Structural Design and Analysis of Zhaogangwang Steel E-commerce Technology Center

Ran Zhao¹, Kai Wei Wang⁺

¹ SHEN KAN ENGINEERING & TECHNOLOGY CORPORATION, MCC, Shenyang, Liaoning, 110167, China
⁺Corresponding author’s e-mail: wkw95@stu.sjzu.edu.cn

Abstract. This project is a complex over-limit multi-storey structure. The structural over-limit items were analyzed, key parts were adopted to improve the seismic grade, the frame column was made of reinforced concrete column, and the thickness of the floor slab and the reinforcement were designed. The conversion truss was located on the 4th floor, with a span of 25.2m, and adopted H-shaped steel member steel truss. The performance-based design of the structure only meets the requirements of the supporting column conversion truss and related vertical members, and meets the non-yielding of large earthquakes.

1. Introduction
The project is located on the northwest side of Jiading, about 10 kilometers away from Jiading New District and 20 kilometers away from the center of Shanghai. The site covers an area of 11356.8 square meters, and the terrain is flat. The northeast side of the site is adjacent to the urban road Xinpei Road, the northwest side is the urban river, the southeast and southwest side is now farmland, and the north side is a substation.

The land used for the project is R&D and design land (land used for R&D headquarters) and consists of three buildings. The main project is a five-story reinforced concrete structure research and development room (building area is 38938.11m²), with a total height of five floors above ground, three floors in part, building height 23.99m and have two floors underground, staff canteen and gymnasium and garage supporting room are in the underground floors, the storey heights are 5.55m and 3.9m respectively; the first storey is 5.15m high, and the second to fifth storeys are 4.4m high.

The entrance of the main building is cantilevered on three floors with a span of 25.2 meters, the first to third floors are in a "U" shape, the fourth to fifth floors are in a "back" shape, and the center is an open central courtyard.

The entrance of the main building is cantilevered on three floors with a span of 25.2 meters, the first to third floors are in a "U" shape, the fourth to fifth floors are in a "back" shape, and the center is an open central courtyard.
Figure 1 Architectural effect drawing

Figure 2 Floor plan of the building
The safety level of this project is Class II, the basic seismic fortification intensity is 7 degrees, the designed basic seismic acceleration value is 0.1g, and the designed seismic group is divided into the second group. The building site category is IV, and the characteristic period of the design response spectrum is 0.90s. The design service life is 50 years, the local basic wind pressure is 0.55 KN/m², and the ground roughness is Class B.

2. Structural system and structural layout

2.1 Structural system
It can be seen from Figure 2 that the above-ground building is in an irregular "U" shape, with a bidirectional average size of 94mx71m. The plane shape is complicated, and the above-ground part is composed of two parts, 1~9 axis and 10~13 axis, with seismic joints and expansion joints. The seam divides the upper structure into two structural units.

According to the structural height of the building, the basic fortification intensity, the plane layout and the function of the ground and underground 1# The above ground part of the building 1~9axis adopts a frame-shear wall structure, and the stair elevator is used to set up a shear wall tube to increase the structural rigidity, 10~13 axis adopts frame structure. The foundation adopts piled raft foundation, the thickness of the raft board is 600mm, the pile adopts prestressed concrete pipe pile, the pile model is PHC500AB, the pile length is 34m, and it can withstand pressure and pull force.

The seismic grade of the frame and the shear wall of the above-ground part of the project are both Grade 3. The seismic grade of the supporting column steel truss is Grade 3; the seismic grade of other steel structural members is Grade 4; the seismic grade of the frame column connected with the transfer truss is Grade 2.

2.2 Structural layout
The typical column mesh size of the 1~9axis part is 8.4mx8.4m, the outer frame beam is 500mmx800mm, and the frame beam typical section size is 400mmx700mm; a secondary beam is arranged in the 8.4m span, and the secondary beam typical section is 250mmx600mm. The upper and lower chords of the connecting body truss set the reinforced truss floor deck, the floor slab thickness is 150mm, and the adjacent floor slab is connected and the steel beam is connected to transmit seismic force. The large-span part of the 10~13 axis adopts prestressed well-shaped beams, and the beam section is 500mmx1500mm.

2.3 Floor system
The floor of this project adopts a prefabricated structural floor system, and the following measures are taken for the floor 1. When opening a large hole in the floor or a part of the missing part of the floor, increase the thickness of the floor slab around the opening. The thickness of the entire floor of the 1~9 axis part is not less than 130mm, and the double-layer two-way general-length reinforcement is adopted. The directional reinforcement ratio is not less than 0.25%;

2) The thickness of the upper and lower chord of the connecting body truss and its adjacent inner span is 150mm, with double-layer bidirectional reinforcement, and the reinforcement ratio is not less than 0.25%. The large-span part of the 10~13 axis adopts the prestressed well-shaped beam cast-in-place concrete beam slab system.

3. Design of the embedded end of the basement roof
The embedded end of the superstructure of this project is on the ceiling of the basement. The basement roof has no large openings. The cast-in-place reinforced concrete floor slab system is adopted in the design. The floor slab thickness is 180mm, double-layer two-way reinforcement, and the reinforcement ratio is not less than 0.25%.

1) The scope is partly expanded from the first floor above the ground. There is a structural height difference of 1.55m between the expanded basement roof and the main first floor slab, which is greater
than the beam height of the first floor beams. In order to meet the conditions for the upper structure to be embedded in the ceiling of the basement, vertical haunches are set at the height difference of the basement beams and slabs, and the seismic resistance of adjacent frame columns is increased by one level.

2) Add shear walls to appropriate parts of the basement to ensure that the equivalent shear stiffness of the first underground floor is twice the equivalent shear stiffness of the first floor above the ground.

4. Out-of-limit structure

According to the Code for Seismic Design of Buildings (GB50011-2010)[1], there are 4 general irregularities in the Shanghai Super-Limited High-rise Buildings Seismic Fortification Management Implementation Rules, including torsion irregularities and partial discontinuities of floor slabs, which belong to special irregular buildings:

1) Support Column conversion: The 4th floor is equipped with a supporting column conversion steel truss to form a discontinuity of the anti-side force member;
2) The torsional displacement ratio considering the accidental eccentricity is 1.39, which is greater than 1.2;
3) The size of the deep notch in the floor plan of the 2nd and 3rd floors Greater than 30% of the corresponding side length (the protruding length is greater than 50% of the corresponding total size, and the protruding width is less than 50% of the protruding length);
4) The floor is incomplete, and the effective width of the floor of more than 4 floors is less than 50%.

5. Overall calculation and analysis of structure

5.1 Calculation and analysis under small earthquakes

Part of 1~9 axis structure calculation adopts SATWE and PMSAP software for analysis and comparison. The main calculation results are shown in Table 1[3].

| Calculation Software | SATWE  | PMSAP  |
|----------------------|--------|--------|
| Natural vibration period /s | T₁     | 0.7079 | 0.675  |
|                       | T₂     | 0.6397 | 0.618  |
|                       | T₃     | 0.5801 | 0.521  |
| Cycle ratio           |        | 0.82   | 0.77   |
| Effective mass factor | X      | 99.72% | 93.79% |
|                       | Y      | 99.66% | 93.23% |
| Shear force of basement under earthquake /kN | X      | 22886  | 22545  |
|                       | Y      | 25196  | 25172  |
Minimum shear weight ratio

|       | X direction | Y direction |
|-------|-------------|-------------|
| X     | 6.04%       | 5.52%       |
| Y     | 6.84%       | 6.77%       |

Stiffness-to-weight ratio

|       | X direction | Y direction |
|-------|-------------|-------------|
| X     | 19.25       | 17.11       |
| Y     | 14.98       | 16.78       |

Maximum displacement angle between floors under earthquake action (floor)

|       | X direction | Y direction |
|-------|-------------|-------------|
| X     | 1/1696(floor 4) | 1/1708(floor 4) |
| Y     | 1/1224(floors 5) | 1/1269(floors 5) |

Maximum displacement ratio (floor)

|       | X direction | Y direction |
|-------|-------------|-------------|
| X     | 1.40(floor 5) | 1.39(floor 5) |
| Y     | 1.35(floor 6) | 1.37(floor 6) |

It can be seen from Table 1 that the calculation results of the two software are relatively close, all indicators meet the requirements of the specification, and the structural system has good lateral stiffness.

5.2 Elastic time history analysis under the action of small earthquakes

Two natural waves TH1TG090, Tg(0.90) and Landers_NO_850, Tg(0.90) were selected, and one natural wave ArtWave-RH4TG090, Tg(0.90) was used for elastic time history analysis. In order to effectively compare the results of the elastic time history analysis with the results of the CQC method, the input peak value of the main direction of the acceleration time history is 35.0cm/s², and the input peak value of the secondary direction is 29.75cm/s². The calculation results show that the bottom shear force of the structure calculated by each time history curve is greater than 65% of the calculation result of the mode decomposition response spectrum method [3], and the average value of the structure bottom shear force calculated by the three time history curves is greater than the mode decomposition response spectrum method 80% of the calculation result. The calculation result takes the larger value of the envelope value of the time history method and the mode decomposition response spectrum method.

5.3 Calculation of non-yielding of large earthquakes

This project is a multi-storey building. Although there are four irregularities, it is not an over-limit high-rise building [4]. Therefore, the performance-based design of the structure is only for the requirements of the supporting column conversion truss and related vertical components, and meets the requirements of large earthquakes without yielding. The calculation results show that both the converted steel truss and its connected steel-concrete frame columns meet the non-yielding requirements of large earthquakes.

6. Conversion steel truss design

Due to the functional requirements of the building plan and facade, the entrance of this project has three floors with a span of 25.2m. The 4th floor (altitude 13.850m) to the 5th floor (18.250m) are connected as a whole, the connection part adopts steel structure truss, a total of 3 beams, rigidly connected with the main steel concrete frame columns on both sides, and the upper and lower chords of the connection
The steel grade of the truss members is Q345.

7. Conclusion
All indexes of elastic calculation and analysis meet the requirements of the specification, which the rationality of the structural system was verified. This project is a multi-layer over-limit structure. In view of the over-limit content of the structure, design measures have been taken to improve the seismic grade and strengthened the thickness of the floor and the reinforcement; Reinforcement measures are taken for steel reinforced concrete frame columns. The results of stress analysis show that the truss and its related vertical steel reinforced concrete frame columns meet the stress requirements for non-yielding in a large earthquake.

The design of this project is unique and has strong research value and reference significance. The research on the seismic direction of high-rise structures is always the top priority. The reciprocating wind load at high places will cause fatigue damage to the steel, which is extremely unfavorable for earthquake resistance. This paper meets the stress requirements of high-rise structures by improving the design method, which has guiding significance for the design of similar projects.

References
[1] GB 50011-2010 Code for seismic design of buildings (2016)[S]. Beijing: China Construction Industry Press, 2016.
[2] Du W B, Sun J C, Wang Y, Zhou J X, Yang X M, Pan N. Over-limit design of the Great Wall Financial Building with conversion truss structure [J]. Building Structure, 2016, 46(13): 13-18.

[3] Du M J, Xu P, Hu Z J, Wang P. The structural design of a high-rise large-span conjoined over-limit high-rise building [J]. Building Structure, 2019, 49(05): 83-89.

[4] Hou Bo. Numerical simulation of fluctuating wind load and wind-induced vibration response analysis of mega-frame damping structure[D]. Hefei University of Technology, 2012.

[5] Zhang Y T. Research on Intensity Index and Collapse Safety Reserve of Super High-rise Building Structure[D]. Dalian University of Technology, 2019.

[6] Li Y M, Liu S Y. Research on time lag effect of seismic displacement and shear response peak of super high-rise structure[J]. Journal of Building Structures, 2017, 38(07): 93-102.