mass centers of various floors are obtained and are shown in Table 4.5 along with storey drift. Since the building configuration is same in both the directions, the displacement values are same in either direction.

Table 4.5 Storey drift values

STOREY	HEIGHT(m)	DISPLACEMENT (mm)	DRIFT (mm)
0 (Footing top)	0	0	0
1 (Below plinth)	1.1	0.267	0.3
2 (Ground floor)	4.9	4.812	4.545
3 (First floor)	8	11.75	6.942
4 (Second floor)	11.1	20.78	9.029
5 (Third floor)	14.2	31.21	10.43
6 (Fourth floor)	17.3	42.44	11.233
7 (Fifth floor)	20.4	53.99	11.554
8 (Sixth floor)	23.5	65.55	11.553
9 (Seventh floor)	26.6	76.98	11.428

Maximum drift is for second storey = 11.554 mm. Maximum drift permitted = $0.004 \times 20400 = 81.6$ mm. Hence, ok. Sometimes it may so happen that the requirement of storey drift is not satisfied. However, as per Clause 7.11.1, IS: 1893 (Part 1): 2002; "For the purpose of displacement requirements only, it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force." In such cases one may check storey drifts by using the relatively lower magnitude seismic forces obtained from a dynamic analysis.

5. CONCLUSION

In the present study, G+7 Commercial building has been drawn in Auto CAD software and designed (Beams, Columns, Footings and Seismic load analysis by using Equivalent Static method) using STAAD Pro software. The dead load, live load and earthquake loads are calculated using IS: 456-2000 and IS 1893: 2016. Concrete grade M20 and HYSD bars Fe415 are used. From the analysis done in staad pro. We can conclude the following.

- The design base shear (V_b) occurred manually = 3351.58kN
- The design base shear (V_b) by using STAAD pro = 2750kN
- Maximum support reactions of footings = 3040kN
- It experiences static as well as dynamic analysis of the structure and gives accurate results which are required. The following points have been obtained at the end of the design.
- The time period by using STAAD calculation is 0.875 seconds which is safe for earthquake
- In the slab design the deflection check is proved to be safe and it is an economic section.
- The footing design has passed the one-way shear check and two-way shear check and the type of footing is trapezoidal which is economical.
- To conclude, STADD. Pro is versatile software having the ability to determine the reinforcement required for any concrete section based on its loading and determine the nodal deflections against lateral forces.
- The proposed commercial RCC building has been analysed and designed. All the results of design and analysis are found to be safe. The limit state method of design is used for the design of all the components So, the designed RCC building is an earthquake resistant structure.

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