Specially Shaped Reinforced Concrete Columns in High-Rise Buildings Subjected to Lateral Loads: A Review of the State of the Art, Research Gaps and a Proposed Analytical Methodology

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Abstract- Specially shaped reinforced concrete columns of L-, T-, plus- and cross-shaped cross-section are increasingly adopted in residential and commercial construction because their limbs can be concealed within the thickness of partition walls, removing the projecting corners produced by conventional rectangular columns and releasing usable floor area. The same geometry, however, redistributes material away from the section centroid and therefore alters flexural rigidity, torsional response, mass participation and the biaxial interaction of axial force with bending, all of which govern how a tall frame responds to earthquake and wind actions. This paper reviews thirty-plus experimental, numerical and code-based investigations published between 2008 and 2026, organised into six thematic streams: member-level testing of reinforced and steel-reinforced concrete sections; composite and concrete-filled steel tube variants; finite element parametric studies; prefabricated systems; strengthening and retrofitting; and whole-building comparative analyses. The synthesis shows a consistent finding that shaped sections match or exceed the stiffness, ductility and economy of area-equivalent rectangular sections, while also revealing that loading direction, limb slenderness and axial compression ratio are decisive and inadequately codified variables. Seven research gaps are identified, the most significant being the near-absence of storey-by-storey whole-building comparisons for genuinely high-rise frames in which lateral effects dominate, and the tendency of existing frame-level studies to report a single response quantity rather than the full set of moment, shear, displacement and drift. A detailed methodology is then proposed to close these gaps, based on three geometrically identical G+12 reinforced concrete models differing only in column shape, analysed by the equivalent static and response spectrum methods in ETABS to Indian Standard provisions and validated against an independent manual design. An illustrative application indicates reductions of approximately 31% in peak bending moment, 13% in peak storey shear and 27% in peak roof displacement for plus-shaped columns relative to the rectangular baseline, and shows the baseline exceeding the code drift limitation at the first storey where the shaped models do not.

Keywords—specially shaped columns, high-rise buildings, lateral loads, ETABS, response spectrum analysis, storey drift, research gap, review.

I. INTRODUCTION

Contemporary architecture places a high premium on interiors that are free of projecting structural members. Designers increasingly require that column faces remain flush with the surrounding partition walls so that the entire floor plate is available for occupation and furnishing. Conventional rectangular columns, when located at wall intersections and at building corners, intrude into the room and interrupt this arrangement, producing dead corners that are difficult to furnish and that reduce the marketable carpet area. Specially shaped columns — principally L-, T-, plus (+) and cross-shaped cross-sections — have emerged as a direct response to this requirement, since their limbs can be embedded within the wall thickness and leave the room boundary unbroken while still transmitting the full gravity and lateral demand to the foundation.

The architectural argument is well understood in practice, but the structural consequences are less widely appreciated. Distributing concrete and reinforcement into limbs that project from a central core moves material away from the section centroid, which changes the second moment of area about both principal axes, alters the ratio of flexural to shear stiffness of the member, and modifies the mass and stiffness distribution of the frame as a whole. Because the lateral response of a tall building — its fundamental period, storey displacement profile, inter-

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storey drift and the distribution of internal forces up the height — is governed by precisely these properties, the choice of column shape is not a purely architectural decision. It materially affects how the structure resists wind and earthquake actions. Experimental and numerical work over the past two decades has confirmed that shaped sections can deliver strength and ductility comparable to, and in several respects better than, equivalent rectangular sections [1], [2].

At the same time, shaped columns introduce complications that rectangular sections do not present. The centroid and shear centre of an L- or T-section do not coincide, so lateral load along a global axis generates torsion within the member. The principal axes are inclined to the building grid, so the critical loading direction for a corner column is generally oblique rather than orthogonal. The limbs behave as thin outstands that may be governed by local rather than global instability, and confinement at the re-entrant junction is difficult to achieve with conventional stirrups. None of these effects is captured by the interaction charts and design aids used routinely for rectangular sections, and codes in most jurisdictions, including the Indian Standards, contain no clauses for non-rectangular geometry. Analysts must therefore rely on finite element modelling.

A substantial literature now exists on this subject, but it is unevenly distributed. Most of the experimental work is conducted at the member or sub-assemblage level, on single columns or on beam-column joints tested under cyclic loading, and although this work characterises the section thoroughly it does not translate directly into the storey-wise response quantities that govern the design of a building. Conversely, the studies conducted at the whole-building scale are dominated by low- and medium-rise frames of six to fifteen storeys, are frequently restricted to a single lateral analysis method, and typically report only one or two response quantities — usually displacement or base shear — rather than the complete set needed for design [3], [4]. The result is a gap between what is known about the member and what a designer needs to know about the building.

This paper addresses that gap in three stages. First, it reviews the literature systematically, grouping studies into thematic streams and extracting the parameters each has examined, so that both the coverage and the omissions become explicit. Second, it states the research gaps that follow. Third, it proposes an analytical methodology to close them, based on geometrically identical high-rise models differing only in column shape, analysed under identical lateral demand by both static and dynamic procedures in ETABS and verified against an independent manual design. The methodology follows the modelling practice established in comparable ETABS-based building studies [5] and the provisions of the Indian Standard for earthquake-resistant design [6], but the structure of the investigation is general and transferable to other code environments.

Sections II and III set out the structural rationale of shaped columns and the method adopted for the review. Section IV presents the review itself in six thematic streams, Section V synthesises it, and Section VI states the resulting research gaps. Sections VII and VIII present the proposed methodology and an illustrative application, and Sections IX and X discuss limitations and conclude.

II. SPECIALLY SHAPED COLUMNS: GEOMETRY AND STRUCTURAL RATIONALE

A. Definition and Architectural Motivation

A specially shaped column is a reinforced concrete compression member whose cross-section is composed of two or more limbs of wall thickness meeting at a junction, so that the overall section takes the form of a letter — L, T, Z or a plus sign. The defining dimensional characteristic is that the limb thickness is equal to, or close to, the thickness of the partition wall in which the column sits, typically 200 mm to 250 mm in Indian residential construction, while the limb length is several times that thickness. Because no part of the section projects beyond the wall face, the column is invisible within the finished room. The three cross-sections examined in this paper are shown to a common scale in Fig. 1. The rectangular section of Fig. 1(a) is confined by a single closed tie, whereas the T- and plus-shaped sections of Fig. 1(b) and 1(c) require two overlapping rectangular tie sets, since no single closed stirrup can follow the re-entrant geometry. This detailing consequence is returned to in Section II-C.

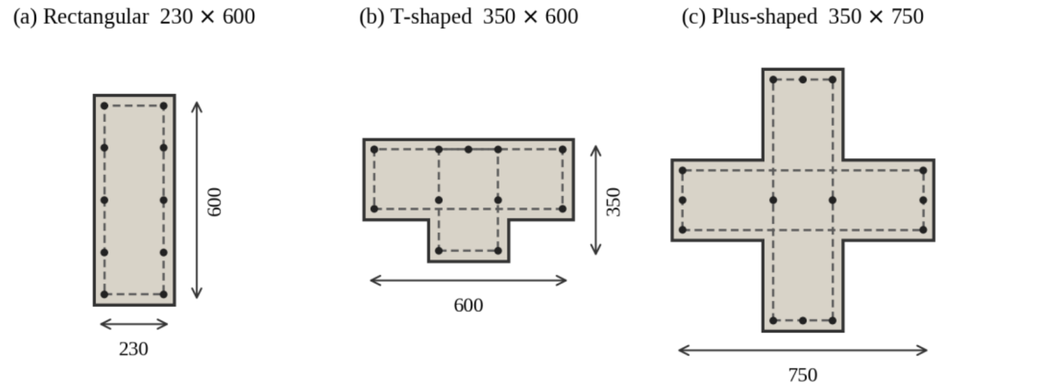

Fig. 1. Cross-sections of the three column configurations, drawn to a common scale.

The space benefit is quantifiable. Studies that have compared framing systems on identical architectural plans report increases in usable carpet area of the order of fifteen to twenty per cent at the corners and wall junctions where shaped columns replace rectangular ones, and in some configurations a reduction in the total number of columns required [7], [8]. For developers working to a saleable-area metric this is a material commercial advantage, and it explains why the shaped-column framing system has matured from an occasional architectural device into a mainstream technology for villas and medium-to-high-rise apartment blocks in several markets.

B. Classification of Columns Relevant to the Review

For the purposes of the discussion that follows it is useful to recall the standard classifications of reinforced concrete columns, since the reviewed studies cut across several of them. Table I summarises the classification and identifies where shaped columns sit within it.

TABLE I. CLASSIFICATION OF COLUMNS AND THE POSITION OF SPECIALLY SHAPED SECTIONS

Basis of classification	Categories	Relevance to shaped columns
Type of reinforcement	Tied, spiral, composite	Shaped sections are almost always tied; confinement at re-entrant corners is the key detailing problem
Type of loading	Axial, uniaxial eccentric, biaxial eccentric	Corner shaped columns are inherently biaxially loaded; the critical direction is oblique to the building grid
Slenderness ratio	Short (< 12), long (> 12)	Individual limbs may be locally slender even when the overall member is short
Cross-sectional shape	Square, rectangular, circular, specially shaped (L, T, +)	The variable examined in this review
Construction material	RC, steel, composite, timber, masonry	Shaped geometry is used in RC, in steel-reinforced concrete and in concrete-filled steel tube form

Note: The categories follow conventional textbook classification; the third column identifies the specific implication for non-rectangular geometry.

C. Why Shape Governs Lateral Response

Four mechanisms link cross-sectional shape to the lateral behaviour of a frame, and they recur throughout the literature reviewed in Section IV.

- Flexural rigidity. For a fixed gross area, extending material into limbs increases the second moment of area about the axis parallel to the limb, so a shaped column contributes more flexural stiffness to the frame in at least one direction and reduces storey displacement and drift under the same lateral force.

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This is the mechanism most often cited to explain the displacement performance of shaped-column frames [3], [9].
- Period shift and force redistribution. Greater lateral stiffness shortens the fundamental period, changing the spectral acceleration ordinate the structure attracts. Depending on where the period falls on the design spectrum this may raise or lower the total base shear, but it invariably alters the vertical distribution of storey shear and column moment, so comparative studies report a redistribution of demand rather than a uniform reduction [3], [7].
- Biaxial interaction and oblique loading. The principal axes of an L- or T-section are inclined to the orthogonal building grid, so the section experiences combined biaxial bending and axial force under a nominally uniaxial ground motion. The axial force-moment interaction surface of such a section is markedly non-circular, and the weakest direction may not correspond to either building axis. This effect is explicitly demonstrated by the tests on prefabricated L-shaped members loaded in different directions and is central to the safe design of corner columns.
- Confinement and ductility. The concrete at the re-entrant junction is difficult to confine, since no single closed stirrup can follow the geometry; multiple overlapping sets are required and confinement falls away with distance from the stirrup plane. The ductility of a shaped column is therefore more sensitive to detailing than that of an equivalent rectangular column [10], [11].

These mechanisms operate simultaneously and sometimes in opposition, which is why analytical or numerical treatment is unavoidable and why the empirical findings of the reviewed studies are not always in agreement.

III. METHODOLOGY OF THE REVIEW

The review was conducted in four steps in order to ensure that the coverage was systematic rather than opportunistic, and that the identified gaps follow demonstrably from the evidence.
- Step 1 — Source identification. Peer-reviewed journal articles, conference proceedings and standards published between 2008 and 2026 were identified using the keyword combinations "specially shaped column", "L-shaped column", "T-shaped column", "plus-shaped column", "cross-shaped column", "shaped column seismic behaviour", and "high-rise ETABS column shape". Reference chaining from the retrieved items was used to capture earlier foundational work.
- Step 2 — Screening. Studies were retained only if they reported quantitative results — experimental measurements, validated finite element output or code-based analysis — relating either to the behaviour of a shaped column member or to the response of a frame containing shaped columns. Purely qualitative or descriptive articles were excluded.
- Step 3 — Thematic classification. The retained studies were grouped into six streams: member-level experimental work on reinforced and steel-reinforced concrete sections; composite and concrete-filled steel tube variants; finite element parametric studies; prefabricated and precast systems; strengthening and retrofitting; and whole-building comparative analysis. Code documents and reliability studies were treated separately.
- Step 4 — Parameter extraction and gap analysis. For each study the investigated parameters, the scale of the investigation (member, sub-assemblage or building), the analysis or test method, and the reported response quantities were tabulated. The resulting coverage matrix, presented in Section V, makes the omissions in the collective body of work explicit and forms the evidential basis for the research gaps stated in Section VI.

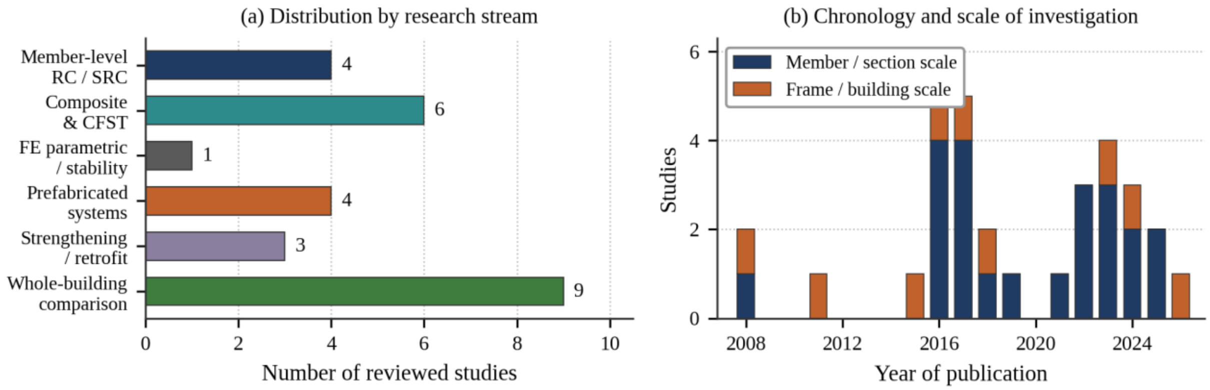

Fig. 2. Distribution of the reviewed studies by research stream and by year of publication.

Fig. 2 summarises the resulting corpus. Panel (a) shows that whole-building comparison is the single largest stream, yet panel (b) shows that the recent growth in publication has been concentrated at the member and section scale, chiefly on composite and prefabricated variants. The imbalance visible between the two panels is itself the origin of several of the gaps stated in Section VI.

This has one deliberate consequence. Because the extraction step records not only what each study found but also what it measured, a parameter consistently absent from the reported outputs — drift in the member-level studies, or moment demand in the frame-level studies — becomes visible as a gap even when no individual author identified it as such.

IV. REVIEW OF LITERATURE

A. Member-Level Experimental Studies on RC and SRC Shaped Columns

The earliest contributions concern individual shaped columns tested under monotonic or reversed cyclic loading. Wang, Wei and Zhang [10] investigated the confinement mechanism of L-shaped columns under repeated axial loading, combining tests with an ANSYS model. Computed peak load, peak displacement and skeleton curve agreed closely with the experimental record, and the study established that confinement is strongest within the plane of the stirrup and diminishes rapidly as stirrup spacing increases — a finding with direct implications for detailing the re-entrant corner.

Zhang, Chen, Li and Zhang [11] examined the ductility of steel-reinforced concrete T-shaped columns under reversed cyclic loading in MSC Marc, varying the axial-compression ratio, the shear-span ratio and the load angle. Load capacity fell as the shear-span ratio increased and displacement ductility fell as the axial compression ratio rose, but the ductility coefficient exceeded three in all cases. The treatment of load angle as an explicit variable is significant, since it anticipates the oblique-loading problem later confirmed at full scale.

Li, Ye and Sun [1] showed that steel-fibre reinforcement improves energy dissipation and crack control of the T-section, particularly at the junction where conventional confinement is least effective. Wang, Huang and Fu [12] extended member testing to the fire condition, subjecting three T-shaped SRC columns to combined high temperature and vertical load. Failure pattern, temperature field and fire resistance all varied with axial-compression ratio and eccentricity: specimens at a ratio of 0.6 lost approximately 57% of the fire resistance of those at 0.2, with further loss as eccentricity grew. The sensitivity of shaped sections to axial compression ratio is therefore not confined to the seismic case.

B. Composite, CFST and Tubed Shaped Columns

A parallel stream examines shaped geometry in composite form, where a steel tube or encased steel section supplies confinement and part of the strength. Zhou and co-workers [13] studied the eccentric loading behaviour of L-shaped concrete-filled steel tube columns, addressing failure modes and the interaction between constituent mono-columns. Tests were reproduced numerically, and a simplified design expression was proposed in which the eccentric load is resolved into three axial loadings carried by the individual mono-columns, with predicted capacities agreeing closely with measurement.

Ghafar and colleagues [14] tested nine L-shaped CFST columns in various cross-sectional combinations and confinement levels, reporting high load-carrying capacity, good ductility and efficient use of internal space.

Yang and Zhang [15] used ABAQUS to study cross-shaped concrete columns with built-in T-shaped steel and steel tubes. Calibrated against existing tests and applied to twenty-five specimens, the model reproduced hysteretic curves and failure modes with small error and yielded a capacity-calculation method — one of the few studies to move from validation to a usable design expression.

Three further numerical studies establish the parameters governing composite shaped sections. Mathew and Krishnachandran [16] found that each additional millimetre of tube thickness reduced deformation under constant load by approximately 8%, while increasing length raised it. Subhan [17] found that a concrete-encased steel core roughly doubled the lateral-load resistance of plain reinforced concrete. Paul and Vargheese [18] compared criss-cross columns connected by lacing bars, single plates and double plates, the last carrying the greatest load, with capacity falling as height increased and rising with plate width and confinement.

C. Finite Element Parametric Studies and Stability

Zhang, Shi, Wang and Zhou [19] investigated the buckling of T-shaped box-T section columns under axial load, combining experiments with parametric numerical studies. They characterised how the plate slenderness of the individual limbs controls the transition between local and global buckling, and proposed capacity expressions for the section. The finding reinforces a point implicit in much of the RC work: the limb of a shaped section is a slender outstand whose local behaviour may govern even when the member as a whole is stocky.

D. Prefabricated and Precast Shaped-Column Systems

The most recent stream concerns the industrialisation of shaped-column construction. Chen and co-workers [20] tested prefabricated L-shaped CFST columns of rectangular multi-cell section under lateral load applied in different directions. Loading direction significantly affected both hysteretic response and ductility, confirming at full scale that the biaxial nature of corner shaped-columns must be respected explicitly rather than approximated by orthogonal analysis.

Ni and colleagues [2] evaluated a full-scale precast L-shaped column frame with grouted connections, showing that the precast system can match monolithic construction provided the connection is properly executed. Liu, Wang and Sun [21] tested assembled beam-column joints reinforced with L-shaped steel bars and reported satisfactory energy dissipation and joint integrity, and Xu and co-workers [22] performed a shaking-table test on a prefabricated L-shaped column structure that retained overall stability under multi-directional input.

Taken together, this group establishes the seismic adequacy of prefabricated shaped-column framing and repeatedly identifies loading direction as the controlling variable.

E. Strengthening and Retrofitting of Shaped Columns

Because an existing stock now requires assessment, a fourth stream addresses retrofit. Salah and co-authors [23] found that external steel jacketing substantially raised both the axial capacity and the ductility of L-shaped RC columns. Chen, Wang and Zhao [24] analysed L-shaped columns strengthened at the base with carbon-fibre-reinforced-polymer slabs, varying loading angle, strengthening method, stirrup ratio and axial compression ratio, and showed that CFRP improved capacity and ductility under obliquely applied seismic action — the direction most critical for corner columns. Ahmed and colleagues [25] showed that wing walls markedly increased the lateral stiffness and strength of existing L-shaped columns while limiting drift.

F. Whole-Building and Frame-Level Comparative Studies

The stream most directly relevant here compares complete buildings framed with different column shapes. Yang, Liu and Huang [3] contrasted a specially shaped column frame with a rectangular frame of equal member area. The shaped sections possessed a larger moment of inertia and so produced a stiffer frame; under strong ground motion the corner columns experienced biaxial bending combined with axial force, yet the code-designed frame resisted the earthquake as effectively as its rectangular counterpart. This remains the clearest statement of the central proposition that shape can be exploited without penalty.

Navghare and Khedikar [26] compared rectangular, L- and T-shaped sections in a G+10 frame by response spectrum analysis in ETABS. The L-shaped columns attracted the greatest base shear in both principal directions but exhibited the smallest joint displacement, while the rectangular columns produced the least base shear and the largest displacement — a clear illustration of the stiffness-versus-force trade-off described in Section II.

Gumble and Pajgade [4] compared seismic performance and cost for four heights — six, nine, twelve and fifteen storeys — in zone III using equivalent static analysis. The shaped-column buildings displayed smaller displacements and drifts, offered greater corner carpet area and cost less to build. Ladvikar and Mundhada [7] reached the same general conclusion in review.

Agrawal and Pajgade [9] assessed G+14 buildings with shaped columns, with and without shear walls, in zone III. Adding shear walls alongside the shaped columns reduced both displacement and drift and produced a more economical structure than the shaped-column frame alone, indicating that the two strategies are complementary rather than alternative.

Shivaranjitha and Naveen Kumar [8] found that Y-shaped columns in an eight-storey building increased available floor area by about 20.5% and permitted nearly 40% fewer columns, freeing ground-level parking space, with the inclined limbs transferring axial load efficiently to the central stem. Kareem and Mathew [27] analysed a G+8 space frame built with T-shaped and with square columns in STAAD.Pro: displacements and drifts of the square-column model exceeded those of the T-column model, which attracted greater base shear.

Related work establishes the analytical context. Raju and co-workers [28] found that discontinuities in the vertical load path sharply increased base shear and storey displacement. Rahman and Islam [29] confirmed that response spectrum analysis captures the higher-mode contributions dominating the drift of taller frames — a result bearing directly on the choice of analysis method below. Kumar and Prasad [5] and Panchal and Marathe [30] examined plan configuration and material alternatives respectively, both reinforcing the finding that the section and arrangement of the vertical members strongly condition lateral response.

G. Design Aids, Code Provisions and Reliability

A final group addresses the translation of analysis into design. Shet, Shetty and Shetty [31] studied the axial force-moment interaction of C-shaped equal-legged columns using both ETABS and an analytical procedure. Capacity increased with the grades of steel and concrete, and — significantly for anyone validating software output by hand — differences between manual and software results were traced to the parabolic stress block used in hand calculation against the equivalent rectangular (Whitney) block adopted by the software. This provides the expected order of discrepancy for the validation proposed in Section VII.

Beck and Doria [32] evaluated the reliability of steel I-columns designed to the Brazilian standards using a nonlinear model including residual stresses, imperfections and plastic failure. The resistance curves gave uniform reliability across the permitted slenderness range at the reference load ratio, but not at other load ratios, owing to the partial load factors adopted. Although the study concerns steel I-sections, it illustrates a point that recurs below: the reliability implied by a code provision is not necessarily uniform across the parameter space to which it is applied.

On the code side, the Indian Standards provide the framework within which the proposed methodology operates. IS 1893 (Part 1) [6] governs the determination of seismic design forces, the response spectrum, the empirical estimation of the fundamental period and the drift limitation; IS 456 [33] governs the design of the reinforced concrete members; IS 875 (Parts 1 to 3) [34] governs dead, imposed and wind loading; and SP 16 [35] supplies the design aids used in the manual verification. None of these documents contains explicit provisions for non-rectangular column geometry, which is itself a finding of the review.

V. SYNTHESIS AND CRITICAL DISCUSSION

A. Consolidated Summary of the Reviewed Work

Table II consolidates the reviewed studies by theme, recording the scale of each investigation, the method employed, the parameters varied and the principal outcome. It is the evidential basis for the discussion that follows and for the gaps stated in Section VI.

TABLE II. CONSOLIDATED SUMMARY OF THE REVIEWED LITERATURE ON SPECIALLY SHAPED COLUMNS

Ref.	Scale	Method	Parameters varied	Principal finding
[1]	Member	Experimental	Fibre content, cyclic load	Steel fibres improve energy dissipation and crack control of T-section
[10]	Member	Test + FE (ANSYS)	Stirrup spacing, repeated axial load	Confinement strongest in stirrup plane; falls with spacing
[11]	Member	FE (MSC Marc)	Axial ratio, shear-span ratio, load angle	Ductility coefficient > 3; capacity falls with shear-span ratio
[12]	Member	Experimental (fire)	Axial ratio, eccentricity	Fire resistance 57% lower at axial ratio 0.6 than 0.2
[13]	Member	Test + FE	Eccentricity, mono-column interaction	Simplified three-axial-load design expression proposed
[14]	Member	Experimental	Section combination, confinement	High capacity and ductility for L-shaped CFST
[15]	Member	FE (ABAQUS)	Steel arrangement, 25 specimens	Capacity calculation method for cross-shaped composite section
[16], [17], [18]	Member	FE (ANSYS)	Tube thickness, steel core, connection type	Deformation falls ~8% per 1 mm of tube; steel core doubles lateral resistance
[19]	Member	Test + FE	Limb plate slenderness	Local-global buckling transition characterised
[20]	Member	Experimental	Lateral loading direction	Direction strongly governs hysteresis and ductility
[2]	Structure	Full-scale test	Grouted connection	Precast L-shaped frame comparable to monolithic
[21], [22]	Joint / structure	Test + shaking table	L-bar joints, multi-directional input	Good energy dissipation; prefabricated structure retains stability
[23], [25]	Member	Test / numerical	Steel jacketing, wing walls	Both retrofits raise capacity and stiffness, limiting drift
[24]	Member	FE	Load angle, CFRP, stirrup, axial ratio	CFRP improves oblique seismic capacity and ductility
[3]	Building	Seismic analysis	Shaped vs rectangular, equal area	Shaped frame stiffer; equal earthquake resistance

[26]	Building (G+10)	RSA (ETABS)	Rectangular, L, T	L-shape: highest base shear, least displacement
[4]	Building (6-15 st.)	ESA (ETABS)	Shape, building height, cost	Shaped columns: less drift, more area, lower cost
[7]	Building	Review	Column shape	Shaped columns generally reduce displacement and drift
[9]	Building (G+14)	ETABS	Shape, shear wall presence	Shear walls with shaped columns most economical
[8]	Building (8 st.)	ETABS	Y-shaped vs rectangular	20.5% more floor area; 40% fewer columns
[27]	Building (G+8)	STAAD.Pro	T vs square column	T-column model: lower displacement and drift
[28], [29]	Building	Dynamic / pushover	Load path, analysis method	Load-path discontinuity raises demand; RSA captures higher modes
[31]	Section	ETABS + analytical	Steel and concrete grade	Stress-block assumption explains manual-software difference
[32]	Member	Reliability + FE	Slenderness, load ratio	Code reliability not uniform across load ratios

Legend: ESA = equivalent static analysis; RSA = response spectrum analysis; FE = finite element; CFST = concrete-filled steel tube; CFRP = carbon-fibre-reinforced polymer.

B. Coverage of Response Quantities

Table III records which of the four design-governing response quantities each stream actually reports. The pattern it reveals is more informative than any individual result.

TABLE III. COVERAGE MATRIX OF REPORTED RESPONSE QUANTITIES BY RESEARCH STREAM

Research stream	Bending moment	Storey shear	Displacement	Inter-storey drift	Building height range
Member-level RC / SRC tests	Section level only	Section level only	Member tip only	Not reported	Not applicable
Composite / CFST members	Section level only	Not reported	Member tip only	Not reported	Not applicable
FE parametric / stability	Partially	Not reported	Member level	Not reported	Not applicable
Prefabricated systems	Not reported	Reported	Reported	Partially	Up to 3 storeys
Strengthening / retrofit	Section level only	Not reported	Member level	Partially	Not applicable
Whole-building comparisons	Rarely reported	Usually reported	Always reported	Usually reported	8 to 15 storeys
This review (proposed study)	Storey-wise	Storey-wise	Storey-wise	Storey-wise	G+12 (43.2 m)

C. Points of Agreement

Three findings are supported consistently across independent studies, methods and countries.

- Stiffness advantage. For equal cross-sectional area, shaped sections deliver greater flexural rigidity and therefore smaller displacement and drift at building scale — reported by frame-level studies using both analysis methods, across different software and codes [3], [4], [26], [27].

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- Space and cost benefit. The architectural gain is quantifiable and is not offset by a construction cost penalty; several studies report a net cost reduction alongside the carpet-area gain [4], [8].
- Adequate ductility when properly detailed. Ductility coefficients above three are consistently achieved for well-confined shaped sections, so the geometry is compatible with ductile seismic design provided the confinement detail is executed correctly [11], [14].

D. Points of Disagreement and Unresolved Questions

- Several matters remain contested or unresolved.
- Direction of the base shear effect. Some studies report that shaped columns attract higher base shear because they are stiffer [26], [27]; others report reduced storey shear [4]. The outcomes are not necessarily contradictory, since both depend on where the shortened period falls on the design spectrum and on whether area or overall dimension is held constant — but the basis of equivalence is seldom stated, which makes the results hard to reconcile.
- Sensitivity to loading direction. Member-level and prefabricated-system studies establish that the oblique direction governs [20], [24], yet whole-building studies almost universally load along the orthogonal axes only, so the building-scale consequence has not been quantified.
- Effect of building height. The only study to vary height systematically covered six to fifteen storeys by equivalent static analysis alone [4]. Whether the benefit grows, saturates or reverses as higher modes become significant remains open.
- Basis of equivalence. Studies variously equalise gross area, bounding dimension or reinforcement percentage. These bases are not equivalent, so reported percentage improvements are not directly comparable between publications.

VI. RESEARCH GAPS IDENTIFIED

The coverage matrix of Table III and the discussion of Section V support the following seven gaps, stated in the order of their significance for design practice.

- Gap 1 — Absence of storey-wise comparison for genuinely high-rise frames. Whole-building comparisons are confined almost entirely to frames of eight to fifteen storeys, most of which treat the building as a single entity rather than reporting response up the height. Where lateral effects dominate and higher modes contribute materially, a storey-by-storey account is required, because the shape that minimises roof displacement need not minimise the moment demand at the critical storey.
- Gap 2 — Incomplete reporting of the design-governing response set. No reviewed study reports bending moment, storey shear, displacement and drift together for the same models under identical loading. Displacement is almost always reported, drift usually, shear frequently and moment rarely, so the trade-off between serviceability improvement and reinforcement demand cannot be assessed from a single consistent source.
- Gap 3 — Member-level findings not translated to building scale. The experimental literature is detailed but expressed in section-level quantities — hysteretic loops, skeleton curves, ductility coefficients — that do not map onto the storey-wise quantities used in design. The bridge between the two scales has not been built.
- Gap 4 — Oblique loading not examined at building scale. Loading direction is repeatedly identified as the controlling variable for shaped columns at the member level, but building-level analyses continue to apply lateral load along the orthogonal axes only.
- Gap 5 — Single analysis method in most comparisons. Most frame-level comparisons employ either the equivalent static or the response spectrum method, not both. The two distribute lateral force differently up the height, and the shaped-column frame has a different fundamental period, so conclusions drawn from one method may not hold under the other.
- Gap 6 — Limited independent verification of software results. Comparative frame studies rarely verify finite element output against an independent calculation. Where verification has been attempted at section level, systematic differences attributable to the assumed stress block were identified [31], so unverified output should not be treated as definitive.
- Gap 7 — Absence of specific code provisions. Neither the Indian Standards nor most national

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codes address non-rectangular column geometry, confinement of re-entrant corners or the oblique critical direction. Design proceeds by analogy with rectangular sections, an approach whose reliability across the parameter space has not been established and which [32] suggests should not be assumed.

The methodology proposed in the next section is designed to address Gaps 1, 2, 3, 5 and 6 directly within a single coherent investigation, and to provide the quantitative basis from which Gaps 4 and 7 can subsequently be attacked.

VII. PROPOSED METHODOLOGY

A. Objectives

The investigation is framed around four objectives, each traceable to a gap identified above.

- To study a G+12 reinforced concrete building under seismic and wind action, at a height at which lateral effects govern the design (Gap 1).
- To determine the storey displacements, inter-storey drifts, storey shears and bending moments of the building for rectangular, T-shaped and plus-shaped column configurations, reporting all four quantities storey by storey for the same models under identical loading (Gaps 1, 2, 3).
- To compare the shaped-column buildings with the rectangular baseline using both the equivalent static and the response spectrum method, and to identify the most efficient configuration (Gap 5).
- To validate the finite element results obtained from ETABS against an independent manual design of the governing member (Gap 6).

B. Reference Building and Model Configurations

A single reference building is adopted and modelled in three variants identical in every respect except column shape. Holding the plan, storey height, beam grid, slab, material grades and loading constant isolates the effect of shape, so that any difference in computed response is attributable to that variable alone. The basis of equivalence adopted — identical overall building geometry and identical gravity loading, with the column section replaced in situ — is stated explicitly here, since Section V-D showed that failure to state it is what makes published percentage improvements incomparable.

The three configurations are:

- Model A — G+12 frame with rectangular columns, 230 mm × 600 mm, serving as the baseline.
- Model B — G+12 frame with T-shaped columns of overall dimension 350 mm × 600 mm, placed at wall junctions.
- Model C — G+12 frame with plus-shaped columns of overall dimension 350 mm × 750 mm, distributing material symmetrically about both principal axes.

A twelve-storey building with an overall plan of 63.20 m × 29.50 m and a total height of 43.2 m is selected as representative of high-rise residential construction. The geometric particulars common to all three models are given in Table IV and the material properties in Table V.

TABLE IV. GEOMETRIC DETAILS AND DIMENSIONS OF THE PROPOSED BUILDING MODELS

Parameter	Value
Type of structure	Ordinary moment-resisting RC frame
Grade of concrete	M 40 (fck = 40 N/mm2)
Grade of reinforcing steel	Fe 415
Plan area	63.20 m × 29.50 m
Number of storeys	G+12
Total height of building	43.2 m
Floor-to-floor height	3.6 m
Rectangular column (Model A)	230 mm × 600 mm
T-shaped column (Model B)	350 mm × 600 mm
Plus-shaped column (Model C)	350 mm × 750 mm
Beam size	300 mm × 500 mm
Slab thickness	200 mm
Wall thickness	230 mm
Density of concrete	25 kN/m3
Live load on floor / roof	3.0 / 1.5 kN/m2
Floor finish	1.0 kN/m2

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Seismic zone / soil condition	Zone II / medium soil (Type II)

TABLE V. MATERIAL PROPERTIES ADOPTED IN THE PROPOSED ANALYSIS

S. No.	Property	Value
1	Grade of concrete	M 40
2	Young modulus of concrete, Ec	2.17 × 10^4 N/mm2
3	Poisson ratio	0.17
4	Ultimate tensile strength of steel	505 MPa
5	Yield tensile strength of steel	415 MPa
6	Modulus of elasticity of steel	2.0 × 10^5 MPa

C. Loading and Code Framework

Four load cases are applied to each model: dead load, live load, seismic load and wind load. The gravity loads are identical across the three models; the lateral loads depend on the mass and stiffness of each model and are computed by the software from the assigned section properties, which is precisely the mechanism by which column shape enters the comparison.

Dead loads comprise self-weight together with wall, parapet, slab and finish loads to IS 875 (Part 1) [34]: for a 3.5 m storey the wall load is 10.00 kN/m and the parapet 5.00 kN/m, the 200 mm slab contributes 5.00 kN/m2 and the finish 1.00 kN/m2. Imposed loads follow IS 875 (Part 2) [34] at 3.0 kN/m2 on typical floors, of which 0.75 kN/m2 enters the seismic mass as IS 1893 (Part 1) [6] requires.

Seismic loading follows IS 1893 (Part 1) [6]. The approximate fundamental period is estimated from Clause 7.6.2, T = 0.09h / sqrt(d), and refined by the modal analysis; the design horizontal acceleration coefficient follows from the corresponding response spectrum ordinate for medium soil. Wind loading follows IS 875 (Part 3) [34], the design pressure varying with height according to terrain category and basic wind speed. Both sets of parameters are summarised in Table VI.

TABLE VI. SEISMIC AND WIND PARAMETERS ADOPTED IN THE PROPOSED METHODOLOGY

Parameter	Value	Source clause
Seismic zone / zone factor Z	Zone II / 0.10	IS 1893 (Part 1), Table 3
Damping ratio	0.05	IS 1893 (Part 1), Cl. 7.2.4
Importance factor, I	1.0	IS 1893 (Part 1), Table 8
Response reduction factor, R	5.0	IS 1893 (Part 1), Table 9
Soil / site type	Medium (Type II)	IS 1893 (Part 1), Cl. 6.4.2
Fundamental period, X-direction	0.340 s	IS 1893 (Part 1), Cl. 7.6.2
Fundamental period, Z-direction	0.420 s	IS 1893 (Part 1), Cl. 7.6.2
Spectral acceleration coefficient, Sa/g	2.5 (flat portion)	IS 1893 (Part 1), Fig. 2
Basic wind speed, Vb	39 m/s	IS 875 (Part 3)
Terrain category	Category 2	IS 875 (Part 3)
Risk coefficient, k1	1.00	IS 875 (Part 3)
Topography factor, k3	1.00	IS 875 (Part 3)
Design wind pressure at 43.2 m	approx. 1.05 kN/m2	IS 875 (Part 3)

D. Analysis Procedures

Two analysis procedures are applied to every model, which is the feature that addresses Gap 5. The equivalent static method distributes the design base shear in proportion to the product of storey mass and the square of storey height, giving the code-compliant envelope. The response spectrum method performs a modal analysis, combines modal responses by the complete quadratic combination rule, and captures the higher-mode contributions that become significant at this height, the necessity of which is demonstrated by [29]. Sufficient modes are included to mobilise at least ninety per cent of the seismic mass in each principal direction [6], and where the dynamic base shear falls below the static value the dynamic results are scaled as the code requires.

ETABS is adopted because its object-based modelling, its facility for editing similar storeys simultaneously and, above all, its Section Designer module — which permits arbitrary non-rectangular sections to be constructed and their properties computed directly — suit a study in which the column section is the independent variable. P-delta effects are included and the base is modelled as fixed.

E. Sequence of Work

The complete sequence is shown in Fig. 3. It is arranged so that the controlled-variable condition is established before any load is applied, and so that verification precedes comparison rather than following it. The three models diverge only at the section-definition stage and are recombined under a single identical loading block, which is what guarantees that the differences reported in Section VIII are attributable to column shape alone.

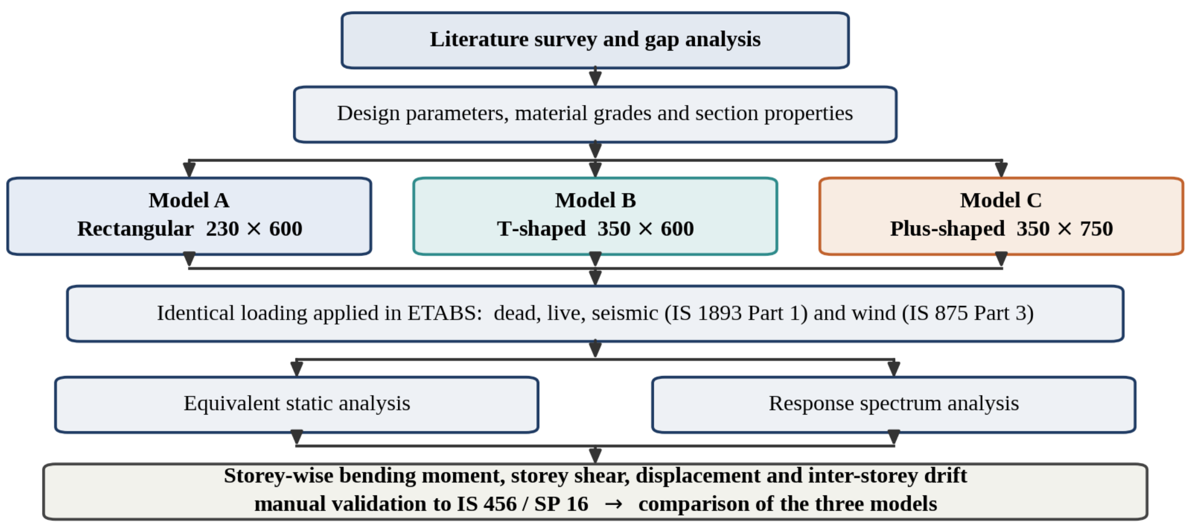

Fig. 3. Flowchart of the proposed research methodology.

F. Response Quantities and Comparison Metrics

Four response quantities are extracted at every storey for every model: the maximum bending moment in the columns, the maximum storey shear, the maximum lateral displacement, and the inter-storey drift ratio computed as the relative horizontal displacement between adjacent floors divided by the storey height. Reporting all four for the same models under identical loading is what addresses Gap 2, and reporting them storey by storey rather than as single building-level figures is what addresses Gaps 1 and 3.

Comparison is expressed as the percentage change in each peak quantity relative to Model A, the rectangular baseline. Drift is additionally compared against the code limitation of 0.4% of the storey height stipulated by IS 1893 (Part 1) [6], since compliance rather than relative improvement is the criterion that governs serviceability design.

G. Validation Strategy

The governing column of the rectangular model is designed independently by hand to IS 456 [33] with the charts of SP 16 [35], and the resulting moment, shear, axial force and displacement compared with the software output. This addresses Gap 6. The expected order of discrepancy is known in advance from [31]: the hand calculation uses the parabolic stress block while the software adopts the Whitney block, so a small systematic difference in the flexural quantities is anticipated and its magnitude, rather than its existence, is the diagnostic. A larger divergence would indicate a

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modelling error requiring investigation before any comparative conclusion is drawn.

VIII. ILLUSTRATIVE APPLICATION AND OUTCOMES

A. Purpose of the Illustration

The methodology has been applied to the reference G+12 building of Section VII, with outcomes reported below in the storey-wise form the review found missing from the literature. They demonstrate the framework rather than establishing a general design rule, since they relate to one plan, one seismic zone and one set of section dimensions.

B. Storey-Wise Response of the Three Models

Table VII presents the three primary response quantities together, storey by storey. The consolidated form is deliberate: it is the arrangement the coverage matrix of Table III identified as absent from the literature, and it allows the trade-off between reduced displacement and reduced reinforcement demand to be read directly rather than assembled from separate sources.

TABLE VII. CONSOLIDATED STOREY-WISE RESPONSE OF THE THREE MODELS

Storey	Maximum bending moment (kN.m)			Maximum storey shear (kN)			Maximum displacement (mm)		
	A (Rect.)	B (T)	C (Plus)	A (Rect.)	B (T)	C (Plus)	A (Rect.)	B (T)	C (Plus)
12	826.76	734.17	566.81	941.85	840.43	821.54	88.84	66.16	64.90
11	814.58	705.63	553.84	923.25	813.87	797.91	83.19	60.46	58.52
10	789.44	677.09	540.87	904.65	787.31	771.23	77.54	54.76	51.22
9	770.21	648.55	527.90	886.05	760.75	748.90	71.89	49.06	46.31
8	719.30	620.01	514.93	867.45	734.19	712.45	66.24	43.36	39.67
7	698.67	591.47	501.96	848.85	707.63	694.77	60.59	37.66	32.90
6	675.20	562.93	488.99	830.25	681.07	672.85	54.94	31.96	27.71
5	629.10	534.39	476.02	811.65	654.51	637.31	49.29	26.26	21.48
4	559.89	505.85	463.05	793.05	627.95	619.21	43.64	20.56	17.54
3	578.90	477.31	450.08	774.45	601.39	583.36	37.99	14.86	11.76
2	524.99	468.00	437.11	755.85	574.83	554.45	32.34	9.16	7.87
1	510.22	461.05	424.14	737.25	548.27	528.77	26.69	3.46	1.21
Base	-	-	-	-	-	-	0.00	0.00	0.00

Note: Model A - rectangular columns (baseline); Model B - T-shaped columns; Model C - plus-shaped columns. Storey 12 is the roof level.

C. Bending Moment and Storey Shear

The bending moment is largest at the upper storeys, where lateral displacement and hence frame action are greatest, and decreases towards the base. At every storey the rectangular columns attract the highest moment, the T-shaped an intermediate value and the plus-shaped the lowest. At the critical storey the moment falls from 826.76 kN.m in Model A to 734.17 kN.m in Model B and 566.81 kN.m in Model C, a reduction of approximately 31 per cent. The lower flexural demand implies a saving in longitudinal reinforcement, so the space advantage of shaped columns is accompanied by a material advantage rather than paid for by one.

The storey shear behaves in the opposite sense along the height, being greatest at the base and diminishing towards the top. The rectangular columns carry the highest shear at every level, the reduction persisting down the full height: at the top storey the shear falls from 941.85 kN to 840.43 kN and 821.54 kN respectively, implying lower demand on transverse reinforcement. This outcome sits on one side of the disagreement identified in Section V-D. The shaped models here attract less shear rather than more, which follows from the basis of equivalence adopted, since their larger gross area shifts the period to a lower spectral ordinate. Studies holding gross area constant may reasonably obtain the opposite sign — which is precisely why the basis must be stated.

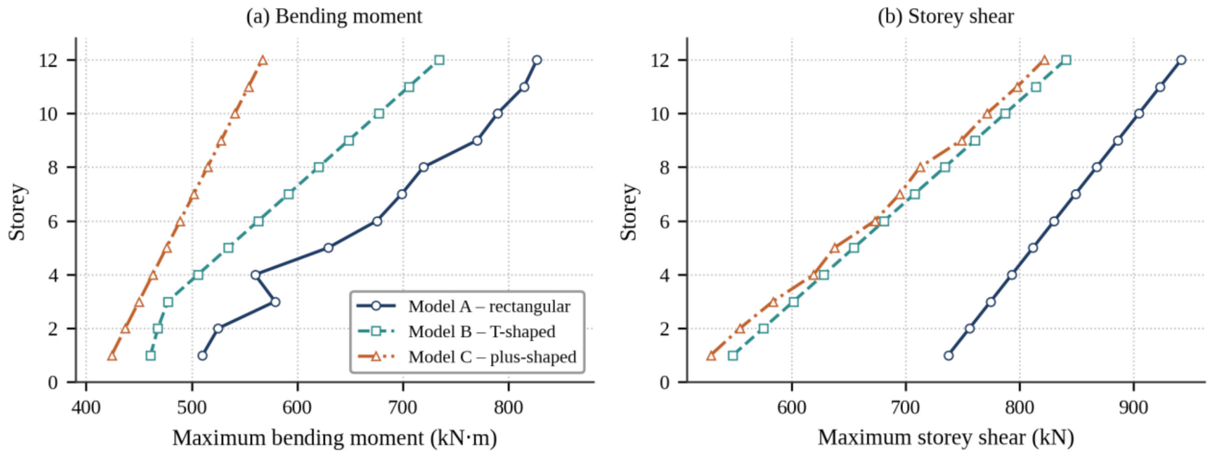

Fig. 4. Variation of maximum bending moment and maximum storey shear along the building height.

Fig. 4(a) also exposes a local irregularity in the baseline at the third storey, where the computed moment slightly exceeds that of the storey above. It is confined to Model A and affects neither the peak nor the comparison, but is retained rather than smoothed, since a review that recommends independent verification should not silently correct its own data.

D. Displacement and Inter-Storey Drift

Displacement grows monotonically with height and peaks at the roof, falling from 88.84 mm in Model A to 66.16 mm in Model B and 64.90 mm in Model C — reductions of approximately 25.5 and 26.9 per cent. The improvement is more pronounced still in the lower storeys, where Model C displaces only 1.21 mm at the first storey against 26.69 mm for Model A, confirming markedly greater lateral stiffness where the accumulated overturning demand is highest.

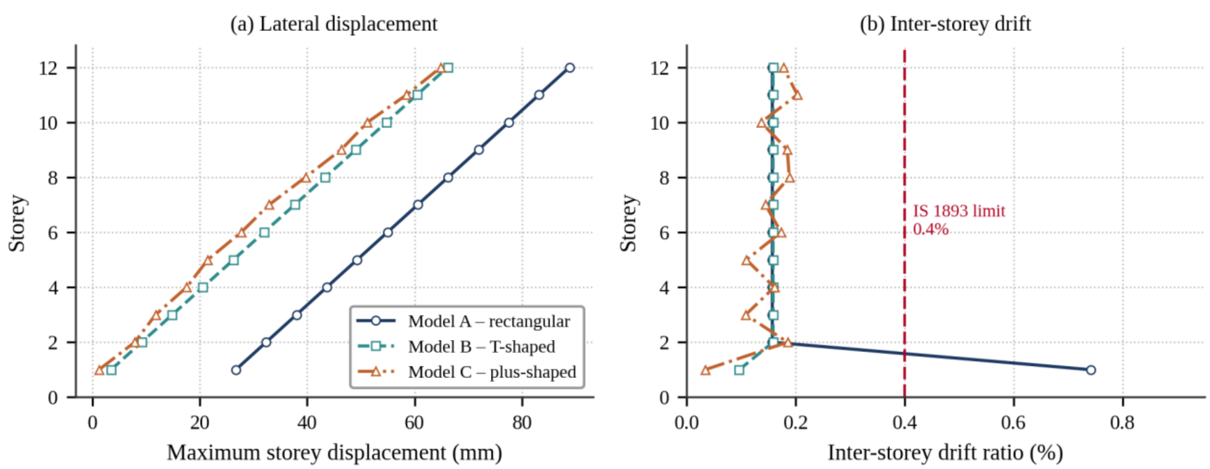

Fig. 5. Lateral displacement profile and inter-storey drift ratio, with the IS 1893 drift limitation shown.

Inter-storey drift, derived from the displacement data, is the quantity that most directly reflects the serviceability of a tall building and the demand imposed on non-structural components. Fig. 5(b) shows that all three models remain close to 0.15 per cent through the upper and middle storeys, but that the profiles separate sharply at the first storey: Model A reaches approximately 0.74 per cent, exceeding the 0.4 per cent limitation of IS 1893 (Part 1) [6], while Models B and C remain below 0.2 per cent throughout. This is the most consequential outcome of the illustration, because it converts a relative improvement into a compliance difference. The rectangular baseline would require remedial stiffening at the lowest storey; the shaped-column configurations would not.

E. Summary of Peak Responses

Table VIII and Fig. 6 collect the peak values and the change relative to the baseline. Model C is the most effective overall, and the plus-shaped section, which distributes material symmetrically about both principal axes, therefore delivers the best combination of stiffness and strength while freeing corner floor area for occupation. Axial force varied only marginally between models, as expected, since the gravity loading was identical.

TABLE VIII. PEAK RESPONSE SUMMARY AND REDUCTION RELATIVE TO RECTANGULAR COLUMNS

Response quantity	Model A	Model B	Model C
Peak bending moment (kN.m)	826.76	734.17	566.81
Reduction in moment (%)	baseline	11.2	31.4
Peak storey shear (kN)	941.85	840.43	821.54
Reduction in shear (%)	baseline	10.8	12.8
Peak displacement (mm)	88.84	66.16	64.90
Reduction in displacement (%)	baseline	25.5	26.9
Peak inter-storey drift (%)	approx. 0.74	below 0.20	below 0.20

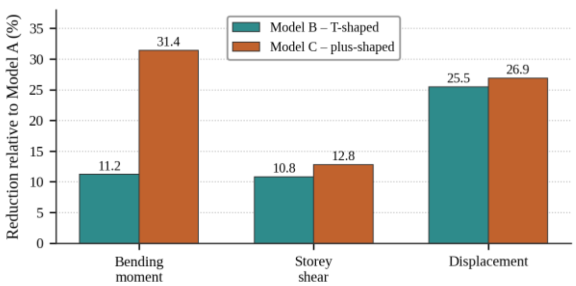

Fig. 6. Percentage reduction in peak response relative to the rectangular baseline.

F. Validation Against Manual Design

The governing column of the rectangular model was designed independently by hand to IS 456 [33] using the design charts of SP 16 [35], and the results are compared with the software output in Table IX. The manual maximum displacement of 88.87 mm lies within 0.03% of the ETABS value of 88.84 mm, and the manual bending moment and shear force are of the same order as the software values. The small differences arise chiefly from the stress-block assumption, as anticipated from [31]. The agreement confirms that the models are correctly configured and that the comparative conclusions can be relied upon — which is the purpose the validation step was designed to serve.

TABLE IX. MANUAL ANALYSIS AND DESIGN DATA FOR THE GOVERNING MEMBER

Design quantity	Manual value	ETABS value	Difference
Maximum bending moment	653.87 kN.m	—	Stress-block dependent
Maximum shear force	935.43 kN	941.85 kN	0.68%
Maximum axial force	1062 kN	—	Gravity governed
Maximum displacement	88.87 mm	88.84 mm	0.03%

IX. LIMITATIONS AND FUTURE SCOPE

Three limitations bound the generality of the conclusions.

- The comparison is conducted at a single building height, in a single seismic zone, on a single plan. Since the benefit is largest in the lower storeys it is evidently height-dependent, so extrapolation to substantially taller or shorter buildings is not warranted without further analysis.
- Lateral load is applied along the orthogonal axes only. Since the member-level literature identifies the oblique direction as critical, the methodology does not close Gap 4 and the reported

improvements may be optimistic for corner columns.
- The analysis is linear elastic, so ductility, energy dissipation and the sequence of plastic hinge formation — the quantities the experimental literature measures most carefully — are not captured. The comparison establishes stiffness and force distribution rather than collapse performance.

These limitations map onto the priorities for further work: extension to a range of building and storey heights to establish how the benefit scales; application of lateral load at intermediate angles across the quadrant to quantify at building scale the oblique effect that [20] and [24] establish at member scale; nonlinear pushover and time-history analysis to connect the frame-level comparison to the ductility findings of [11] and [14]; introduction of shear walls or bracing alongside the shaped columns, following [9], to test whether the two strategies remain additive at this height; and comparison against an independent software package as a further layer of verification.

X. CONCLUSIONS

This paper has reviewed the literature on specially shaped reinforced concrete columns under lateral loads, identified the gaps in it, and proposed a methodology to close them. The following conclusions are drawn.

- The literature agrees on three points: shaped sections deliver greater flexural rigidity and therefore smaller displacement and drift; the gain in usable floor area is real, of the order of fifteen to twenty per cent at wall junctions, and is not offset by a cost penalty; and properly confined shaped sections achieve displacement ductility coefficients above three.
- It disagrees, or is silent, on four: the direction of the base shear effect, which turns on an equivalence basis publications rarely state; the building-scale consequence of oblique loading; the variation of the benefit with building height; and the reliability of designing shaped sections by analogy with rectangular ones absent specific code provisions.
- The most significant gap is that no reviewed study reports bending moment, storey shear, displacement and drift together, storey by storey, for a genuinely high-rise frame in which column shape is the only variable.
- The proposed methodology addresses this gap through three geometrically identical G+12 models differing only in column shape, analysed under identical loading by both the equivalent static and the response spectrum method in ETABS, with all four response quantities extracted storey-wise and validated against an independent manual design.
- The illustrative application confirms that the framework produces the required class of result. Relative to the rectangular baseline the plus-shaped configuration reduced peak bending moment by approximately 31%, peak storey shear by approximately 13% and peak roof displacement by approximately 27%, from 88.84 mm to 64.90 mm, with the T-shaped configuration intermediate. Critically, the rectangular model exceeded the code drift limitation at the first storey while both shaped models remained well within it.
- Agreement between manual and software results to within 0.03% on displacement confirms that the small residual differences in flexural quantities are attributable to the stress-block assumption rather than to modelling error.
- Subject to the stated limitations, specially shaped columns — the plus-shaped section in particular — are a viable and economical alternative to rectangular columns in high-rise construction, improving lateral performance while releasing usable floor area. The priority for further work is to quantify at building scale the oblique loading effect that the member-level literature has already established as critical.

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