Postprint: Design Study of Courtyard Buildings Under Seismic Action

Authors: Zhang Xiuhua; Zhang Guohui

Date: 2017-11-06T00:00:00+00:00

Abstract

The main building of the project has plan dimensions of 99m×79m and a height of 60m, forming a hollow rectangular configuration composed of four towers with different stiffnesses. Based on the project' s characteristics and irregularity conditions, a performance-based seismic design approach was adopted. Under frequent earthquake actions, modal decomposition response spectrum method and elastic time-history analysis method were employed for global structural calculation analysis and stability analysis. Corresponding detailing measures were implemented for the weak and critical locations identified in the aforementioned analysis results to ensure the achievement of seismic performance objectives. The results demonstrate that the structural system is safe and feasible.

Full Text

Preamble

Volume 8, Issue 1, February 2016

Research on Seismic Design of Back-Shaped Buildings

Zhang Xiuhua, Zhang Guohui
(Dalian Economic and Technical Development Zone Planning and Architectural Design Institute, Dalian 116000, China)

Abstract: This project involves a main building with plan dimensions of 99m × 79m and a height of 60m, featuring a back-shaped configuration composed of four towers with different stiffness values. Addressing the project' s characteristics 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 results to ensure the achievement of seismic performance objectives. The results demonstrate that this structural system is safe and feasible.

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

Chinese Library Classification: TU318
Document Code: A
Article ID: 1674-7461(2016)01-0083-04
DOI: 10.16670/j.cnki.cn11-5823/tu.2016.01.15

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 with nuclear grade 6 and conventional grade 6 protection, with the remaining area designated for supporting ancillary facilities. The ground floor functions as an entrance lobby and office hall with a story height of 6m. Floors 2 through 5 serve as research and development spaces with a story height of 5.4m. Floors 6 and above are apartment dormitories, also with a 5.4m story height. The building comprises eleven stories with a total height of 60m. The plan configuration is back-shaped [Figure 1: see original paper] with overall dimensions of 99m $\times$ 79m and an inner courtyard measuring 51m $\times$ 47m.

2. Structural System

The project site is located in a seismic fortification intensity zone of 7 degrees, belonging to the first design earthquake group, with a design basic earthquake acceleration value of 0.159g and a maximum horizontal earthquake influence coefficient of 0.12 under frequent earthquake actions. The site classification is Category II (site characteristic period of 0.35s) with a structural damping ratio of 0.05. The overall back-shaped configuration constitutes an irregular plan shape, while the structural plan dimensions exceed the maximum spacing requirements for expansion joints specified in Article 8.1.1 of the *Code for Design of Concrete Structures* GB50010-2010. The column grid is approximately 5.7m $\times$ 8.1m with limited shear walls.

Due to functional requirements, the east and west building sections (Towers 2 and 4 in [Figure 2: see original paper]) contain large open spaces such as entrance lobbies on the ground floor, while the basement accommodates mechanical parking spaces, severely restricting shear wall placement and necessitating a frame structure. Vertically, the height exceeds the A-grade frame maximum applicable height specified 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 [Figure 2: see original paper]) fully utilize vertical circulation areas to arrange shear walls, enabling the adoption of a frame-shear wall structure.

To ensure structural safety and reliability under various seismic conditions, specialized analyses were conducted for structural scheme selection. **Scheme 1** involves installing four expansion joints to divide the main body into four independent rectangular structural towers [Figure 2: see original paper] for separate analysis using independent structural systems. **Scheme 2** adopts a seamless back-shaped configuration [Figure 3: see original paper] with global analysis of weak elements and corresponding strengthening measures. Primary calculation results for both schemes are presented in , , and .

The high-end and complex functional requirements create numerous structural design challenges. After extensive scheme comparisons, the final design eliminates expansion joints in the superstructure, creating a single integrated large tower [Figure 3: see original paper]. Although this creates a unified structure, the four sections exhibit significantly different stiffness values. By adjusting shear wall quantities, the dynamic characteristics in both principal directions were balanced, while weak locations were strengthened to facilitate achievement of seismic fortification objectives.

The main structure has a calculated height of 60m and was determined to be a frame-shear wall structure with reinforced concrete primary beams and large-slab floor systems. Frame column sections range from 900mm to 600mm, while core tube and shear wall thicknesses vary from 400mm to 200mm. Concrete grades range from C50 to C30. To control overall torsion and lateral displacement, reinforced haunched beams and slabs were installed at the re-entrant corners of the inner courtyard, with a total longitudinal reinforcement ratio of not less than 1% for the full cross-section of each haunched beam.

3. Structural Design and Calculation

Scheme 1 presents several critical issues, primarily concentrated in Towers 2 and 4. Due to restrictions on shear wall placement in the Y-direction, walls are only arranged in the X-direction, severely violating the requirement in Article 7.1.1 of the *Technical Specification for Concrete Structures of Tall Buildings* that "structures should not employ a layout with walls in only one direction during seismic design." This configuration results in drastically different dynamic characteristics in the two structural directions and poor spatial performance, forcing the adoption of a pure frame structure. This is inherently disadvantageous and substantially exceeds the 50m A-grade frame maximum applicable height specified in Article 3.3.1 of the same specification. Such structures require specialized research and effective strengthening measures such as steel-reinforced concrete members or concrete-filled steel tube members, along with mandatory special review according to Ministry of Construction requirements. In practice, Scheme 1 is not a viable structural solution, but it provides valuable comparative data regarding the inherent safety reliability and column base shear response of independent pure-frame Towers 2 and 4.

Computational data for this project indicates that the period is excessively large and the base shear is too small, resulting in an overly flexible overall structure. This validates the rationality of the code provisions regarding maximum applicable heights for frame structures. According to code requirements and regional experience, in 7-degree seismic fortification zones with a peak ground acceleration of 0.159g, pure frame structures exceeding 30m exhibit weak overall stiffness and are neither economical nor practical.

Scheme 2, the actual project solution, was therefore emphasized in the analysis. By eliminating the four expansion joints, the relatively flexible Towers 2 and 4 are integrated with the stiffer Towers 1 and 3 to form a large unified tower with a frame-shear wall structure. This approach avoids special review for ultra-high frames and eliminates structural system irrationalities. Global 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 shows minimal difference. However, the X-direction base shear in Scheme 2 exceeds the sum of base shears in Scheme 1, while the Y-direction base shear is comparable due to reduced Y-direction shear walls in the global analysis.

Further analysis of Scheme 2 examined the contribution of various vibration modes to seismic action. Observations of structural vibration under different modes reveal that the first and third modes are predominantly translational, while the second mode exhibits coupled translational-torsional vibration. Contributions from higher modes are relatively minor. Given the building' s large plan dimensions, relatively short height, and presence of inter-story beams, the early appearance of torsional components in the second mode 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* requiring a value not exceeding 0.9.

4. Elastic Dynamic Time History Analysis

Due to the irregular plan configuration and relatively tall story heights, supplementary elastic dynamic time history analysis under frequent earthquake actions was performed in addition to response spectrum calculations. The primary objectives were to compare base shear, story shear, and inter-story drift angle 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 and TH2TG040) and one artificial wave (RH3TG). The analysis results indicate that the maximum inter-story drift angles and maximum story displacements from the supplementary calculations all meet code requirements [Figure 4: see original paper]. The average base shear in the principal structural direction is not less than 80% of that obtained from modal decomposition response spectrum analysis, with individual wave results all exceeding 65% and no single wave result exceeding 135% of the response spectrum value, while the average of all waves does not exceed 120% [Figure 5: see original paper]. Comparison of the average earthquake influence coefficient from the three time-history waves with the spectrum curve used in response spectrum analysis shows differences of less than 20% at the structural principal vibration period points [Figure 6: see original paper], satisfying minimum safety requirements for time-history analysis.

Comparison between elastic time-history analysis results and modal decomposition response spectrum results reveals that under natural wave TH1TG040, the Y-direction maximum story shear at floors 3–8 is approximately 3%–11% larger than that under response spectrum analysis. Under natural wave TH2TG040, the story shear at floors 9–12 is about 25%–33% larger. In the X-direction, natural wave TH2TG040 produces story shear at floors 9–12 that is approximately 20%–30% greater than response spectrum results. The increased shear at upper floors reflects the strong higher-mode response of this relatively flexible structure, indicating a pronounced whipping effect. The design adopts envelope values from these calculations, amplifying internal forces based on response spectrum results for supplementary member design.

Conclusion

The project has passed special review and is currently in the construction drawing design phase. Two schemes were comparatively evaluated 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. Calculation results demonstrate that the structural system is safe and feasible. Due to space limitations, detailed discussions on strengthening measures for the super-long structure and minimally-walled frame system will be presented in subsequent publications.

With scientific and technological advancement, building heights and plan dimensions continue to increase—this is a natural progression. The "maximum applicable height" specified in codes and specifications is not a strict limit but rather a recommended maximum height considering comprehensive factors including seismic performance, economy, rationality, and damage experience. While Scheme 1 with expansion joints could theoretically design Towers 2 and 4 as 60m frame structures, this would be clearly unreasonable and defective. Unreasonable schemes inevitably produce unreasonable results: the separation would cause

Towers 1 and 3 to become frame-shear wall structures with eccentric cores, inducing tensile stresses in some frame columns that are detrimental to the structure and create unnecessary foundation burdens, requiring additional sufficiently rigid shear walls to adjust and improve structural performance.

Furthermore, current software programs still have limitations in analyzing multi-tower structures. However, viewing this project from another perspective—considering the back-shaped configuration as a large square minus a small square (seamless design)—allows focus on identifying adverse factors rather than dealing with incomplete analysis functions.

References

[1] GB50010-2010, Code for Design of Concrete Structures [S]. Beijing: China Architecture & Building Press, 2010.
[2] GB50010-2010, Code for Seismic Design of Buildings [S]. Beijing: China Architecture & Building Press, 2010.
[3] JGJ3-2010, Technical Specification for Concrete Structures of Tall Buildings [S]. Beijing: China Architecture & Building Press, 2010.
[4] PMCAD User Manual and Technical Conditions. China Academy of Building Research, PKPMCAD Engineering Department.
[5] SATWE User Manual and Technical Conditions. China Academy of Building Research, PKPMCAD Engineering Department.

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