Postprint: Design Study of Hollow Square Buildings under Seismic Action

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Abstract

The main building of the project features plan dimensions of 99m$\times$79m and a height of 60m, constituting a hollow rectangular configuration composed of four towers with varying stiffness. In response to the project' s characteristics and code-exceeding conditions, a performance-based seismic design methodology was implemented. Under frequent earthquake actions, both the modal decomposition response spectrum method and elastic time-history analysis method were utilized for global structural calculation analysis and stability assessment. Corresponding detailing measures were adopted for the weak and critical elements identified in the aforementioned analyses to ensure the fulfillment of seismic performance objectives. The results indicate that the structural system is safe and feasible.

Full Text

Preamble

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Research on the Seismic Design of Back-Shaped Buildings

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Abstract: The main building of this project features a plan dimension of 99m $\times$ 79m and a height of 60m, comprising a back-shaped structure formed by four towers with different stiffness characteristics. Addressing the project' s unique features and code-exceeding conditions, a performance-based seismic design approach was adopted. Under frequent earthquake actions, modal decomposition response spectrum analysis and elastic time-history analysis were employed for global structural calculation and stability analysis. Corresponding detailing measures were implemented for weak and critical locations identified in the analysis to ensure achievement of the seismic performance objectives. The results demonstrate that the structural system is safe and feasible.

Keywords: super-long structure; complex engineering; time-history analysis; large podium; multi-tower building
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1 Project Overview

The Daming Industrial Park project is located at the intersection of Huanghaixi Sixth Road and Dongbei Avenue in Jinzhou New District, Dalian. It serves as an integrated office, research and development, and apartment dormitory facility for the new district' s industrial park. The back-shaped main building functions as the park' s landmark structure, with a total floor area of approximately 70,000 m^2, including above-ground construction of about 60,000 m^2 and underground construction of about 10,000 m^2. The underground level partially accommodates a Class A air defense basement (nuclear grade 6, conventional grade 6) and partially houses supporting ancillary spaces. The ground floor serves as an entrance lobby and office hall with a story height of 6m, floors 2–5 function as R&D spaces with a story height of 5.4m, and floors 6 and above are apartment dormitories also with a 5.4m story height. The building totals eleven stories with an overall height of 60m, featuring a back-shaped plan configuration [Figure 1: see original paper] measuring 99m × 79m, with an interior courtyard dimension of 51m × 47m.

2 Structural System

The project site is classified as seismic intensity 7, with the first design earthquake group and a basic design ground acceleration of 0.159g. The maximum horizontal seismic influence coefficient is 0.12 under frequent earthquake action. The site category is Class II (characteristic site period of 0.35s) with a structural damping ratio of 0.05. The overall back-shaped configuration constitutes a plan-irregular building, while the structural plan dimensions exceed the maximum expansion joint spacing stipulated in Article 8.1.1 of the *Code for Design of Concrete Structures* GB50010-2010. The column grid measures approximately 5.7m × 8.1m, with limited shear walls placed locally.

Due to functional requirements, the east and west building sections (Towers 2 and 4 in [Figure 2: see original paper]) feature large open spaces such as entrance lobbies on the ground floor, and the basement contains mechanical parking spaces that restrict shear wall placement, necessitating a frame structure. The vertical height exceeds the A-grade frame maximum applicable height in Article 3.3.1 of the *Technical Specification for Concrete Structures of Tall Buildings*

JGJ3-2010 by more than 20%. The north and south building sections (Towers 1 and 3 in the figure) fully utilize vertical circulation areas to arrange shear walls, enabling the adoption of a frame-shear wall structure.

To ensure structural safety and reliability, a comprehensive analysis of various seismic scenarios was conducted for this project. Two structural schemes were developed and compared: Scheme 1 incorporates four expansion joints, dividing the main structure into four independent rectangular towers [Figure 2: see original paper] for separate analysis, while Scheme 2 eliminates joints and designs the building as an integrated back-shaped structure [Figure 3: see original paper], performing weak spot calculations and implementing corresponding strengthening measures. The primary calculation results for both schemes are presented in Tables 1–3.

The high-end, complex functional requirements created numerous structural design challenges. After extensive scheme comparisons, the final design adopted Scheme 2, maintaining the superstructure as a single integrated tower. Although this creates a unified structure, the four sections exhibit significantly different stiffness. By adjusting the quantity of shear walls, the dynamic characteristics in both principal directions were balanced, while weak locations were strengthened to facilitate achievement of the seismic fortification objectives. The main structure has a calculated height of 60m and was determined to be a frame-shear wall system with reinforced concrete primary beams and large-slab floor systems. Frame column sections range from 900mm to 600mm, core tube and shear wall thicknesses from 400mm to 200mm, and concrete grades from C50 to C30. To control overall torsion and lateral displacement, reinforced haunched beams and slabs were installed at the interior courtyard re-entrant corners, with a total longitudinal reinforcement ratio of not less than 1% for the full cross-section of these haunched members.

3 Structural Design and Calculation

Scheme 1 presents several critical issues, particularly for Towers 2 and 4. Due to functional constraints on Y-direction shear wall placement, walls are only arranged in the X-direction, severely violating Article 7.1.1 of the *Technical Specification for Concrete Structures of Tall Buildings* (JGJ3-2010), which states that "for seismic design, structural arrangements with walls in only one direction shall not be adopted." This configuration results in drastically different dynamic characteristics in the two directions and poor spatial structural performance. The structural system can only be classified as a frame structure, which is inherently disadvantageous and significantly exceeds the 50m maximum applicable height for A-grade frames specified in Article 3.3.1 of the same code. Such structures require specialized research and effective strengthening measures such as steel-reinforced concrete members or concrete-filled steel tube members, along with special expert review per Ministry of Construction requirements. In practice, Scheme 1 represents an unadoptable solution, though it provides useful baseline data on the safety reliability and column base shear response of independent pure-frame Towers 2 and 4 for comparative analysis.

Structural calculations for Scheme 1 reveal excessively long periods and insufficient base shear, resulting in an overly flexible overall structure, thereby validating the rationality of the code' s maximum applicable height provisions for frame structures. Based on code requirements and regional experience, pure frame structures exceeding 30m in seismic intensity 7 zones with 0.159g peak ground acceleration exhibit weak overall stiffness and are neither economical nor practical.

Scheme 2, the actual implemented solution, eliminates the four expansion joints, allowing the relatively flexible Towers 2 and 4 to rely on the stiffer Towers 1 and 3, forming a large integrated tower with a frame-shear wall structure. This approach avoids special review for over-height frames and eliminates structural system irrationality. The overall calculation results demonstrate that all ratios —including stiffness ratio, displacement ratio, and shear capacity ratio—satisfy code requirements. Compared with Scheme 1, the total mass remains essentially unchanged, while the X-direction base shear in Scheme 2 exceeds the sum of base shears in Scheme 1. The Y-direction base shear is comparable to Scheme 1' s sum because the overall calculation involves reduced Y-direction shear walls in Scheme 2.

Further analysis of Scheme 2' s modal contributions reveals that the first and third modes are predominantly translational, while the second mode exhibits coupled translational-torsional vibration occurring earlier than typical. Given the building' s large plan dimensions, relatively short height, and presence of inter-story beams, early appearance of torsional modes is difficult to avoid. The period ratio of 0.89 satisfies Article 3.4.5 of the *Technical Specification for Concrete Structures of Tall Buildings* (not exceeding 0.9). The higher-mode seismic contributions are relatively small.

4 Elastic Dynamic Time-History Analysis

Due to the building' s plan irregularity and relatively tall story heights, elastic dynamic time-history analysis under frequent earthquakes was performed as a supplementary calculation to the response spectrum analysis. The primary objectives were to compare base shear, story shear forces, and inter-story drift angles with response spectrum results, and to adjust member internal forces and reinforcement accordingly. Based on actual site conditions and considering spectral characteristics, effective peak values, and duration, three sets of acceleration time-history curves were selected from the analysis software' s database: two natural waves (TH1TG040, TH2TG040) and one artificial wave (RH3TG040). The supplementary calculations indicate that maximum inter-story drift angles and maximum story displacements all satisfy code requirements [Figure 4: see original paper]. The average base shear in the principal structural direction from time-history analysis is not less than 80% of that from modal decomposition response spectrum analysis, with individual wave results all exceeding

65% and not exceeding 135%, and average results from each wave not exceeding 120% [Figure 5: see original paper]. Comparison of the average seismic influence coefficients from the three time-history waves with the response spectrum curve shows differences of less than 20% at the structural primary vibration period points [Figure 6: see original paper], meeting minimum safety requirements for time-history analysis.

Comparison between elastic time-history analysis and modal decomposition response spectrum results shows that under natural wave TH1TG040, Y-direction maximum story shear forces at floors 3–8 are approximately 3%–11% greater than response spectrum values; under natural wave TH2TG040, forces at floors 9–12 are about 25%–33% greater. X-direction maximum story shear forces under TH2TG040 at floors 9–12 exceed response spectrum values by approximately 20%–30%. The increased shear forces at upper floors reflect the strong higher-mode response characteristic of tall-flexible structures, indicating pronounced whipping effects. Structural design adopts envelope values from these calculations, amplifying internal forces based on response spectrum results for supplementary member calculations.

The project has passed special expert review and entered the construction drawing design phase. Two schemes were comparatively analyzed based on actual conditions. Under frequent earthquake actions, both modal decomposition response spectrum analysis and elastic time-history analysis were employed for global structural calculation, with envelope design adopted to ensure seismic performance objectives. The calculation results demonstrate that the structural system is safe and feasible. Due to space limitations, strengthening measures for the super-long structure and minimally-walled frame system will be addressed in subsequent publications.

With scientific and technological advancement, building heights and plan dimensions continue to increase—a natural progression. The "maximum applicable height" specified in codes and specifications does not represent an absolute limit but rather a comprehensive consideration of seismic performance, economy, rational applicability, and damage experience for different structural systems. While Scheme 1 with expansion joints could theoretically design Towers 2 and 4 as 60m frame structures, such an approach would be clearly unreasonable and defective. Unreasonable schemes inevitably produce unreasonable results: the expansion joints would cause Towers 1 and 3 to become frame-shear wall structures with eccentric core tubes, inducing tensile stresses in some frame columns that are detrimental to the structure and create unnecessary foundation burdens, requiring additional sufficiently rigid shear walls for adjustment and improvement. Meanwhile, current software programs still have incomplete analysis capabilities for multi-tower structures. From another perspective, viewing the back-shaped configuration as a large square minus a small square (seamless design) simplifies the problem to identifying adverse factors.

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Note: Figure translations are in progress. See original paper for figures.