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Numerical Modeling, Analysis and Structural Design of G+4 RC Building with Re-Entrant Corners

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ABSTRACT---- ***The architectural design of structures like schools, offices, and hotels often features asymmetry and plan irregularities, demanding careful consideration during seismic analysis and structural design. Re-entrant corners with certain conditions in a building are considered a horizontal irregularity by seismic codes such as IS 1893 (Part I):2002. This study focuses on the analysis and design of a multi-storey office building situated in Zone V with a re-entrant corner horizontal irregularity. It encompasses the preliminary design of structural components and seismic analysis using a numerical model based on IS 1893 (Part I):2002, implemented in ETABS v20.0.0. Key responses, including time period, modal mass participation ratios, and storey drift ratio, are assessed to ensure compliance with codal requirements. Following this, structural design for beams, columns, and slabs is executed, and the required longitudinal and transverse reinforcement in different locations of these components is compared across various floors.***

Keywords: Re-entrant corner; Horizontal irregularity; Equivalent static method; Response spectrum method

1. INTRODUCTION

In contemporary architecture, intricate designs present significant challenges for structural designers due to asymmetries and irregularities. IS 1893(Part I):2002 defines various irregularities in building plans and elevation, emphasizing the adverse impact of plan irregularities, which cannot be ignored. In extreme cases, plan irregularities can induce torsional effects, potentially leading to the complete collapse of a building. The 2015 Gorkha Seismic Sequence attributed structural damage to inadequate seismic provisions and material deficiencies, affecting 755,549 buildings in Nepal [1]. Proper seismic-resistant design not only saves lives but also reduces the economic impact of earthquakes by minimizing damage to structures.

Horizontal irregularities, resulting from factors like re-entrant corners, diaphragm discontinuity, and out-of-plane offset of lateral load-resisting elements, pose specific challenges. Re-entrant corner irregularity is commonly observed in modern architectural designs, offering aesthetic and functional benefits but presenting unique challenges for structural engineers, especially in earthquake-prone regions. Seismic analysis for structures in Zone V is crucial, with structural demands increasing with the zone factor [2].

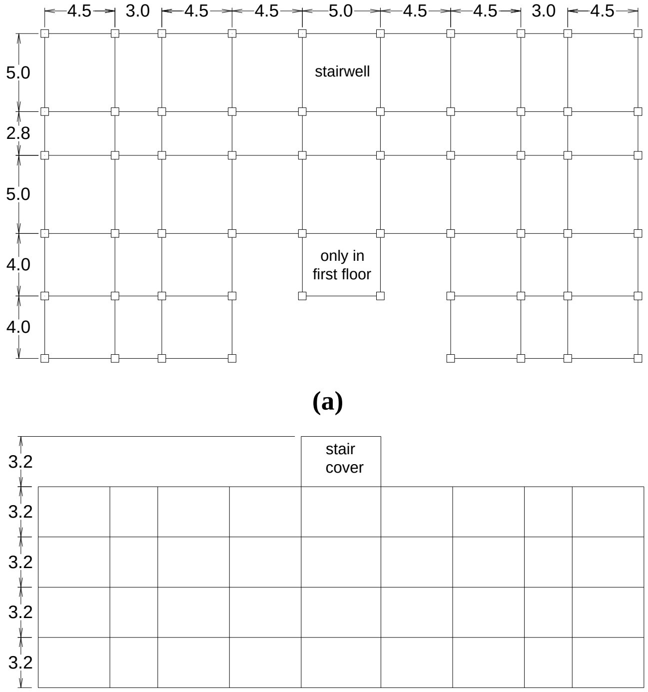
While extensive research exists on RC building seismic performance, studies specifically focusing on re-entrant corners are limited. A parametric study of buildings with different plan shapes, such as L, H, and U, indicated that a lateral length ratio equal to unity is the most stable configuration [3]. Current design codes often lack specific guidance for such irregularities, leading to conservative and potentially inefficient designs. A study found that the Equivalent Static Method (ESM) can be safely used for up to 12-storey buildings with any degree of re-entrant corner irregularity [4]. However, IS 1893(Part I):2002 limits it to 12m in Zones IV and V and 40m in Zones II and III above which the Response Spectrum Method (RSM) needs to be performed. This research aims to bridge these gaps by comprehensively analyzing and designing a G+4 RC building with re-entrant corners using ETABS v20.0.0 software.

Utilizing both ESM and RSM as provisioned in (IS 1893(Part I):2002 Criteria for Earthquake Resistant Design of Structures. General Provisions and Buildings, 2002), the study focuses on seismic response, emphasizing key aspects like time period, modal mass participation ratios, storey drift, storey shear, and peak reinforcement requirements for beams, columns, and slabs. Emphasis will be placed on implementing ductile detailing requirements as per IS 13920:2016 such as column/beam capacity ratios (CBCR) to enhance energy dissipation and improve earthquake resilience. The design of reinforced concrete structural components like beams, columns, and slabs is carried out as per IS 456:2000. This research is expected to significantly contribute to the field by providing valuable insights into the seismic behavior of re-entrant corner buildings, developing a robust analytical framework for future design projects, optimizing material usage while ensuring safety and serviceability, and proposing recommendations for incorporating specific provisions for such irregularities in building codes.

2. METHODOLOGY

2.1 Description of building

The subject of this study is a five-storey office building designed with a reinforced concrete frame structural system. The building's architectural and functional specifications include a total height of 16.750 meters, consisting of floor heights of 3.15 meters each. The external walls, as well as some internal walls, are 230 mm thick, while a few internal walls measure 110 mm. The plaster thickness is specified at 12 mm, and the plinth area is 655 m^2. There is a balcony on the first floor only and a stair cover on the roof floor as illustrated in Figure 1.



(a)

(b)

Fig -1: Building (a) Plan (b) Front Elevation

2.2 Idealization for analysis

The general procedure to design a building is shown in Figure 2. Prior to modeling, the process of idealization is paramount in simplifying the geometry and behavior of a building into an analytical model. The idealization encompasses various aspects, including loads, and structural elements such as members, joints, and restraints.

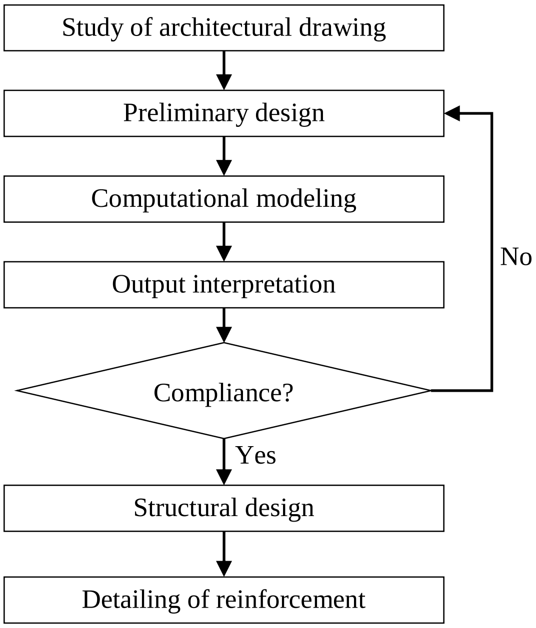


Fig -2: Flowchart of seismic analysis and structural design

The base fixity of the structure at the foundation is idealized as perfectly fixed. This assumption eliminates the need to consider the complex interaction between the soil and the structure. The foundation fixity ensures a stable base for subsequent analyses. Floor and roof slabs are modeled as thin shells by IS 1893 (Part I):2002, provisions for RC monolithic slab-beam floors [5]. The floor diaphragm is treated as rigid. Additionally, as the staircase is not integrated into the structural system, an equivalent load is applied to the supporting beams to simplify the model and avoid associated complications. Beams and columns are idealized as linear elements in 3D, simplifying their behavior for analysis. Joints are assumed to be perfectly rigid, contributing to the overall stability of the structural system. Despite the main beam being integrally cast with the slab, it is modeled as a rectangular section for simplicity. After elemental-level idealization, the entire structural system is conceptualized as an unbraced space frame. This theoretical approximation serves as the basis for first-order linear analysis and subsequent design considerations. Loads are explicitly modeled into different load cases and combinations, aligning with the codal recommended a code. This representation ensures that the analysis captures the diverse loading scenarios the structure may encounter. The accuracy of the analysis pivots on the precision of this idealization process, which aims to strike a balance between simplicity and a faithful representation of the building's behavior under various conditions.

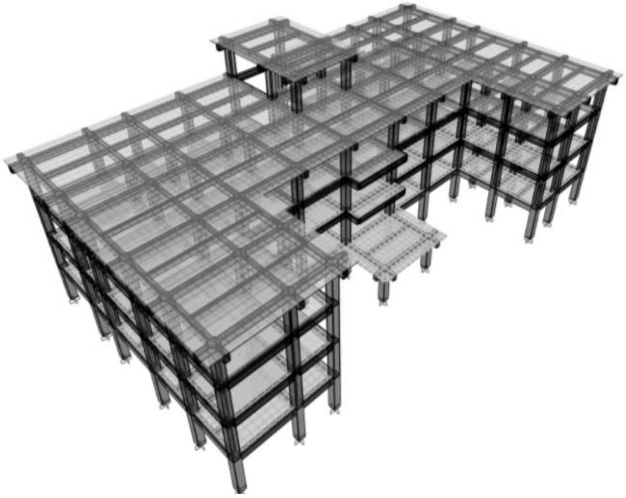


Fig -3: Three dimension building model

2.3 Preliminary Sizing of Structural Members

In preliminary design, the tentative size of structural elements is estimated such that after the analysis the required size shall not deviate considerably, thus, making the design not only safe but economical too.

The size of the slab is approximated using deflection control criteria for the critical slab (slab having the longest shorter dimension) as per IS 456:2000 Clause 23.2.1. For the basic permissible value of effective length (l) to effective depth (d) of 20, and assumed combined modification factors to be around 1.2, the required effective depth of slab is given by,

$$\mathrm{d} \geq \frac{\mathrm{l}}{20\times1.2} \geq \frac{5000}{24} \geq 208 \text{ mm}$$

To reduce the depth, secondary beams are provided. After such provision, the critical slab dimension was reduced to 2800 mm. Thus, the effective depth of the slab is reduced to around 100 mm. Allowing 20 mm clear cover and 10 mm for rebar, the effective cover shall be 25 mm. Thus, the overall depth of the slab becomes 125 mm.

Similarly, the span of longest beam of the building is 5000 mm. For the basic permissible value of effective length to the effective depth of 20, and assumed combined modification factors to be around 0.6, the required effective depth of the main beam is given by,

$$\mathrm{d} \geq \frac{\mathrm{l}}{20\times0.6} \geq \frac{5000}{12} \geq 416.67 \text{ mm}$$

Allowing 30 mm clear cover, 10 mm stirrups, and 20 mm main reinforcement, the effective cover becomes 50 mm. Thus, the overall depth of the beam can be adopted as 500 mm. The width of the beam is taken as 300 mm. The adopted size also complies with IS 13920:2016 which requires the overall depth to be less than one-fourth of the clear span of the shortest beam which is 700 mm. The size of the secondary beam and edge beam is simply adopted as 200 x 350 mm and 150 x 500 mm respectively.

The preliminary design for the column is carried out by considering the tentative load for any interior column around the stairwell. The live load (after 40% reduction as per IS 875:1987 Clause 3.2.1) and dead load acting on the column are computed to be around 104.62 kN and 886.17 kN. The total load is factored by 1.5 times twice to account for design value and earthquake amplification, which computes to be 2229.28 kN. For the square column, 1% assumed percentage of Fe500 steel reinforcement, and M25 concrete, the size of the column using capacity of axially load short column as per IS 456:2000 Clause 39.3 computes to be 410.20 mm. Hence, the size of 500 x 500 mm is adopted.

3. SEISMIC ANALYSIS

In seismic analysis, several parameters and assumptions shape the analytical framework.

3.1 Ground Motion Characteristics

The zone factor which is an estimate of effective peak ground acceleration (PGA) is taken at 0.36 considering the very severe seismic intensity in seismic zone V. The PGA is scaled down to half to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE). The response spectra as given in Figure 2 of IS 1893(Part I):2002 in the form of average response acceleration coefficient for Type II (medium) soil is adopted as shown in Figure 4.

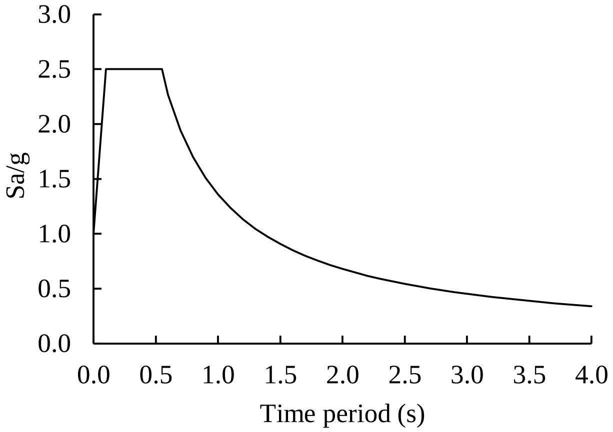


Fig -4: Response spectra for medium soil for 5 percent damping

3.2 Building Configuration

The configuration of the building undergoes a thorough evaluation based on criteria outlined in IS 1893 (Part I):2002 Table 4 and Table 5. The building plan is analyzed for irregularities, with a focus on factors such as drift and eccentricities. The maximum drift, calculated per IS 1893 (Part I) Clause 7.9.2, consistently remains well below 1.2 in both X and Y directions at all levels, with the highest value being 1.17. Frontal projections beyond re-entrant corners are 38% of plan dimension of the building in the same direction, and the floor diaphragm exhibits a 4% opening, exclusively for the stairwell. No out-of-plane offset of vertical elements is observed, and columns align parallel to the major orthogonal axis. Despite these conformities, the presence of re-entrant projections greater than 15% designates the building as irregular in plan.

An assessment of vertical irregularities reveals compliance with specified checks. The soft storey analysis confirms that each storey possesses over 70% lateral stiffness compared to its immediate upper storey and over 80% of the average lateral stiffness of the three stories above. The seismic weight of each storey remains below 200% of that of its adjacent storey, and maximum storey drifts in both X and Y directions are limited to 0.002. The building successfully meets all criteria for not having vertical irregularity.

Due to the identified re-entrant corner irregularity, the building is categorized as irregular. Following IS 1893(Part I):2002 Clause 7.8, linear dynamic analysis by RSM is employed for the subsequent analysis and design phases.

3.3 Seismic Analysis Method

Linear static analysis by ESM is primarily employed to obtain the design lateral seismic force on the building. But ESM incorporates the effect of the fundamental mode of vibration only, which is why, to consider other modes, RSM is also used. Furthermore, as per the IS 1893 (Part I):2002 Clause 7.8.1, all irregular buildings higher than 12 m in Zones IV and V need to be assessed by dynamic analysis. Thus, due to re-entrant corner irregularity, RSM is adopted for dynamic analysis using the design spectrum specified in 1893(Part I):2002 Clause 6.4.2.

3.4 Load intensity

Dead loads for different components of the building and imposed loads for different spaces for the Office Building are taken from IS 875:1987 Table 1 whose intensities are shown in Table 1. Further, these loads are used to determine the seismic weight of the building based on IS 1893(Part I):2002 with some percentage reduction as prescribed by the code.

3.5 Load cases and load combinations

Different load cases are combined taking various factors to obtain the most realistic and critical element stress in the structure in the course of analysis. Four major types of load cases are considered.

Dead load (DL) includes the self-weight of structure, weight of the masonry wall, floor finish, and water tank. Live load (LL) includes the imposed load due to occupants and furnishings. The lateral load in ESM is Earthquake load (EL) and that in RSM is Response Spectrum (RS) load. The various load combinations as per IS 1893(Part I):2002 are shown in Table 2. The four expressions in ESM lead to 13 different combinations as positive and negative earthquake load is taken in both X and Y directions. The three expressions in RSM lead to 6 different combinations as RS is applied in both the X and Y directions.

Table -1: Load intensity

SN	**Item**	**Load intensity(kN/m^3)**
Dead load		
1	RCC	25 kN/m^3
2	Floor finish	1.6 kN/m^2
3	Full Brick thick wall (30% opening deduction)	8.50 kN/m
4	Half Brick thick wall	4.30 kN/m
5	Parapet wall	4.74 kN/m
Live load		
6	General use rooms	2.5 kN/m^2
7	Corridors and staircase	4 kN/m^2
8	Balconies	4 kN/m^2
9	Accessible roof	1.5 kN/m^2
10	Inaccessible roof	0.75 kN/m^2

Table -2: Load Cases and Combinations

SN	ESM	RSM
1	1.5 (DL + LL)	1.2 (DL + LL + RS)
2	1.2 (DL + LL ± EL)	1.5 (DL + RS)
3	1.5 (DL ± EL)	0.9 DL + 1.5 RS
4	0.9 DL ± 1.5 EL	

3.6 Design Seismic Load

According to IS 1893(Part I):2002 Clause 7.5.3, the design seismic load is equal to the product of the design horizontal seismic coefficient (A_h) estimated according to IS 1893(Part I):2002 Clause 6.4.2 and the seismic weight of the building (W) estimated according to IS 1893(Part I):2002 Clause 7.4.1. The importance factor considering post-earthquake functional needs is taken as 1.5. The response reduction factor due to ductile deformation or frictional energy dissipation in the cracks is explicitly taken as 5 from IS 1893(Part I):2002 Table 7 for Special Moment Resisting Frame (SMRF) building. While computing the seismic weight of each floor, the weight of columns and walls in any storey is equally distributed to the floors above and below the storey. The seismic weight of the whole building is the sum of the seismic weights of all the floors. The imposed load on roof is not considered for the calculation of the design seismic forces of the structure as stated by IS 1893(Part I):2002 Clause 7.3.2. The design base shear (V_B) computed above is distributed along the height of the building as per IS 1893(Part I):2002 Clause 7.7.1 as shown in Table 4.

3.7 Scaling of Base Shear for RSA

As per IS 1893(Part I):2002 Clause 7.8.2, the design base shear in dynamic analysis must be equal to or greater than that in static analysis. The initial value of scale factor in RSM is assigned as Ig/2R for ETABS v20.0.0. After obtaining the base shear for initial scale factor, the final scale factor is computed as shown in Table-5 and peak response quantities are combined using Complete Quadratic Combination (CQC) method.

The peak response quantities are combined as per Complete Quadratic Combination (CQC) method as provided in 1893(Part I):2002 Clause 7.8.4.4.

Table -3: Parameters for estimation of base shear

Item	**Value**
Seismic zone	V
Zone factor (Z)	0.36
Height of building (h)	15.750 m
Time period (T_a)	0.593 s
Average response acceleration coefficient (S_a/g)	2.293
Importance Factor (I)	1.5
Response reduction factor (R)	5
Design horizontal seismic coefficient (A_h)	0.124
Type of soil	Medium soil
Response spectra	IS 1893(part I):2002, ζ=5%
Seismic weight (W)	34950 kN
Base Shear (V_B)	4334 kN

Table -4: Distribution of base shear across the height of the building

Storey (i)	**W_i (kN)**	**H_i (m)**	**$W_ih_i^2$ (in 1000s)**	**$Q_i = V_B \times \frac{W_ih_i^2}{\sum W_ih_i^2}$ (kN)**	**V_i (kN)**
5	562	15.75	139.52	237.45	227.00
4	7391	12.60	1173.45	1997.08	1965.00
3	8747	9.45	781.13	1329.40	3391.00
2	9117	6.30	361.85	615.84	4036.00
1	9132	3.15	90.62	154.22	4204.00
Base					4204.00
		$\Sigma W_ih_i^2$	2546.57		

Table -5: Scale factor for RSM

Initial value of scale factor	1.5*9810/(2*5) = 1471.5	
Direction	X	Y
V_B without scaling	3737.22	3739.61
$\overline{V_B}$ i.e. using fundamental period (T_a)	4204.37	4204.37
Scale Factor	1.125	1.095

4. STRUCTURAL DESIGN

4.1 Design Philosophy

Within the realm of structural design philosophies, the Limit State Method (LSM) stands out as an advanced approach that precisely addresses both the ordinary and extreme conditions affecting a structure. The Ultimate Limit State (ULS), often referred to as the limit state of collapse, represents the peak stress scenario induced by design loading just before impending collapse, beyond which the structure can no longer withstand additional loads. On the other hand, the Serviceability Limit State (SLS) characterizes the standard stress scenario resulting from service loads. Beyond this point, the structure becomes unsuitable for occupancy due to excessive deflection, cracking, or vibration, yet it remains functionally operational.

The fundamental philosophy behind LSM is twofold: the structure is engineered for strength at the ULS and for optimal performance at the SLS [6]. Following this methodology ensures that the structure's design does not breach the collapse limit state about flexure, compression, shear, and torsion. Moreover, the structure must meet performance limits concerning deflection, cracking, and vibration. The principal goal of this design approach is to prevent either limit state from being reached throughout the intended lifespan of the structure.

The application of LSM offers several advantages. It fosters a comprehensive understanding of how a structure behaves under diverse loads and conditions. Furthermore, it facilitates the utilization of sophisticated analysis techniques such as non-linear and dynamic analysis, potentially enhancing the efficiency and cost-effectiveness of the design.

4.2 Input parameters and design criteria

After all the analysis results are extracted, interpreted, and checked for compliance against codal requirements, the concrete frame design and concrete slab design is performed in ETABS v20.0.0 itself from which the reinforcement required for beams, columns, slabs, etc. are obtained.

Table -6: Parameters for structural design

Parameter	Value
Concrete	
Elastic Modulus (MPa)	25000
Characterstic compressive strength (MPa)	25
Poisson's ratio (unitless)	0.2
Steel	
Elastic Modulus (MPa)	25000
Characterstic yield strength (MPa)	500

5. RESULTS AND DISCUSSION

5.1 Analysis output

The fundamental mode of vibration is translational. The cumulative sum of modal mass participation ratio of all considered modes as shown in Table 7 is more than 90% meeting the requirement of IS 1893(Part I):2002 Clause 7.8.4.2. The storey drift due design lateral load with partial load factor of 1.0 in X and Y directions are illustrated in Figure 5. The maximum values in all lateral load cases are less than 0.004 complying with the requirement of IS 1893(Part I):2002 Clause 7.11.1.

Table -7: Modal Mass Participation Ratios

Mode	Period (s)	Sum UX	Sum UY	Sum RX	Sum RY	Sum RZ
1	0.532	0.00	0.82	0.19	0.00	0.00
2	0.526	0.75	0.82	0.19	0.17	0.08
3	0.489	0.83	0.82	0.19	0.19	0.83
4	0.174	0.83	0.93	0.83	0.19	0.83
5	0.174	0.92	0.93	0.84	0.78	0.84
6	0.156	0.93	0.93	0.84	0.83	0.94
7	0.126	0.95	0.93	0.84	0.88	0.94
8	0.120	0.95	0.95	0.88	0.88	0.95
9	0.108	0.95	0.95	0.88	0.88	0.95
10	0.092	0.95	0.98	0.96	0.88	0.95
11	0.092	0.98	0.99	0.96	0.95	0.95
12	0.088	0.99	0.99	0.96	0.96	0.99

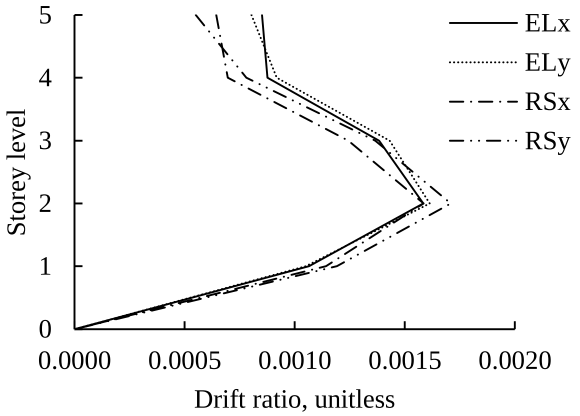


Fig -5: Storey drift

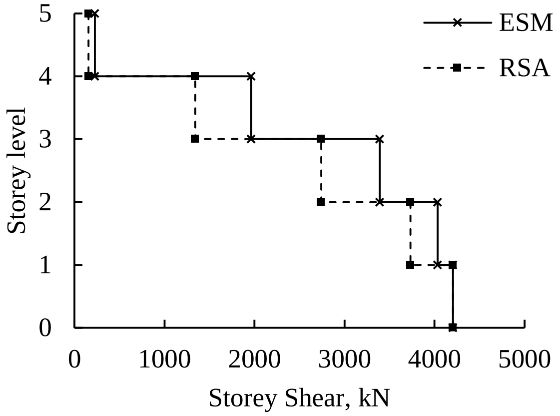


Fig -6: Storey Shear

5.2 Structural design output

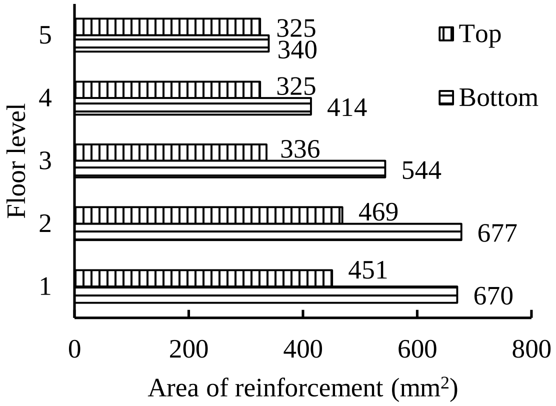
After all the analysis results are extracted, interpreted, and checked for compliance against codal requirements, the concrete frame design and concrete slab design are performed in ETABS v20.0.0 itself from which the reinforcement required for beams, columns, slabs, etc. are obtained some of which are presented hereafter.

After selecting the rebar number and sizes for beams and columns, as per IS 13920:2016 Clause 7.2.1, the CBCR is verified to be greater than 1.4. To perform this check, the end reinforcement in beams and column reinforcements were overwritten in ETABS and the Column/Beam Capacity Ratio is observed from the design output result. The CBCR is greater than 1.4 for all beam-column joints of all floors except the balcony on the first floor, roof floor, and stair cover. Since the capacity of a single column has to be taken for calculating CBCR at such joints, it is well justified to be not satisfied. This requirement is even waived at roof levels as per the same clause.

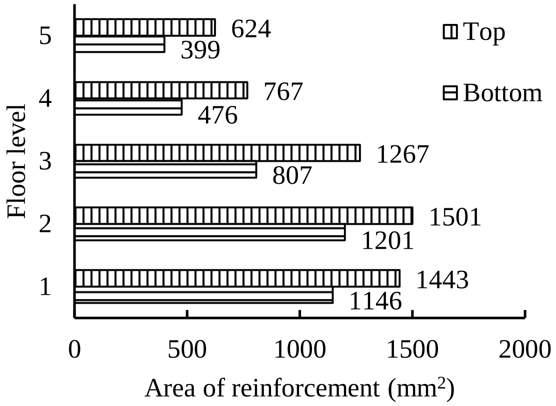
The areas of steel required to resist the design loads acting in the slab as shown in Table 8 are found to be lesser than the minimum value as per the codal recommendation which is 0.12%bD (150 mm^2).

Table -8: Area of steel required to resist design loads in slab (mm^2)

Type of Slab	**Shorter span**		**Longer span**	
	Mid	**End**	**Mid**	**End**
Interior slab	58	79	33	44
One short edge discontinuous	72	97	47	63
One long edge discontinuous	78	104	38	51
Two adjacent edges discontinuous	83	111	48	65
One-way slab	81	67	-	-



(a)



(b)

Fig -7: Maximum required longitudinal steel reinforcement for beams at (a) midspan (b) ends on various floors

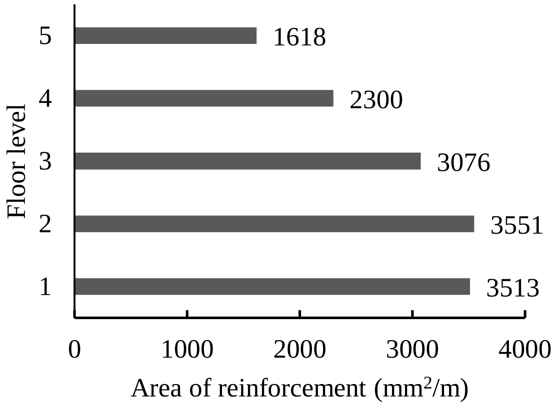


Fig -8: Maximum required shear reinforcement in beams on various floors

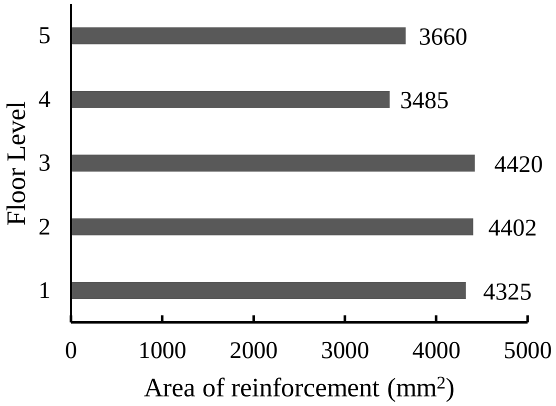


Fig -9: Maximum required longitudinal steel in columns on various floors

5.3 Deflection

The virtual deflection of slab for service load i.e. 1.0 DL+1.0 LL as obtained from ETABS is 3.136 mm. However, the actual deflection after deducting 1.117 mm for axial shortening of the column becomes 2.019 mm. Considering creep using creep coefficient as per IS 456:2000 Clause 6.2.5.1, the final deflection is computed to be 6.461 mm. The allowable deflection for a slab having a shorter span equal to 2500 mm is 7.143 mm. Thus, the deflection is within permissible limits.

6. CONCLUSIONS

The present study is conducted to develop an analytical model and analyze it for various forces, including those due to earthquakes. Due to re-entrant corner horizontal irregularity, the building is analyzed using both linear static (ESM) and linear dynamic (RSM) methods as per IS 1893(Part I):2002. Mainly, the modal mass participation ratios, storey drift, and storey shear are checked for compliance with codal requirements. The conclusions regarding various aspects are as follows:

1. The first three modes of vibration are translational which means torsional effects are not found dominant.
2. The drift ratio due to earthquake load is highest for the second storey but does not exceed 0.004 in either principal direction.
3. The base shear in RSM is lower than in ESM, which is why the former value is scaled to match the latter one, as per the codal recommendation.
4. The maximum required value of longitudinal reinforcement in the middle portion of all beams is always higher at the bottom and greatest on the second floor beams for the building under study.
5. The maximum required value of longitudinal reinforcement in beam ends is always higher at the top and greatest on the second floor beams for the building under study.
6. The maximum required value of shear reinforcement in the beam is greatest on the second-floor beams for the building under study.
7. The maximum required value of longitudinal reinforcement in the column is greatest on the third floor of the building under study.
8. The area of steel reinforcement required in slabs to resist gravity loads is lesser than the minimum required as per codal recommendations for the building under study.

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