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Effect of Frame–Shear Wall Stiffness Distribution on Lateral Displacement of Reinforced Concrete High-Rise Buildings

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Abstract: *This study presents a systematic optimization methodology for minimizing lateral displacement in reinforced concrete (RC) high-rise buildings subjected to combined wind and seismic loading. Grounded in Smith–Coull frame–wall interaction theory and validated through three-dimensional finite element analysis (ETABS 2022), the procedure was applied to a representative 20-story RC structure (H = 72 m) in Thai Nguyen, Vietnam, designed per TCVN 2737-2023 and TCVN 9386:2012. A two-stage optimization strategy was developed: (i) maximizing the non-dimensional stiffness parameter αH through cross-sectional area redistribution at constant concrete volume, and (ii) determining the rational shear-wall termination elevation via bending-moment inflection-point analysis. Redistributing member sections (columns: 800×800 → 750×750 mm; beams: 300×650 → 300×850 mm) reduced roof-level lateral displacement by 18.9% (47.6 → 38.6 mm) and dynamic wind load by 10.1%, attributable to a shortened fundamental period (T_1: 2.10 → 1.90 s). Terminating shear walls at Story 15 (~75% of building height) reduced structural mass by 2.85% (170 t) with only a 1.8% displacement increase. The combined configuration satisfies the TCVN 5574:2018 drift limit at a 40.9% utilization ratio, establishing it as the techno-economic optimum. The proposed procedure is directly applicable to RC high-rise buildings of 15–30 stories in wind–seismic regions.*

Keywords: *RC high-rise buildings; frame–shear wall system; lateral displacement; stiffness optimization; ETABS; TCVN 2737-2023.*

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I. INTRODUCTION

Rapid urbanization in Vietnam over the past two decades has driven sustained growth in the construction of reinforced concrete (RC) high-rise buildings, particularly in Hanoi, Ho Chi Minh City, and secondary urban centers such as Thai Nguyen. As building height increases, lateral loads—wind and seismic actions—progressively dominate structural behavior, imposing stringent requirements on stiffness, stability, and occupant comfort [1, 3, 4].

The frame–shear wall system is the most widely adopted lateral-load-resisting configuration for RC high-rise buildings in Vietnam. Its appeal lies in combining the architectural flexibility of a moment-resisting frame with the high lateral stiffness of reinforced concrete shear walls. TCVN 5574:2018 mandates that the inter-story drift ratio satisfy $\delta/H \leq 1/750$ for frame–wall systems, a criterion that directly bounds both safety and serviceability [5].

A central—and incompletely resolved—design problem is determining the optimal stiffness partition between frame and wall such that lateral drift is minimized for a given total material quantity. Excessive wall stiffness wastes concrete and increases seismic mass, whereas insufficient wall stiffness causes drift non-compliance. Furthermore, design practice commonly maintains shear walls at full building height without examining whether the upper-story walls still contribute meaningfully to drift control or instead act as parasitic mass.

Existing Vietnamese studies have made preliminary contributions. Nguyen Van Quang and Nguyen Quang Tung (2018) [2] investigated frame–wall stiffness partitioning for drift reduction; however, their analysis was confined to a planar model and did not simultaneously address dynamic wind load per TCVN 2737-2023 or seismic load per TCVN 9386:2012. International research by Smith and Coull [7] established the theoretical framework for frame–wall interaction via the dimensionless parameter αH, but application to structures governed by Vietnamese loading standards has not been demonstrated.

This paper addresses these gaps through a methodology that integrates Smith–Coull interaction theory, constraint-based cross-section optimization, bending-moment inflection-point analysis, and full three-dimensional FEM verification in ETABS 2022. The specific objectives are: (1) to derive the analytical relationship between αH and roof-level lateral displacement; (2) to formulate a cross-section redistribution procedure that maximizes αH at constant concrete volume; (3) to determine the rational shear-wall termination elevation from inflection-point analysis; and (4) to quantify the structural and economic performance of four design alternatives under combined wind and seismic loading.

II. NOVELTY AND CONTRIBUTIONS

This study makes the following original contributions to the field of RC high-rise structural design:

(1) Analytical–numerical integration under dual loading: For the first time in the Vietnamese technical literature, Smith–Coull frame–wall interaction theory is rigorously applied alongside simultaneous 3D FEM analysis under both dynamic wind loading (TCVN 2737-2023, gust factor method) and response-spectrum seismic loading (TCVN 9386:2012, DCM, q = 3.9), capturing the competing stiffness–mass trade-off that neither approach alone can resolve.

(2) Volume-constrained stiffness maximization: A practical beam-to-column cross-section redistribution procedure is formalized, in which the total concrete volume is held strictly constant while αH is maximized. This isolates the pure stiffness effect from any material increase and provides engineers with an actionable, code-compliant design method.

(3) Inflection-point-based wall termination criterion: An analytically derived rule for the rational shear-wall termination elevation—grounded in the zero-moment (inflection) point of the wall bending-moment diagram—is proposed and validated numerically. This criterion eliminates the common conservative practice of maintaining full-height walls, yielding a quantified 2.85% mass saving with only 1.8% drift increase.

(4) Dual aerodynamic–seismic interaction quantified: The study explicitly quantifies the competing secondary effects of stiffness increase: a 10.1% reduction in dynamic wind load (shorter period reduces gust factor) versus a 5.1% increase in seismic base shear (greater stiffness attracts more earthquake force). This trade-off analysis is absent from prior Vietnamese high-rise studies.

(5) Directly transferable design procedure: The four-step process (preliminary sizing → αH maximization → inflection-point wall cut → FEM verification) is presented in a form directly usable by practicing engineers in Vietnam, supported by complete numerical data for a representative 20-story benchmark building.

III. METHODOLOGY

3.1. Building model and design parameters

The study object is a 20-story reinforced concrete high-rise building with a typical floor height of h = 3.6 m, a total height of H = 72 m, a column spacing of L = 8 m, and a rigid core located at the center of the floor plan. The materials used include B25 concrete (E_b = 30.000 MPa, f_{ck} = 20 MPa) and CB300V longitudinal reinforcement (E_s = 200.000 MPa).

The applied load system is determined according to current standards: structural dead load and service live load q = 300 daN/m²; wind load according to TCVN 2737-2023 with base wind pressure W_0 = 95 daN/m² (Zone II, terrain type B); and earthquake load according to TCVN 9386:2012 with design ground acceleration ag = 0.9104 m/s² on type B soil in Thai Nguyen. The structure belongs to the medium ductility class (DCM) with a structural behavior coefficient q = 3.9 for a hybrid equivalent frame system.

The fundamental vibration period T is checked to satisfy the condition $0.06n \le T \le 0.1n$ ($1.2\ s \le T \le 2.0\ s$ with n = 20 floors), ensuring reasonable seismic design.

3.2. Frame-wall interaction theory and stiffness parameter αH

In frame-wall structural systems subjected to lateral loads, the rigid wall works according to the flexural cantilever mechanism, while the frame works according to the shear cantilever mechanism. The disharmony in deformation form between the two systems creates interaction reactions that are transmitted through the rigid floor system, leading to redistribution of loads and stresses between the frame and wall along the height [7].

The differential equation of horizontal displacement y(z) of a homogeneous frame-wall system subjected to a uniformly distributed wind load w is established according to Smith and Coull's theory [7]:

$$EI \cdot y''''(z) - GA \cdot y''(z) = w(z) \quad (1)$$

Where: EI is the total bending stiffness of the shear wall (kN·m²), GA is the equivalent shear stiffness of the frame system (kN). The analytical solution of equation (1) for the displacement at the apex y(H) is a function of the following important parameter:

$$\alpha H = H \cdot \sqrt{(GA / EI)} \qquad (2)$$

The parameter αH represents the relative stiffness ratio between the frame and the wall. As αH increases, the frame's contribution to lateral load bearing increases, leading to a decrease in overall lateral displacement. Analysis of the function y(H) shows that this is a decreasing function of αH: with a constant total amount of concrete material (V = const), the optimization problem is to maximize αH by adjusting the cross-sectional distribution ratio between beams and columns.

Another important characteristic is the location of the inflection point of the shear wall—the elevation at which the bending moment in the wall changes sign from positive to negative from base to top. At and above this point, the shear wall shifts from its role of resisting bending to supporting the shear frame with very low efficiency, while increasing the building mass and consequently increasing the base shear force due to earthquakes. This is the theoretical basis for proposing the reduction of shear walls at higher levels.

3.3. Methods for optimizing cross-sections and analysis procedures

The study applies the following four-step process:

Step 1 – Preliminary Selection: Determine the preliminary cross-sections of beams, columns, and walls using empirical formulas and design regulations. The preliminary total concrete volume V is calculated as a fixed constraint.

Step 2 – Optimization of Frame Cross-Section Distribution: Keep V = const, simultaneously adjust the dimensions of beams and columns to maximize αH. When increasing the beam cross-section height, the bending stiffness of the beam increases cubically, significantly improving GA; at the same time, reduce the corresponding column cross-section to preserve volume. This process is carried out by surveying the function y(H) according to the variation of cross-section in the constraint space.

Step 3 – Determining Wall Reduction Elevations: Determine the location of the bending inversion point from the results of the analysis of the moment diagram in the wall as a function of height, as a basis for proposing the optimal wall reduction elevation.

Step 4 – Numerical Verification: Perform finite element analysis for the four survey cases with full loads using ETABS 2022, comparing displacements, vibration frequencies, applied loads, and structural mass.

Wind load is determined according to TCVN 2737-2023 using the following formula:

$$W_k = W_{3s,10} \cdot k_{(ze)} \cdot c \cdot G_f \quad (3)$$

In which: the shock effect coefficient Gf is fully calculated considering the resonance component for structures with T > 1 s (reinforced concrete structure damping coefficient $\beta = 0.02$).Earthquake loads are determined using the response spectrum analysis method according to TCVN 9386:2012, considering 3 modes of vibration in each direction (total effective mass > 90%). Applied design spectral parameters: foundation type B (S = 1.20; TB = 0.15 s; TC = 0.50 s; TD = 2.0 s).

IV. RESEARCH RESULTS

4.1.Preliminary cross-section selection results

The preliminary cross-section is determined according to the standard design procedure for reinforced concrete high-rise buildings: main beam b×h = 300×650 mm^2; square column b×h = 800×800 mm^2; shear wall thickness t = 200 mm. The total concrete volume of the entire frame-wall system V = 69.60 m^3. The fundamental vibration period T_1 = 1.992 s satisfies the verification condition range [1.2; 2.0] s. Thus, the reasonableness of the initial cross-section selection is confirmed.

4.2.Optimize the distribution of frame cross-sections with V = const.

Through surveying the peak displacement function y(H) according to the beam-column cross-sectional distribution ratio with the constraint V = const, the study determined the theoretical minimum point at beam cross-section b×h = 300×1190 mm^2 and column cross-section b×h = 650×650 mm^2, for y(H) = 0.05 m and αH = 4.12. However, this beam size is not architecturally feasible because the beam height/floor height ratio is approximately 33%, failing to meet the required clear height.

After considering architectural constraints, the optimal feasible cross-sections were determined as: beam b×h = 300×850 mm^2, column b×h = 750×750 mm^2. This option gives αH = 3.53 and y(H) = 0.07 m—the optimal balance point between structural efficiency and construction requirements.

Four complete case studies are summarized in Table 1.

Table 1. Summary of vibration and horizontal displacement characteristics at the top of the structure.

Comparison criteria	TH1 (Preliminary)	TH2 (Optimize cross-section)	TH3 (Cutting the floor wall 15+)	TH4 (Overall optimization)
Column cross-section (mm×mm)	800×800	750×750	750×750	750×750
Beam cross-section (mm×mm)	300×650	300×850	300×850	350×900 (T14–20)
Shear wall	t = 200 mm,	t = 200 mm,	Cut from the	Cut from the 15th

Comparison criteria	**TH1 (Preliminary)**	**TH2 (Optimize cross-section)**	**TH3 (Cutting the floor wall 15+)**	**TH4 (Overall optimization)**
	total height	total height	15th floor upwards.	floor upwards.
Oscillation period T (s)	2,10	1,90	1,905	1,91
Oscillating displacement yH (mm)	44,5	42,2	44,1	42,0
Actual displacement at the apex (mm)	**47,6**	**38,6**	**39,3**	**38,0**
Reduction compared to TH1 (%)	—	**↓ 18,9%**	↓ 17,4%	↓ 20,2%

4.3.Analysis of Lateral Loads and Secondary Effects

Table 2 summarizes the total wind loads and bottom shear forces due to earthquakes for the four survey cases, reflecting the secondary effects when changing the stiffness and mass of the structure.

Table 2. Comparison of lateral loads and structural mass between cases.

Criteria	**TH1**	**TH2**	**TH3**	**TH4**
Total wind load F (kN)	1.375	1.236	1.360	1.360
Difference compared to TH1	—	**↓ 10,1%**	↑ 1,1%	↑ 1,1%
Frame-wall mass (Ton)	7.620	7.620	7.450	7.615
Weight savings compared to TH1	—	—	**↓ 170 t (2,85%)**	↓ 5 t (0,07%)
Bottom shear force due to earthquake Fb (kN)	2.550	2.680	2.608	2.670

Notably, TH2 achieved the lowest total wind load among the four cases (1,236 kN, a 10.1% reduction compared to TH1 = 1,375 kN), due to the oscillation period decreasing from 2.10 s to 1.90 s, thus reducing the shock effect coefficient Gf.

Conversely, the earthquake-induced bottom shear force of TH2 increased slightly by 5.1% compared to TH1 (2,680 kN versus 2,550 kN), consistent with the seismic resistance principle: structures with greater stiffness attract more earthquake forces, but simultaneously experience smaller displacements. To visualize this dual effect, Table 5 summarizes the combined efficiency index of each option—considering both wind load reduction, mass savings, and displacement reduction.

Table 3 presents the wind load distribution along the height for Case 1 (representative reference case), where floor 15 is the elevation determined for shear wall reduction based on inflection point analysis.

Table 3. Distribution of wind load according to height (Case 1, representative quote)

Story	**z Elevation (m)**	**k(ze)**	**Wtt (kN/m²)**	**Mj (Ton)**	**yji (mm)**	**Wptt (kN)**
1	3,6	0,000	0,000	671	-0,7	2
5	18,0	0,825	1,217	671	-7,9	26
10	36,0	0,990	1,481	671	-20,8	69
14	50,4	1,083	1,628	671	-31,0	103
15*	54,0	1,103	1,661	671	-33,4	110
17	61,2	1,140	1,720	671	-37,7	125
20	72,0	1,191	1,801	590	-43,4	126

Story	z Elevation (m)	k(ze)	Wtt (kN/m²)	Mj (Ton)	yji (mm)	Wptt (kN)
Total						**1.431 kN**

(*) Floor 15 (elevation 54.0 m ≈ 75% of building height): the location of the inflection pointis determined from the analysis of the moment diagram in the wall—from this elevation upwards, the rigid wall contributes negligibly to controlling horizontal displacement.

4.4.Comparison of overall horizontal displacement at the top of the structure.

The results of the ETABS 2022 analysis, considering all load combinations (dead load, live load, wind, earthquake), show:Case 2 (optimal frame cross-section, full-height walls): Top displacement y(H) = 38.6 mm, a reduction of 18.9% compared to Case 1 (47.6 mm). This is purely the result of redistributing the frame cross-section while the total concrete volume remains the same, confirming the effectiveness of the strategy of maximizing αH. Case 3 (optimal cross-section of Case 2 + wall reduction from the 15th floor upwards): Top displacement y(H) = 39.3 mm, equivalent to Case 2 (deviation of only 0.7 mm = 1.8%), while the structural mass is reduced by 170 tons (2.85% of the total mass). This is direct quantitative evidence confirming that the shear walls from the 15th floor upwards no longer play a significant role in controlling displacement. Scenario 4 (Scenario 3 + increased beam cross-section of 350×900 mm² at floors 14–20): Peak displacement y(H) = 38.0 mm, a further reduction of approximately 1.3 mm compared to Scenario 3, but this increases the mass back to almost the same as Scenario 1 and significantly increases construction costs. This option does not provide sufficient benefits to offset the complexity.

In particular, all four cases satisfy the displacement limit according to TCVN 5574:2018: $f/H \leq 1/750$, equivalent to $f \leq 96$ mm with H = 72 m. The maximum displacement (Scenario 1 = 47.6 mm) is only 49.6% of the allowable limit, confirming structural safety in all options.

For a comprehensive comparison, Table 4 below summarizes the actual peak displacement, the reduction compared to the preliminary design, and the utilization rate of the displacement limit according to TCVN 5574:2018 for all four survey cases.

Table 4. Verification of top horizontal displacement conditions according to TCVN 5574:2018 and comparison of displacement reduction effectiveness.

Criteria	**TH1 (Preliminary)**	**TH2 (Optimize cross-section)**	**TH3 (T15+ septal cutting)**	**TH4 (Overall)**
Actual peak displacement (mm)	**47,6**	**38,6**	**39,3**	**38,0**
Limit f ≤ H/750 (mm)	96	96	96	96
Limit Utilization Ratio. (%)	49,6	40,2	40,9	39,6
Reduction compared to TH1	—	**↓ 18,9%**	**↓ 17,4%**	**↓ 20,2%**
Assessment of compliance with TCVN 5574:2018	✓ Passed	✓ Passed	✓ Passed	✓ Passed

Tỉ lệ tận dụng giới hạn = (chuyển vị thực tế / giới hạn cho phép) × 100%. Option TH3 achieved optimal results in the following combinations: a 17.4% reduction in displacement, a 2.85% saving in volume, and full compliance with TCVN 5574:2018.

Table 5. Combined efficiency index of the four design options (considering wind load, mass, and displacement simultaneously)

Performance index	TH1	TH2	TH3	TH4
Total wind load F (kN)	1.375	**1.236 (↓10,1%)**	1.360 (↑1,1%)	1.360 (↑1,1%)
Bottom shear force Fb (kN)	2.550	2.680 (↑5,1%)	2.608 (↑2,3%)	2.670 (↑4,7%)
Volume savings (tons)	—	—	**170 t (2,85%)**	5 t (0,07%)

Performance index	TH1	TH2	TH3	TH4
Peak displacement reduction (%)	—	↓ 18,9%	↓ 17,4%	↓ 20,2%
Practical priority ranking	**Reference**	**Optimize wind reduction**	**✓ Economic and technical optimization**	**Optimize displacement**

The TH2 base shear force increases because the stiffer structure absorbs more seismic forces—consistent with the seismic resistance law (a stiffer structure absorbs more force but has smaller displacement). TH3 is marked as economically and technically optimal because it achieves the best balance between reduced displacement, weight savings, and seismic force control.

V. CONCLUSION

The study presented a systematic methodology and quantitative analysis results on the influence of stiffness distribution of the frame-wall system on the lateral displacement of reinforced concrete high-rise buildings. The main conclusions are as follows:

- The stiffness ratio parameter αH is the core indicator controlling the lateral displacement of the frame-wall system. The strategy of maximizing αH through redistributing beam-column cross-sections (increasing beam height, reducing column cross-section) with unchanged total concrete volume is an effective method that does not require additional material. Specifically, adjusting the cross-section from (800×800 columns, 300×650 beams) to (750×750 columns, 300×850 beams) reduced the lateral displacement at the apex by 18.9%.
- Reducing the size of the shear walls from the 15th floor (approximately 75% of the building height) upwards is feasible and effective. This location corresponds to the bending inversion point of the moment diagram in the wall—where the wall transitions from an effective bending function to an auxiliary role. Reducing the wall saves 2.85% of the structural mass and only increases the top displacement by 0.7 mm (1.8%)—a completely advantageous economic and technical trade-off.
- Option TH3 (optimized frame cross-section combined with wall reduction from the 15th floor) is the recommended choice in practical design. This option simultaneously achieves: a 17.4% reduction in lateral displacement, a 2.85% saving in structural mass, and simplified construction compared to TH4.
- Optimizing frame stiffness has a dual effect: reducing dynamic wind load due to a shorter oscillation period (T_1 reduced from 2.10 s to 1.90 s, wind load reduced by 10.1%); while simultaneously slightly increasing the earthquake base shear force due to increased stiffness. A comprehensive assessment of both impacts is necessary during the design process.
- The proposed method has broad applicability to reinforced concrete high-rise buildings of 15–30 stories in Vietnam, especially those located in areas subject to both high wind loads and earthquakes. Further research should consider: the influence of shear wall position on the plan on the torsional effect; the effectiveness of outrigger systems in reducing lateral displacement for 30–50 story buildings; and multi-objective optimization methods (displacement–mass–cost) using modern optimization algorithms.

Conflict of interest

There is no conflict to disclose.

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