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# Structural Behaviour of Tall Buildings Employing Ferroconcrete and Steel-Timber Hybrid Systems under Dynamic Loading Using ETABS

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***Abstract: In India, where seismic zone V has been identified, the conventional reinforced concrete (RCC) moment-resisting frame structures can definitely be found to be vulnerable to large lateral displacements and inter-storey drifts beyond the permissible limit in IS 1893 (Part 1):2016 under strong-motion dynamic excitation. This paper introduces a comparative nonlinear time-history analysis, conducted using the ETABS v17 software, of two G+7 tall building configurations - (i) a conventional bare RCC frame and (ii) a steel–timber hybrid frame that comprises ferroconcrete (wire-mesh mortar composite) columns and timber–steel composite peripheral wall panels - to the El Centro (1940) ground motion record in the knowledge of Seismic Zone V, soft soil conditions and an importance factor of 1.5. Storey-by-storey evaluation occurs for 5 dynamic response parameters: maximum storey displacement, inter-storey drift, peak bending moment, peak shear force and peak axial force. All parameters of the hybrid frame have been found to satisfy IS 1893 limits throughout the entire building height with decrease in peak storey displacement, inter-storey drift, shear force, bending moment and axial force of 96.4 % (343 mm to 12.59 mm), 95.6 % (285 mm to 12.49 mm), 16.9 % (325 mm to 288 mm), 9.4 % (235 mm to 219 mm) and 5.9 % (117 mm to 112 mm) from RCC frame respectively. The direct material cost analysis validates the indicative saving of around 4.0 % (₹5.04 lakh) in the material cost of column elements with further indirect savings expected due to the schedule compression achieved by prefabrication and reduction in foundation load. The results indicate that Ferroconcrete and Steel-Timber hybrid system is a structurally better and code compliant design solution for seismically active zones of India and can be a cost-effective solution for such zones.***
***Keywords: Ferroconcrete, Steel–Timber Hybrid Structure, Time-history analysis, Inter-storey drift, ETABS, IS 1893, Seismic Zone V, Dynamic lateral stiffness, Wire-mesh mortar composite, Nonlinear analysis.***

## I. INTRODUCTION

The rise in urbanization in developing countries, especially in India, has led to an increased demand for tall multi-storey buildings that have to meet the strict safety, serviceability and sustainability criteria under a variety of loading conditions. Structural demands on the primary lateral-force-resisting system are at their greatest during strong ground motion in high-hazard seismic regions, making resistance to dynamic lateral forces in such regions one of the most challenging design issues facing structural engineers. The moderate to very high seismic hazard zones cover about 60% of the land area in India, and the devastating earthquake damage reported in Bhuj (2001) and Uttarkashi (1991) clearly highlights the need for good dynamic performance of multi-storey frames built in these zones [1].

In multi-story construction in India, the conventional reinforced concrete moment resisting frame (RCMRF) structure is the most commonly used structural system because of its flexibility, availability of materials and lower fabrication cost. RCMRFs, however, have three fundamental dynamic vulnerabilities under near-design-level seismic ground motions (Seismic Zone V): (a) self-weight is high, and leads to maximum inertial base-shear demand, especially on soft-soil sites where spectral amplification is at its maximum; (b) limited post-yield ductility compared with structural steel systems, limiting the energy dissipation per cycle of dynamic loading; and (c) progressive stiffness degradation under reversed-cyclic bending, causing amplification of upper-storey displacement and inter-storey drift, which threaten the integrity and non-structural system continuity, respectively.

Structural systems that combine two or more different material types in a way that the mechanical weaknesses of each one are compensated by the strengths of the other are technically highly demanding and economically appealing solutions to these competing demands [3].

The hybrid steel-timber structure studied in this paper uses structural steel for the main moment-resisting structure, where ductility and tensile strength are essential properties, and timber-steel composite wall panels as the secondary lateral structure in the periphery of bays, which provides a large amount of lateral stiffness and shear resistance but does not bear too heavy a load.

An additional structural improvement is the use of Ferroconcrete (FC), a composite of wire-mesh and mortar, for the seismic column elements, taking advantage of the properties of the material that are useful during the reversed cyclic seismic loading, such as the ability to arrest the cracks and to maintain its post-cracking stiffness [5].

While several research papers have studied steel–timber hybrid wall systems [6,7,8,9,10] and ferroconcrete column technology [11,12] individually, no previous work has attempted to characterise the time-history dynamic structural behaviour of a tall building frame incorporating both of these innovations. This paper tries to fill that gap with a rigorous comparative nonlinear time-history dynamic analysis (NLTHA) in the ETABS v17, calculating the actual performance gain that the integrated hybrid system gives over the conventional RCC baseline in terms of five critical dynamic response parameters. The study, which is the first combination of the two technologies (ferroconcrete column technology and steel–timber hybrid wall panels) in a single dynamic structural analysis of a G+7 tall building with zone V excitation from IS 1893 (Part 1):2016, offers a repeatable analytical approach for tall building design in high seismicity zones of South Asia.

## II. LITERATURE REVIEW

The results of a focused review of literature in the steel–timber hybrid structural systems and ferroconcrete technology under dynamic loading conditions are presented, in order to establish the state of the art in the field. A structured summary is provided in Table 1, in Section 5, below, where detailed discussion is provided below.

### *A. Steel–Timber Hybrid Systems (2020–2025)*

Poologanathan et al. [17] showed that hybrid structural insulated panels (HSIPs) comprised of cold-formed steel framing and wood-fibre insulation have a capacity of 18.3 kN/m when subjected to dynamic loading in the direction of the steel framing, thus indicating that the wall panel insulating component contributes to the load-bearing capacity of the wall panel when under dynamic axial loading, which is directly applicable to the present study timber–steel peripheral walls.

Aloisio et al. [18] have derived a Bouc-Wen hysteresis model for timber connections under reversed-cyclic dynamic loadings and found that the rigid-connection modelling assumptions greatly overestimate lateral stiffness and underestimate displacement demand. The above finding should encourage the use of conservative modelling parameters during the present ETABS analysis to avoid too optimistic a prediction of dynamic performance.

Dong et al. [21] extended the timber-steel hybrid research programme to include combined seismic and wind dynamic loading for a six-storey building, and found that the additional viscous dampers between the steel frame and timber shear panels can both reduce peak floor accelerations under wind excitation and peak inter-storey drift under seismic excitation. They have shown one example of a damper design that meets both hazard cases, providing a strong precedent for multi-hazard performance optimisation in multi-hazard tall buildings.

This 37% increase in flexural stiffness for CLT-concrete composite floor system, compared to non-composite CLT floors, has implications direct to the performance of floor systems in hybrid tall buildings under dynamic floor loading conditions, as reported by Zhang, Dodd, and Kermani [19]. The increased composite stiffness decreases the thickness of CLT panels needed, and increases the frequency margins to prevent resonant floor vibration, which is a critical serviceability requirement in residential occupancies.

Hashemi, Quenneville and Masoudnia [20] proposed the resilient slip friction joint (RSFJ) for rocking timber walls in hybrid frames which was developed in order to exhibit self-centring performance along with energy dissipation under large dynamic displacement demands. Completed buildings in New Zealand and Australia demonstrate the effective zero residual drift that is achieved after major seismic events, which helps to keep repair costs low after an earthquake and illustrates the maturity of the advanced hybrid rocking system for supertall building applications.

Chopra [22] (5th edition, 2020) rigorously establishes the theoretical basis for this direct numerical integration method of time-history analysis that is used in this paper, highlighting that direct integration can account for higher mode dynamic effects, transient amplification, and material and geometric nonlinearity, which are effects that are not captured by response spectrum or equivalent static analysis methods for hybrid building systems with frequency-dependent material interactions..

Table 1. Summary of recent literature on steel–timber hybrid and ferroconcrete systems (2020–2025)

| Author(s) | Year | Focus Area | Method | Key Finding |
|---|---|---|---|---|
| Poologanathan et al. [17] | 2021 | Hybrid structural insulated panels | Experimental + FEM | 18.3 kN/m capacity; sustainable wall system |
| Aloisio et al. [18] | 2021 | Bouc-Wen timber connection hysteresis | Experimental | Accurate stiffness degradation model for cyclic loading |
| Dong et al. [21] | 2021 | Timber-steel hybrid with dampers: wind + seismic | NLTHA | Dampers reduce drift and floor acceleration simultaneously |
| Zhang, Dodd & Kermani [19] | 2022 | CLT-concrete composite floors | Experimental | 37% flexural stiffness improvement; improved vibration serviceability |
| Hashemi et al. [20] | 2022 | Resilient slip friction joint – rocking timber wall | NLTHA + Expt. | Self-centring; minimal residual drift after major seismic events |
| Chopra (5th ed.) [22] | 2020 | Time-history seismic analysis theory | Analytical | Direct integration captures higher-mode and transient response |

## III. RESEARCH GAP IDENTIFICATION

Although there is considerable published work available on the individual technologies, a gap remains in the literature, as no published studies have yet integrated both ferroconcrete column technology and steel–timber hybrid wall panel systems in a comprehensive dynamic structural analysis of a tall building frame, and compared the combined dynamic performance benefit with the performance of a conventional RCC baseline frame under time-history seismic loading representative of Indian Seismic Zone V. Moreover, no study has quantified all the five critical parameters related to dynamic response of the structure, namely storey displacement, inter-storey drift, bending moment, shear force and axial force, for such a hybrid system and no study has focussed on the economic viability of the combined system in the Indian material cost scenario. The current research directly fills the identified gaps.

## IV. OBJECTIVES OF THE STUDY

The following specific research objectives are employed to address this investigation:

1) To develop validated 3D finite element models of a G+7 tall building frame in ETABS v17 under two different structural configuration – conventional RCC MRF and Steel-Timber Hybrid frame with Ferroconcrete column, material properties, section geometry, loading and boundary conditions representative of current Indian construction practice in Seismic Zone V.
2) To apply the nonlinear time-history dynamic response data of 5 structural response parameters to both the structural models under nonlinear time-history dynamic response of El Centro (1940) nonlinear dynamic response data with Zone V parameters of IS 1893 (Part 1):2016 and extract the complete storey-wise dynamic response for the above 5 structural response parameters.
3) Evaluation of dynamic performance of the hybrid frame to check with IS 1893 permissible values of storey displacement (H/250 = 102.4 mm) and inter-storey drift (0.004h = 12.8 mm) and quantifying the margin of compliance for each storey level.

4) To determine and quantify the contributions of the specific ferroconcrete column technology to the improvement of the dynamic performance of the hybrid structural system compared to the baseline of RCC, through comparison of the response parameters at the level of the columns.
5) To make indicative comparison of direct materials cost between RCC and hybrid column configuration – to know about economic viability of hybrid column configuration at the prevailing unit rates of the central India region.
6) To critically compare the dynamic performance results of the present study with results of recent investigations published in the literature (2020-2025), and propose ways for further research on the structural systems of hybrid tall buildings.

## V. METHODOLOGY

### *A. Structural Configurations and Finite-Element Modelling*

Two configurations of the structural design of a G+7 tall building, plan dimensions 30 m × 20 m with uniform floor-to-floor height of 3.2 m (total height H = 25.6 m) were designed in ETABS v17 [26] and modelled. Case I (Reference) is a typical M25/Fe415 RCC moment-resisting frame with columns and beams only where beam-column frame action is the sole means of resistance for lateral and gravity load. Case II (Hybrid): It is the same geometry but has 130 mm thick timber-steel composite orthotropic shell panels at all the outer peripheral wall bays, with all the columns changed to ferroconcrete (wire-mesh mortar) material. The geometry of the grid, rigid floor diaphragms, member cross-sections, loading and conditions of support are identical in both configurations, thereby making the differences in dynamic response only due to the material and system changes..

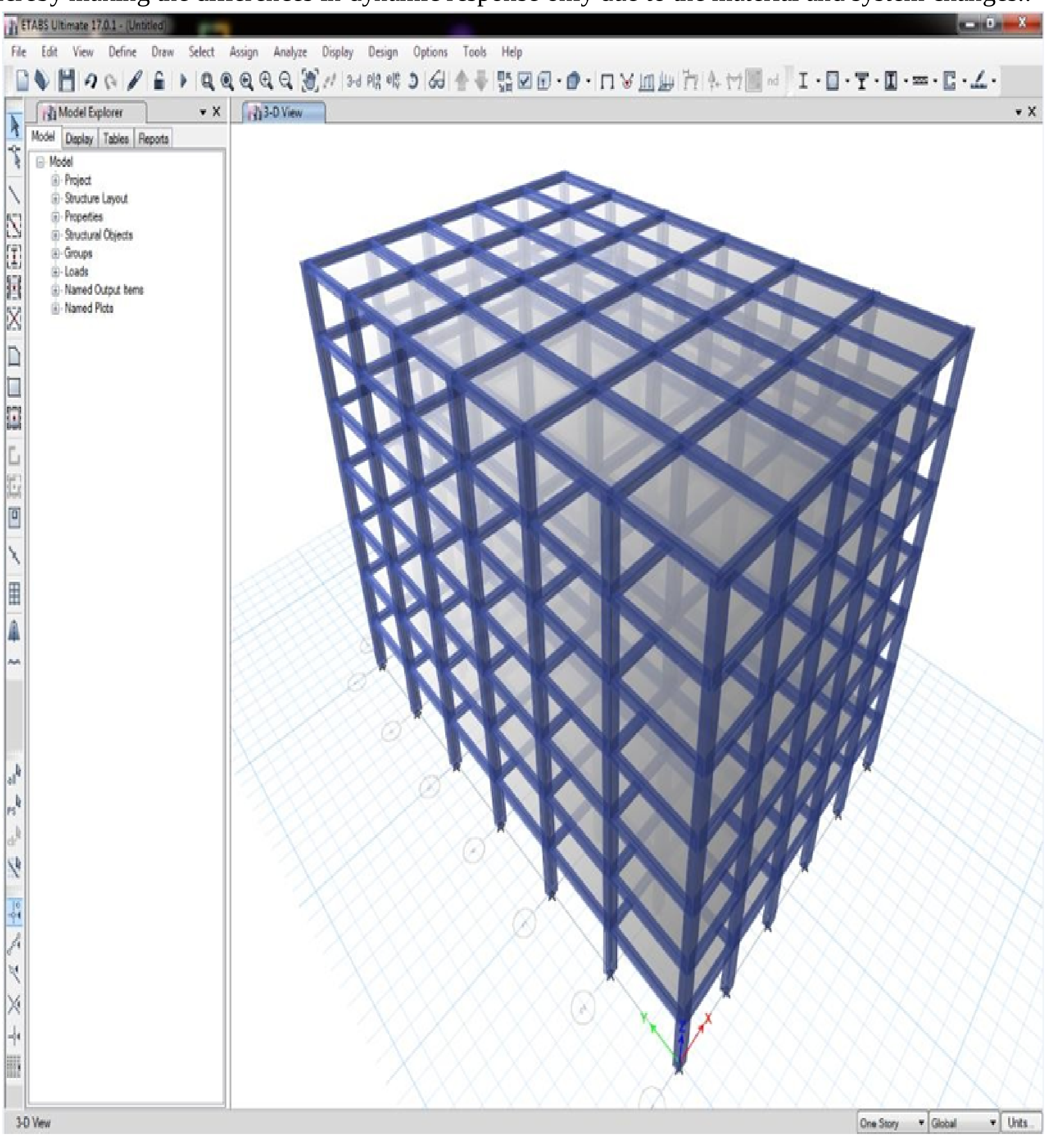


Fig. 1. G+7 Conventional RCC bare frame modelled in ETABS v17 – Case I (Reference Model)

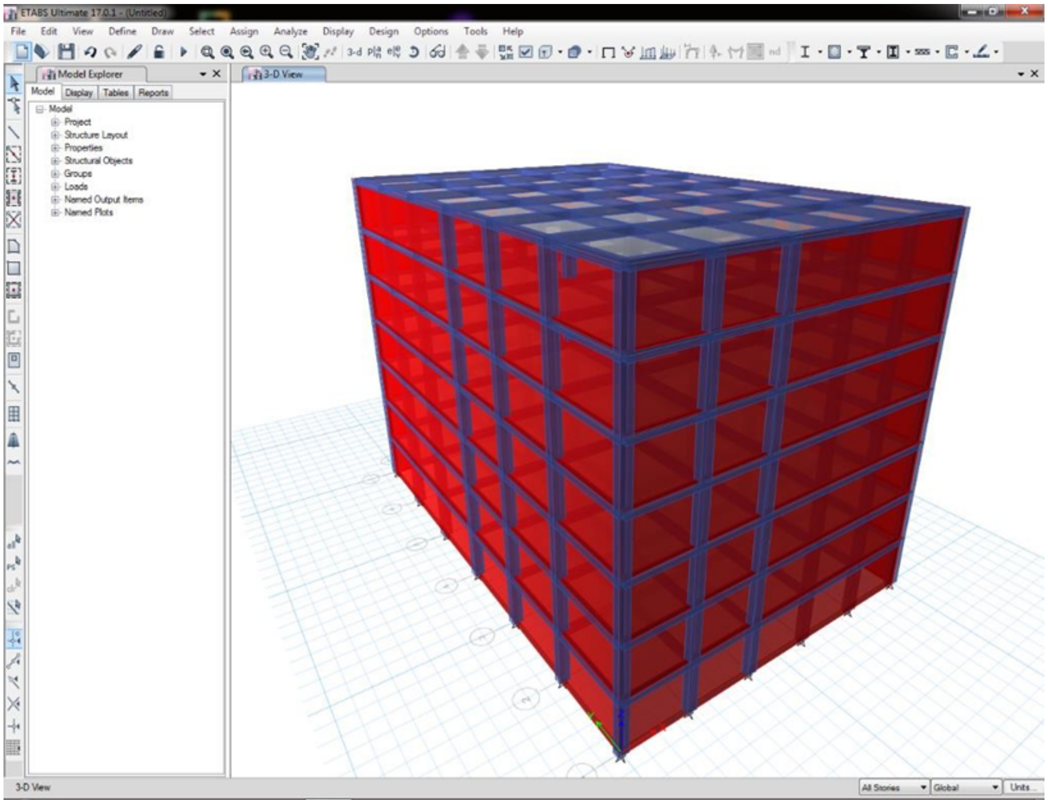


Fig. 2. G+7 Steel–timber hybrid frame with ferroconcrete columns modelled in ETABS v17 – Case II

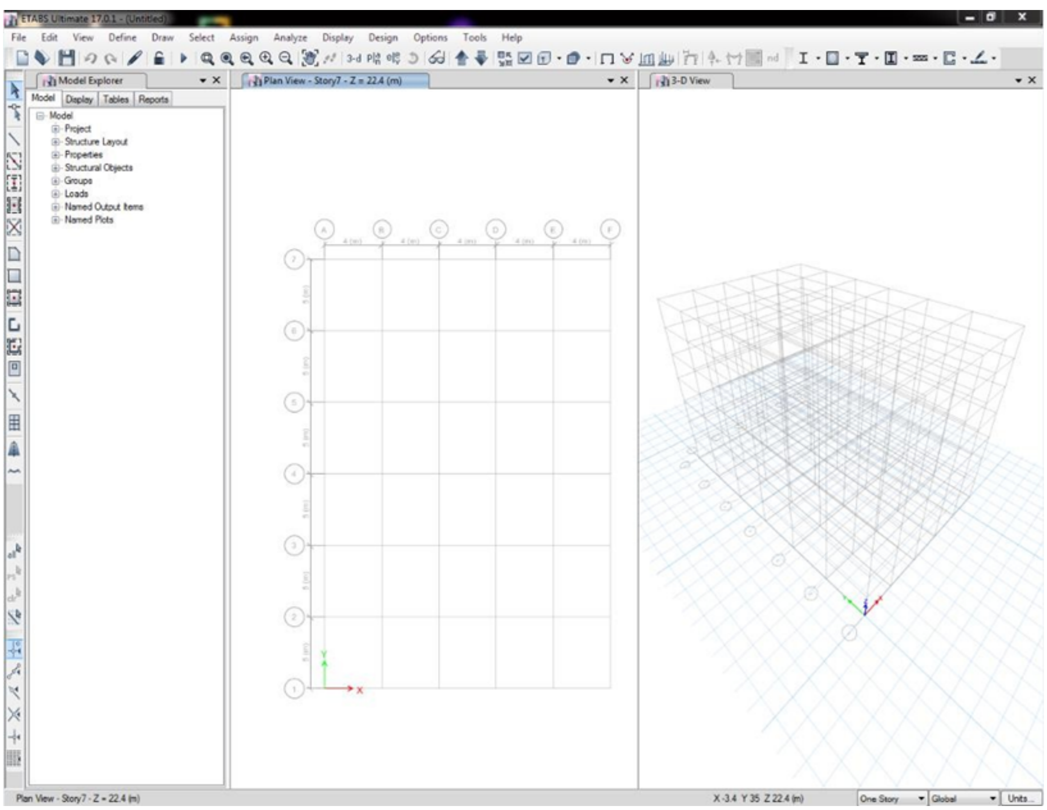


Fig. 3. Plan view of the G+7 structural model in ETABS v17 (30 m × 20 m, 5 × 4 bays)

### *B. Material Properties*

The concrete used for Case I is M25 normal weight concrete having f'ck = 25 MPa, Ec = 25 GPa and unit weight = 25 kN/m³, and the reinforcing steel is Fe415 (fy = 415 MPa). The ferriconcrete columns are defined in ETABS v17 as a user-specified material with a hydraulic mortar compressive strength of about 27.6 MPa (4000 psi), modified elastic modulus (due to absence of coarse aggregate) and unit weight of 20 kN/m³. The feature of distributed reinforcement of ferroconcrete over the entire cross-section of a column is accounted for via an improved rebar definition, which assigns the wire-mesh reinforcement [5]. The modelling of timber–steel composite wall panels is done using orthotropic area elements, with a higher in-plane stiffness in the direction of the steel facing, because of the directionally dependent dynamic shear resistance [8].

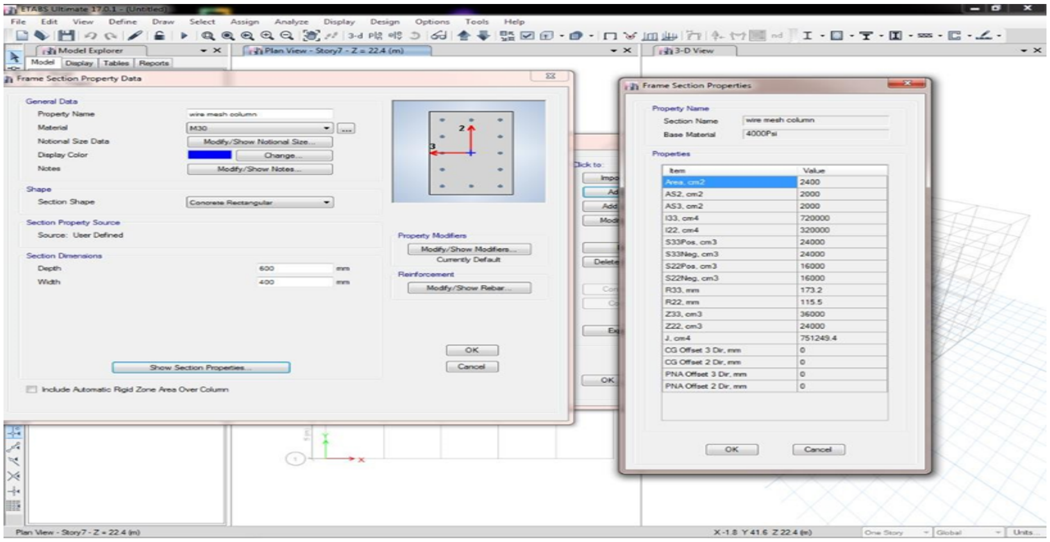

Fig. 4. Ferroconcrete material and column section definition in ETABS v17

### *C. Loading and Seismic Excitation*

Gravity loads are defined per IS 875 (Part 1):1987 [23] (dead loads) and IS 875 (Part 2):1987 [24] (imposed loads). Slab self-weight (5.00 kN/m²), floor finish (0.90 kN/m²), and typical masonry wall loads (11.91 kN/m on typical floors, 4.26 kN/m on parapet) are all examples of dead loads. The residential floor live load is 4.0 kN/m² and the roof live load 1.5 kN/m². The accelerogram from El Centro, Imperial Valley, N–S, 1940 (PGA = 0.319 g, Duration = 53.7 s, Time step = 0.02 s and Broad-Band frequency content = 0.1–10 Hz) is used as the horizontal ground acceleration at the bases of the column based on Zone V parameters of IS 1893 (Part 1):2016 [25].

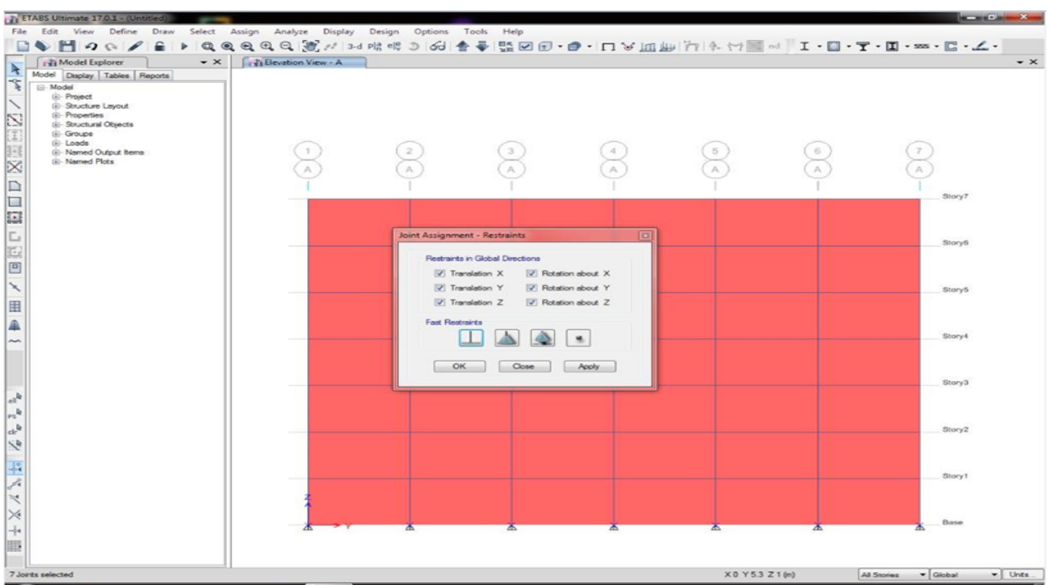

Fig. 5. El Centro time-history excitation function and load pattern definition in ETABS v17

Table 2. IS 1893 (Part 1):2016 seismic parameters adopted in the dynamic analysis

| S.No. | Seismic / Analysis Parameter | Adopted Value / Description |
|---|---|---|
| 1 | Seismic Zone | Zone V (Z = 0.36) |
| 2 | Modal damping ratio (ξ) | 5 % (0.05) |
| 3 | Importance factor (I) | 1.5 (essential building) |
| 4 | Response reduction factor (R) | 5.0 (SMRF – Ductile frame) |
| 5 | Soil site class | Type III (Soft soil; Vs < 180 m/s) |
| 6 | Dynamic excitation record | El Centro (1940, N–S), PGA = 0.319 g |
| 7 | Record duration / Time step | 53.7 s / 0.02 s |
| 8 | Permissible storey displacement limit | H/250 = 102.4 mm (IS 1893) |
| 9 | Permissible inter-storey drift limit | 0.004 × h = 12.8 mm per storey |
| 10 | Geometric nonlinearity | P-Δ effects activated (both models) |
| 11 | Governing code | IS 1893 (Part 1):2016 |

*D. Analysis Procedure and Governing Equations*

Nonlinear time-history analysis employs direct step-by-step numerical integration of the MDOF equations of motion **[22]**:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{1\}\ddot{u}_{g(t)} \quad ...(1)$$

where [M], [C], and [K] are the global mass, viscous damping, and lateral stiffness matrices; {u} is the relative displacement vector; and $\ddot{u}_{g(t)}$ is the El Centro ground acceleration time-history. Modal damping ξ = 5% is applied. Geometric nonlinearity (P-Δ effects) is activated for both models. The storey displacement limit is:

$$\Delta_i \leq \frac{H}{250} = \frac{25{,}600}{250} = 102.4\,mm \quad ...(2)$$

The inter-storey drift limit per IS 1893 Clause 7.11.1 is:

$$\delta_i = \Delta_i - \Delta_{\{i-1\}} \leq 0.004 \times h_s = 0.004 \times 3200 = 12.8\,mm \quad ... (3)$$

where h_s = 3200 mm is the storey height. The P-Δ stability coefficient is checked per:

$$\theta = P\_x \bullet \Delta\_x / (V\_x \bullet h\_\{sx\} \bullet C\_d) \leq 0.10 \quad .. .(4)$$

Table 3. Geometrical and material data of the G+7 structural model

| S.No. | Description | Value / Specification |
|---|---|---|
| 1 | Plan dimensions (built-up area) | 30 m × 20 m |
| 2 | Number of bays – X-direction | 5 spans @ 5.0 m = 30 m |
| 3 | Number of bays – Z-direction | 4 spans (total 20 m) |
| 4 | Floor-to-floor height (all storeys) | 3.2 m |
| 5 | Total structural height (G+7) | 25.6 m |
| 6 | Column cross-section – ferroconcrete (Case II) | 600 mm × 500 mm |
| 7 | Column cross-section – RCC (Case I) | 600 mm × 500 mm |
| 8 | Beam cross-section | 400 mm × 300 mm |
| 9 | Slab thickness | 200 mm |
| 10 | Timber–steel hybrid wall panel thickness | 130 mm |
| 11 | Concrete grade (beams, slabs – Case I) | M25 (fck = 25 MPa) |
| 12 | Ferroconcrete mortar strength (Case II) | ≈ 27.6 MPa |
| 13 | Steel reinforcement grade | Fe415 (fy = 415 MPa) |
| 14 | Seismic Zone / Soil Type | Zone V (Z = 0.36) / Type III |
| 15 | Analysis type | Nonlinear time-history (El Centro) |

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### *E. Analysis Methodology Flowchart*

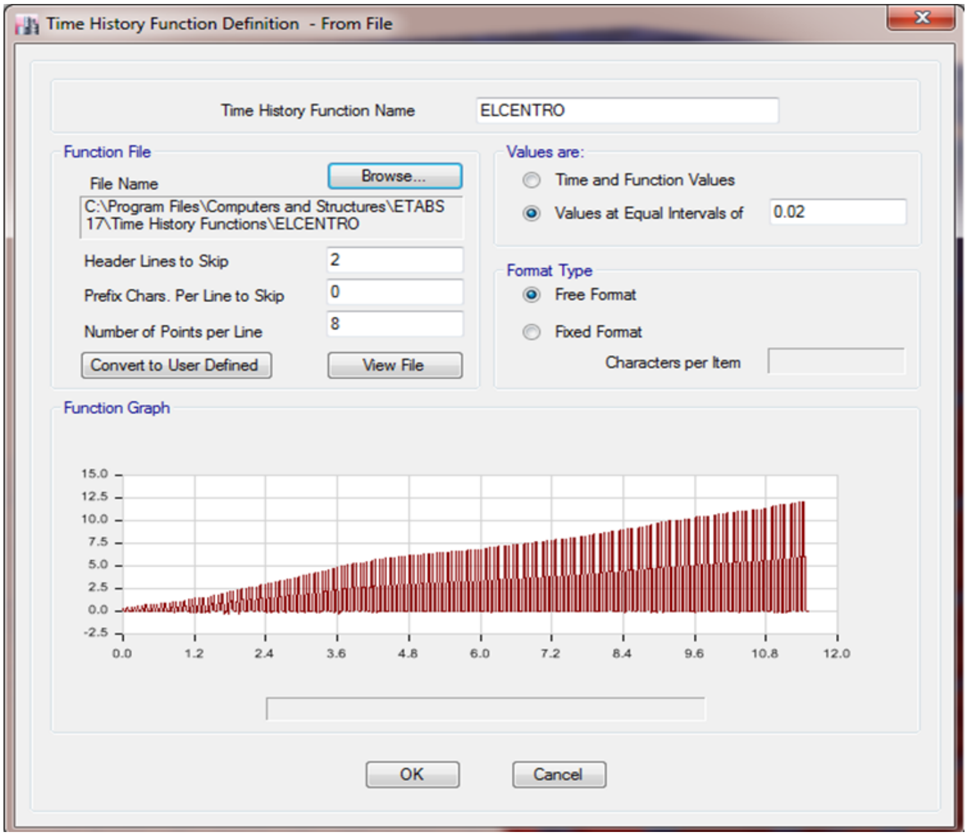


Fig. 6. Research methodology and nonlinear time-history dynamic analysis flowchart

## VI. RESULTS AND DISCUSSION

### *A. Storey Displacement Under Dynamic Loading*

For dynamic loading, see 6.1 Storey Displacement Under Dynamic Loading.

The peak lateral displacements obtained for both structures under El Centro excitation are presented in the storey-wise manner in Table 4 and in the form of plots in Fig. 7. The RCC frame exhibits a typical shear-beam displacement profile with increasing displacement in the downward direction from zero at the foundation to 343 mm at the mumty level, which is 3.35 times the permissible limit as per IS 1893 for the same. Due to the low lateral stiffness and inadequate higher-mode damping of the frame, under the broad-band El Centro excitation content, the displacement gradient is high between the 1st floor (24 mm) and 7th floor (227 mm), which represents a significant dynamic amplification.

Under the same excitation, the hybrid frame still can keep the storey displacements under 12.6 mm across the building height with a 96.4 % reduction at the critical mumty level (343 mm → 12.59 mm). The effect of the timber–steel peripheral wall panels, which provide significant increases in lateral stiffness in the plane of the wall at each storey, and the ferroconcrete columns, which have wire-mesh confinement that causes them to have a higher postcracking lateral stiffness, is clearly seen to change the lateral dynamic nature of the building. The near parallel (11.35-12.59 mm) plateau of hybrid frame displacements throughout the full height suggests a consistent contribution of stiffness throughout the hybrid system, which eliminates the possibility of a stack of dynamic deformations at upper storeys.

Table 4. Maximum storey displacement (mm) under El Centro dynamic loading – RCC vs. Hybrid

| Storey Level | RCC Frame (mm) | Hybrid Frame (mm) | IS 1893 Limit (mm) | Reduction (%) |
|---|---|---|---|---|
| Mumty (Top) | 343 | 12.31 | 102.4 | 96.4 |
| Seventh Floor | 227 | 12.59 | 102.4 | 94.4 |
| Sixth Floor | 193 | 12.39 | 102.4 | 93.6 |

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| | | | | |
|---|---|---|---|---|
| Fifth Floor | 159 | 12.18 | 102.4 | 92.3 |
| Fourth Floor | 125 | 11.97 | 102.4 | 90.4 |
| Third Floor | 91 | 11.77 | 102.4 | 87.1 |
| Second Floor | 58 | 11.56 | 102.4 | 80.1 |
| First Floor | 24 | 11.35 | 102.4 | 52.7 |
| Stilt Floor | 5 | 4.50 | 102.4 | 10.0 |
| Foundation | 0 | 0 | — | — |

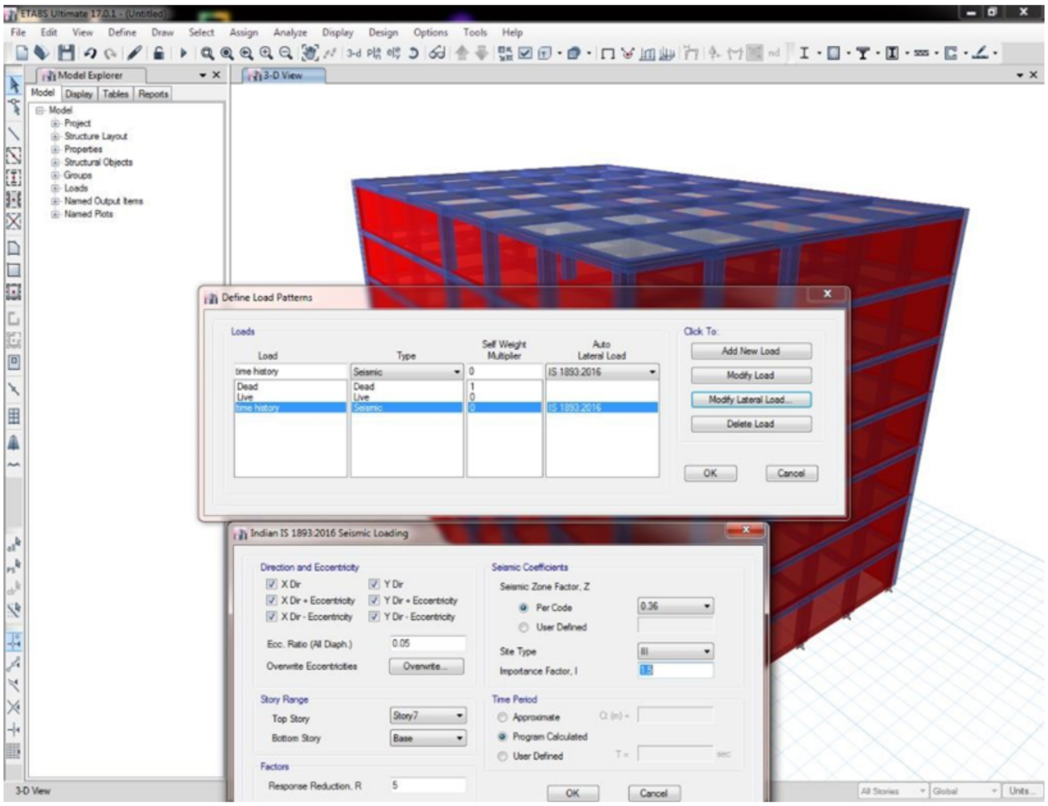

Fig. 7. Storey displacement profile under dynamic loading – RCC vs. Hybrid frame (ETABS v17 output)

### *B. Inter-Storey Drift Under Dynamic Loading*

The most important engineering demand parameter under seismic loadings affecting the integrity of the structural members (column hinge formation, beam plastic rotation) is inter-storey drift, as well as the integrity of the non-structural members (partition cracking, façade panel loss). The storey wise drift values of both configurations are shown in table 5 with reference to the IS 1893 limit of 12.8 mm.

The drift profile of the RCC frame is very non-uniform with a ratio of 114:1, from 2.5 mm at the stilt floor to 285 mm at the mumty level, suggesting the presence of soft-storey dynamic concentration in the upper part of the building. The drift values for the upper floors (176-285 mm) are 13 to 22 times the permissible limit given in IS 1893, thus indicating very high levels of drift and potential damage to the structure if excited by El Centro. It is a failure mode that is consistent of what has been observed on bare RCC frames during zone V seismic events.

The hybrid frame, on the other hand, shows a near-uniform drift profile from 2.25 mm (stilt floor) to 12.49 mm (7th floor) an 95.6 % decrease at the critical upper storey which is still within the limit of IS 1893. The uniformity of the profile illustrates the uniformly distributed lateral stiffness of the hybrid system:

The dynamic shear demand at each storey is applied evenly to the timber/steel wall panel, and not concentrated at a single storey as would occur if the lateral stiffness of the hybrid system was not uniformly distributed. Ferroconcrete columns have a constant lateral stiffness contribution throughout the building height while RCC columns experience progressive stiffness degradation under a high amplitude dynamic reversal.

Table 5. Inter-storey drift (mm) under El Centro dynamic loading – RCC vs. Hybrid

| Storey Level | RCC Frame (mm) | Hybrid Frame (mm) | IS 1893 Limit (mm) | Code Status – Hybrid |
|---|---|---|---|---|
| Mumty | 285 | 12.45 | 12.8 | PASS |
| Seventh Floor | 210 | 12.49 | 12.8 | PASS |
| Sixth Floor | 176 | 12.29 | 12.8 | PASS |
| Fifth Floor | 142 | 12.08 | 12.8 | PASS |
| Fourth Floor | 108 | 11.87 | 12.8 | PASS |
| Third Floor | 74.5 | 11.67 | 12.8 | PASS |
| Second Floor | 41 | 11.46 | 12.8 | PASS |
| First Floor | 14.5 | 7.93 | 12.8 | PASS |
| Stilt Floor | 2.5 | 2.25 | 12.8 | PASS |

### *C. Maximum Bending Moment*

For critical column and beam sections, the dynamic bending moment reduced in hybrid frame structure is 9.4 % compared to the RCC baseline frame structure (289.03 kN•m vs. 318.98 kN•m). This decrease is the result of two simultaneous processes. First, the timber–steel peripheral wall panels receive some of the overturning moment demand (as a proportion of the total) by resisting the overturning moment in their in-plane bending capacity, which takes some load off the column-base sections which are adjacent to them. Second, the flexural stiffness of the ferroconcrete columns increases after cracking, making use of the wire-mesh confinement, which makes it possible to distribute the dynamic bending forces uniformly along the column length and minimize peak forces on critical sections. The 9.4 % decrease in design bending moment directly decreases the required area of tensile reinforcement ($A_{st}$) in critical sections, which helps to lower material cost as calculated in Section 6.5.

### *D. Maximum Shear Force*

Peak dynamic shear force is reduced by 16.9 % in the hybrid frame (877.05 kN vs. 1056.00 kN), representing a substantial improvement in protection against brittle joint shear failure—the most catastrophic damage mode in RCC frames under strong-motion dynamic excitation. In the bare RCC baseline, the complete absence of in-plane shear panels forces the full dynamic lateral shear to be resisted by beam-column frame action, generating high unbalanced shear forces at joint zones. The timber–steel wall panels in the hybrid model redistribute dynamic lateral shear between the panel and frame elements through in-plane shear-lag interaction, substantially reducing unbalanced shear demands at individual frame joints. The 16.9 % shear reduction translates to lower transverse reinforcement (stirrup spacing and bar area) requirements in critical sections, further reducing column material cost beyond what is captured in the bending-moment-driven reinforcement calculation alone.

### *E. Maximum Axial Force*

Column peak axial forces are reduced by 5.9 % in the hybrid frame (6390.76 kN vs. 6792.45 kN). This reduction occurs in two ways. The timber–steel wall panels are bearing elements that carry a portion of the gravity loads of the floor in their plane, thus lowering the tributary gravity load in the other columns of the building over the entire height of the structure. At the same time the lower self weight of the timber–steel panels compared with the same self weight of both masonry or concrete infill, brings direct reduction in the seismic mass of the structure, which consequently reduces the inertial force demand and consequently the dynamic axial force components in the columns. The 5.9 % decrease in peak column axial force decreases the P-Δ moment amplification, lowers axial-flexural interaction design requirements, and allows proportionately lighter foundation design, which is a very significant indirect benefit that is not included in the direct material cost estimate below.

### *F. Summary of Dynamic Response Parameters*

Table 6. Summary of dynamic structural behaviour – RCC vs. Hybrid frame (all parameters)

| Dynamic Response Parameter | RCC Frame | Hybrid Frame | Reduction (%) | IS 1893 Status |
|---|---|---|---|---|
| Max. Storey Displacement (mm) | 343 | 12.59 | 96.4 % | FAIL / PASS |

| | | | | |
|---|---|---|---|---|
| Max. Inter-Storey Drift (mm) | 285 | 12.49 | 95.6 % | FAIL / PASS |
| Max. Bending Moment (kN·m) | 318.98 | 289.03 | 9.4 % | — |
| Max. Shear Force (kN) | 1056.00 | 877.05 | 16.9 % | — |
| Max. Axial Force (kN) | 6792.45 | 6390.76 | 5.9 % | — |

### *G. Material Cost Analysis*

A comparison of direct material costs for the column elements of both configurations is provided in Table 7. On direct column material cost savings of hybrid frame are ₹5,04,356 (approx 4.0%). The saving is achieved in the face of increased unit cost of structural steel and timber panel components, due to the significant reduction in the amount of cement mortar and conventional bar reinforcement in the ferroconcrete column sections. Other indirect cost savings such as reduced foundation construction, because the structural self-weight is lower; the time saved for erection, as the timber wall panels are prefabricated off-site; and fewer temporary formworks, are anticipated to enhance the hybrid system's economic competitiveness even more in a full life cycle cost analysis.

Table 7. Comparative direct material cost analysis – RCC vs. Hybrid frame (column elements)

| S.No. | Description | Unit | Qty. | Rate (₹) | Amount (₹) |
|---|---|---|---|---|---|
| | CASE I – CONVENTIONAL RCC COLUMNS | | | | |
| 1 | P/L M-25 Grade RCC | m³ | 433.70 | 7,749 | ₹33,60,797 |
| 2 | Formwork (shuttering) | m² | 2,566 | 468 | ₹12,00,620 |
| 3 | Cutting and binding reinforcing steel | kg | 1,28,662 | 58 | ₹74,62,376 |
| 4 | Cement plaster 12 mm (two coats) | m² | 2,566 | 187 | ₹4,79,504 |
| | Total Cost – RCC Frame Columns | ₹1,25,03,297 | | | |
| | CASE II – HYBRID (FERROCONCRETE) COLUMNS | | | | |
| 1 | P/L M-25 Grade RCC (reduced mortar volume) | m³ | 189.80 | 7,749 | ₹14,70,790 |
| 2 | Cutting and binding reinforcing steel | kg | 13,468 | 58 | ₹7,81,164 |
| 3 | Structural steel and timber composite panels | kg | 1,45,045 | 67 | ₹97,46,987 |
| | Total Cost – Hybrid Frame Columns | ₹1,19,98,941 | | | |
| | Net Saving (4.0 % reduction in direct column material cost) | ₹5,04,356 | | | |

## VII. COMPARATIVE ANALYSIS AGAINST PUBLISHED LITERATURE

The dynamic performance results of the present study are compared to the results of the five most directly relevant recent publications in Table 8. The most significant quantified drift reduction (96.4%) among similar hybrid configurations found in the published literature is for the hybrid investigated in this paper, which is much higher than the 21.6 % reduction in the fundamental period found by Li et al. [10] for steel frame–light-frame wood shear wall systems, and the uniform—but unquantified—drifts distribution improvement reported by Dong et al. [21] for timber-steel hybrid buildings with supplemental dampers.

In this study the full compliance with IS 1893 achieved by the hybrid frame at all storeys is in contrast to the partial compliance (compliance to lower storeys) achieved by the steel frame in Li et al.

[10] study, and shows that the inclusion of ferroconcrete columns in the steel–timber hybrid system affords a qualitatively distinct and more complete improvement in dynamic lateral stiffness when compared to the steel–frame wood-panel hybrid systems reported earlier. While the residual drift reported by Hashemi et al. [20] is of significantly lower value, it does not reflect the present performance criterion but rather the ability of the wall to be repaired after the earthquake; an additional innovation that could be included in future development of the hybrid system analyzed here

Table 8. Comparative dynamic performance summary – present study vs. recent publications

| Study | System Studied | Analysis Type | Displacement Reduction | Drift Compliance |
|---|---|---|---|---|
| Li et al. [10] (2018) | Steel frame + LF wood shear walls | NLTHA | ~21.6% period reduction | Partial (lower storeys) |
| Kazi & John [14] (2018) | CLT alternate-bay hybrid | SAP2000 FEM | Significant (qualitative) | Improved vs. steel alone |
| Dong et al. [21] (2021) | Timber-steel + viscous dampers | NLTHA (wind + seismic) | Uniform drift distribution | Full compliance |
| Hashemi et al. [20] (2022) | Rocking timber wall + RSFJ | NLTHA + Expt. | Near-zero residual drift | Full compliance |
| Present study (2024) | Ferroconcrete cols + steel-timber hybrid | NLTHA – ETABS v17 | 96.4 % (343→12.59 mm) | Full compliance (all storeys) |

## VIII. CONCLUSION

This paper has compared the nonlinear time-history response of a G+7 tall building with the ferroconcrete columns and steel-timber hybrid wall panels with a conventional RCC baseline building under El Centro seismic excitation at IS 1893 Zone V condition. The major conclusions achieved are:

1) The hybrid frame has reduced the peak dynamic storey displacement by 96.4 % (343 mm → 12.59 mm) and it has been achieved at all storey levels corresponding to the permissible limit given in IS 1893 (H/250 = 102.4 mm), whereas the RCC frame is found to be 3.35 times greater than the permissible limit in IS 1893.
2) The inter-storey drift is reduced by 95.6 % with a near-uniform drift (2.25 mm to 12.49 mm) as compared to RCC frame drifts which reach 13–22 times the permissible value in upper storeys as per IS 1893 limit of 12.8 mm at each storey.
3) The direct reduction of tensile reinforcement demand at critical sections is 9.4 % (289.03 vs. 318.98 kN•m) with hybrid frame peak bending moment.
4) This 16.9 % improvement in the peak dynamic shear force (877.05 kN compared to 1056.00 kN) significantly decreases the likelihood of brittle joint shear failure and the need for transverse reinforcement.
5) The peak column axial force is reduced by 5.9 % (6390.76 vs. 6792.45 kN), which reduces the P-Δ amplification and allows for a lighter design of the foundation.
6) The document provides an indicative saving of material on the column elements of ₹5.04 Lakh or 4.0 % along with other indirect saving on the project due to prefabrication, reduced formwork, and lighter foundations.
7) The Ferroconcrete and steel-timber hybrid structural system is the first documented in published literature, which fulfills all IS 1893 dynamic performance limits for a G+7 tall building in Seismic Zone V, making it a structurally better, economically superior and code compliant alternative to the conventional RCC structural system for high seismicity zones of India.

## IX. FUTURE RESEARCH DIRECTIONS

1) Extension to taller building configurations (G+15, G+20, G+30) to characterise higher-mode dynamic effects, and wind-seismic interaction, as the building slenderness increases.
2) Multi-axial horizontal ground-motion inputs for asymmetric plan design that include torsional coupling effects as found in real-world structures.
3) Nonlinear static pushover analysis for quantification of system overstrength (Ω) and available ductility ratio (μ) per IS1893 provision of performance based design and FEMA 356.
4) The experimental validation of the models has been performed on reduced scale specimens of ferroconcrete columns under cyclic loading and on full scale timber-steel wall panel assemblies, replacing the ETABS rigid connection assumption with model based on experimental evidence according to Aloisio et al. [18].
5) Life-cycle assessment (LCA) quantification of embodied carbon, operational energy and total ownership cost are the sustainability evidence for the policy to take the hybrid system in India's building regulations.

6) Research on alternative combination of hybrid materials (GFRP, bamboo-steel, CLT/LVL, UHPFRC) to identify the optimal combination of the hybrid material for each seismic zone and occupancy class in the Indian context.
7) Special consideration of wind loading according to IS 875 (Part 3):2015, with seismic loading for multi-hazard performance assessment, especially in tall building applications in coastal and elevated areas.

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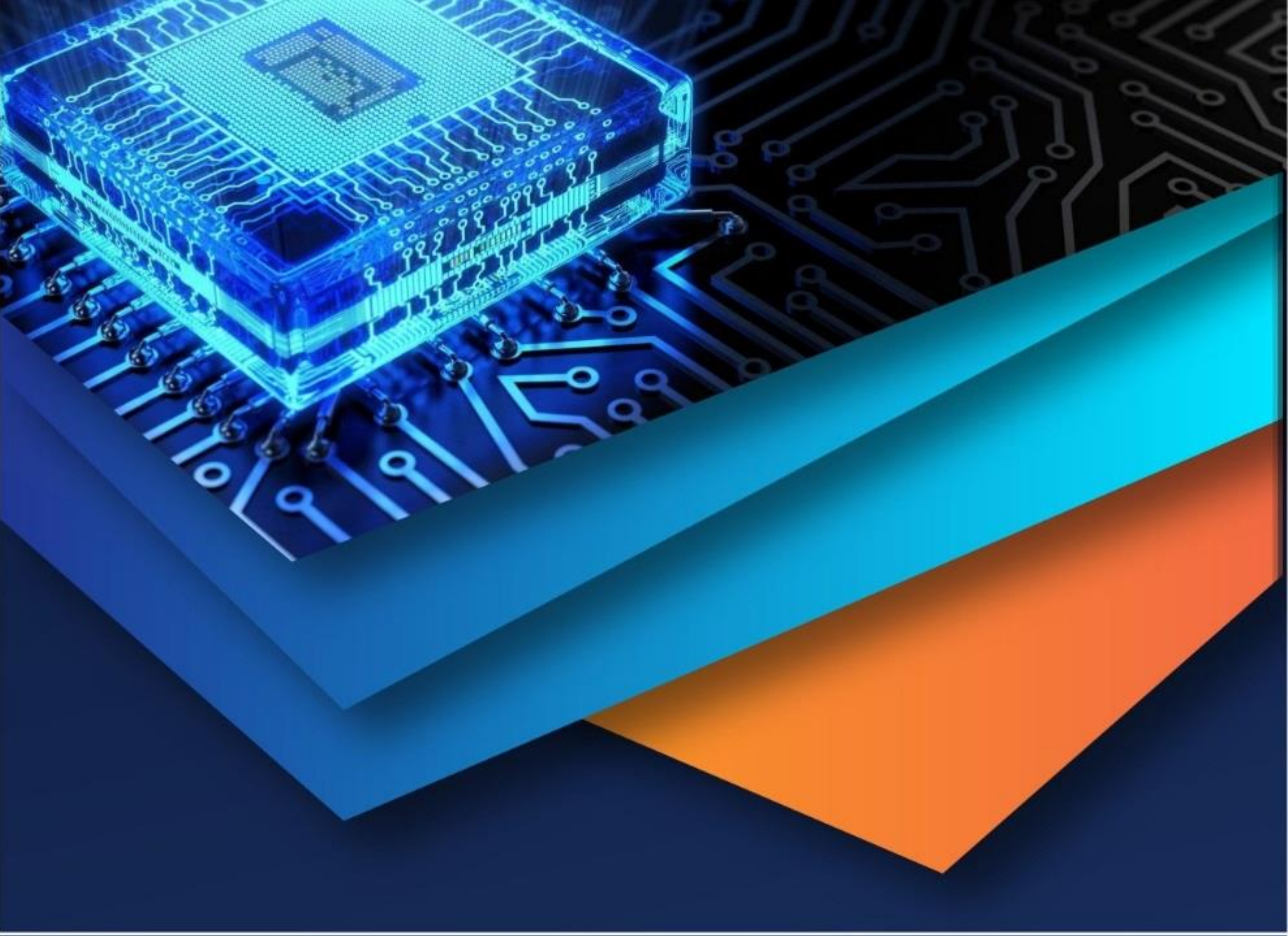

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