Discussion on the optimum configuration of section steel for the middle seismic tensile stress of the shear wall of an over-limit high-rise civil building

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Abstract: A super high-rise civil building was originally designed with a relatively large area of shear wall section steel configuration, coupled with the configuration of steel bars, resulting in the construction of the project more difficult. To solve this problem, PKPM was used to build the model to extract and calculate the tensile stress of inflexible wall limbs under moderate earthquakes, and the configuration of the section steel in the original design was analyzed. An in-depth interpretation of the “Technical Points for Special Examination of Seismic Fortification of High-rise Buildings Exceeding Limits” (Jianzhi [2015] No. 67), Chapter 4, Article 12, Paragraph 4, on the provision of shear wall section steel configuration was conducted. Finally, based on the original design, an optimization criterion for the configuration of the section steel in the wall was proposed.

1. Introduction

When the seismic fortification intensity is high, it is very common that the tensile stress of the bottom shear wall members of the over-limit high-rise building exceeds 2Ftk under the action of middle earthquakes. Technical Points for Special Examination of Seismic Fortification of High-rise Buildings Exceeding Limits [1] (Jianzhi [2015] No. 67) stipulates that concrete members with small eccentric tension during middle earthquakes shall be used the special first-class structure specified in the Technical Specification for High-rise Concrete Structures. When the average nominal tensile stress generated by the axial force of the full section of the wall under the two-way horizontal earthquake during the moderate earthquake exceeds the standard value of the concrete tensile strength, the section steel should be set to bear the full tensile force. The average nominal tensile stress should not exceed 2 times the standard value of concrete tensile strength (Ftk) (the effect of steel bars, section steels and steel plates can be converted according to the modulus of elasticity), when the steel content of full-section steel and steel plate exceeds 2.5%, the average nominal tensile stress can be appropriately relaxed.

Jiao Ke et al. [2] analyzed the seismic performance of the out-of-limit structure of the core tube and found that the wall limbs with larger tensile stress suffered tensile-shearing or tensile-bending failure under the action of a large earthquake; The fine finite element analysis shows that the tensile and shear resistance of the tensile wall limbs are significantly improved after adding section steel. Dai Yunjing et al. [3] designed an office building with a height of 143.20m. The main structure adopts a reinforced concrete frame-core tube structure system. After reviewing the average nominal tensile stress...
generated by the axial force in the full section of the wall limbs under the action of a moderate earthquake and a two-way horizontal earthquake, the concealed columns of the outer wall of the core tube are equipped with section steel to bear all the tensile forces, improving the seismic ductility of the outer wall of the core tube.

The above-mentioned documents have proved that the tensile stress of non-yielding wall limbs exceeding $F_{tk}$ under moderate earthquakes in high-intensity areas is common, and the seismic performance of wall limbs is improved after the configuration of section steel. The $2F_{tk}$ index comes from the conceptual judgment of the national over-limit review experts. At present, there is a big controversy about the configuration of section steel, and more in-depth experiments and theoretical research are needed. Based on the national "technical points", this paper proposes the optimization criteria for the configuration of wall pier sections.

2. Project overview

The project site is located in Taiyuan City. The building is a reinforced concrete shear wall structure. The structure has 3 floors underground (including an equipment floor) and 48 floors above ground, which are connected with the second underground garage of the community at the top of the basement. The embedded end of the main building is located at ±0.000 floor. The total height of the structure is 153.90m, and they are all high-rise buildings with super-B height. ±0.000m is equivalent to the absolute elevation of 779.950m. The construction area is 50206.35m². The project surpassed the requirements of the Shanxi Province Seismic Fortification Super High-rise Building Project Definition Regulations (Jin Jian Zhi [2016] No. 69) in terms of height, aspect ratio, and irregular torsion. In consideration of structural safety and economy, party A entrusts us to carry out structural review calculations for the structure and optimize the structure to save costs.

3. Structural calculation and analysis

The SATWE model was established for this project. The selected concrete strength grade of the structural members is not less than C30, the steel bars are HPB300, HRB400, and the main stress members are Q345C steel. The overall model of the structure is shown in Figure 1.

The project is located in a Class III site in an area of 8 degrees (0.2 g). The earthquakes are grouped into the second group. The maximum earthquake impact coefficient during moderate earthquakes is $\alpha_{max}=0.45$, and the characteristic period is 0.55s. According to the requirements of the over-limit review, the tensile stress of the shear wall during moderate earthquakes is calculated based on the non-yielding calculation results of moderate earthquakes. According to the non-yielding calculation of moderate earthquakes, the axial force (tension is positive, the pressure is negative) and the pressure under the action of dead load and live load can be obtained by the axial force of each wall limb caused by the earthquake action, and the average axial tension of the wall limb can be obtained. The ratio of the tensile stress to the standard value of concrete tensile strength can also be obtained.

The exterior walls are numbered, and the wall numbering diagram is shown in Figure 2. The SATWE software is used to calculate the moderate earthquake non-yielding calculation of the structure, and the tensile stress of all the numbered wall limbs on different floors in the figure is calculated. The situation that the tensile stress of the wall pier exceeds $F_{k}$ mainly occurs in the short wall pier (when the length of the wall pier is less than 4 times the thickness of the wall, it is a short wall pier). In view of space limitations, Table 1 only lists the tensile stresses of the walls under Q1, Q2, and Q5 without yielding under moderate earthquake. In Table 1, $F_{c}$ represents the standard value of the maximum axial force of the wall under moderate earthquakes, $F_{d}$ represents the standard value of the maximum axial force of the wall under dead load, $F_{l}$ represents the standard value of the maximum axial force of the wall under live load, $F_{T}$ represents average nominal tensile stress of the wall.
Figure 1. The main model of the structure.

Figure 2. Wall number.

Table 1. Tensile stress of wall.

| Wall number | Wall width (mm) | Wall length (mm) | Wall area (mm²) | Layer number | Concrete marking | \( F_e \) (KN) | \( F_d \) (KN) | \( F_L \) (KN) | \( F_T \) (N/mm²) | \( F_T/F_k \) |
|-------------|----------------|-----------------|----------------|--------------|-----------------|----------------|----------------|----------------|-----------------|-----------|
| Q1          | 500            | 1400            | 700000         | 1            | C60             | 8696           | 3598           | 283            | 7.1             | 2.5       |
|             | 500            | 1400            | 700000         | 2            | C60             | 7232           | 3470           | 276            | 5.2             | 1.8       |
|             | 500            | 1400            | 700000         | 3            | C60             | 6629           | 3405           | 273            | 4.4             | 1.5       |
|             | 500            | 1400            | 700000         | 4            | C60             | 6234           | 3321           | 268            | 4.0             | 1.4       |
|             | 500            | 1400            | 700000         | 5            | C60             | 5959           | 3239           | 263            | 3.7             | 1.3       |
|             | 500            | 1400            | 700000         | 6            | C55             | 5666           | 3132           | 256            | 3.4             | 1.3       |
|             | 500            | 1400            | 700000         | 7            | C55             | 5425           | 3048           | 251            | 3.2             | 1.2       |
|             | 450            | 1400            | 630000         | 8            | C55             | 5147           | 2949           | 244            | 3.3             | 1.2       |
|             | 450            | 1400            | 630000         | 9            | C55             | 4977           | 2869           | 239            | 3.2             | 1.2       |
4. Discussion on configuration criteria of section steel

4.1. The principle of section steel configuration in the original design

In the original design, section steel is configured when \( F_t \) exceeds \( F_{tk} \). The calculation formula for the section steel area given in the calculation book is shown in formula (1). Its main purpose is to realize that after the section steel is configured, the average nominal stress is less than \( F_{tk} \) (that is, the coefficient before \( F_{tk} \leq 1 \)). In the original design, when \( F_t \) exceeds \( 2F_{tk} \), the average nominal stress after configuration of section steel is also controlled to be around \( 2F_{tk} \); when \( F_t \) is between \( F_{tk} \sim 2F_{tk} \), the average nominal stress after configuration of section steel is also between \( F_{tk} \sim 2F_{tk} \). Through research and analysis, the original design may have misunderstood "when the steel content of section steel and steel plate exceeds 2.5%, the average nominal tensile stress can be appropriately relaxed". Therefore, the coefficient before \( F_{tk} \) is adjusted artificially to achieve a steel content of 2.5% in most of the wall sections.

4.2. In-depth understanding principles of section steel configuration in technical points

4.2.1. Explain that "the average nominal tensile stress should not exceed \( 2F_{tk} \)". In the design of frequent earthquakes, the second paragraph of Article 7.2.11 of the Technical Specification for Concrete Structures of High-rise Buildings [4] stipulates that the shear capacity of the oblique section of the eccentric tensile shear wall shall conform to formula (2).

\[
V \leq \frac{1}{\gamma_{ge}} \left[ \frac{1}{\lambda - 0.5} \left( 0.4f_t b_w h_{w0} - 0.1N \frac{A_s}{A} \right) + 0.8f_{ps} \frac{A_s}{s} h_{w0} \right] \leq 0
\]  

(2)

Where \( f_t \) represents design value of concrete tensile strength, \( A \) represents full section area of shear wall, \( A_w \) represents the area of the web of the shear wall with T-shaped or I-shaped cross-section and \( A_s \) should be taken for rectangular cross-section, \( s \) represents shear wall horizontal distribution steel bar spacing, \( h_{w0} \) represents effective height of shear wall section.

When the calculated value in square brackets at the right end of the above formula is less than \( 0.8f_{ps} \frac{A_s}{s} h_{w0} \), it shall be equal to this value. This sentence indicates that due to the large tensile force of the wall, \( 0.4f_t b_w h_{w0} - 0.1N \frac{A_s}{A} \leq 0 \), the shear resistance of the concrete part of the shear wall cannot be considered. Among them, \( A \approx A_w \approx b_w h_{w0} \), that is, \( N/A \geq 4f_t \). According to the relationship between the

|   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Q2 | 450 | 1400 | 630000 | 10 | C55 | 4786 | 2786 | 233 | 3.0 | 1.1 |   |   |   |   |
|   | 500 | 1300 | 650000 | 1  | C60 | 7501 | 4092 | 329 | 5.0 | 1.8 |   |   |   |   |
|   | 500 | 1300 | 650000 | 2  | C60 | 6531 | 3634 | 291 | 4.2 | 1.5 |   |   |   |   |
|   | 500 | 1300 | 650000 | 3  | C60 | 5974 | 3509 | 281 | 3.6 | 1.3 |   |   |   |   |
|   | 500 | 1300 | 650000 | 4  | C60 | 5537 | 3386 | 272 | 3.1 | 1.1 |   |   |   |   |
| Q5 | 500 | 1300 | 650000 | 1  | C60 | 9412 | 4099 | 329 | 7.9 | 2.8 |   |   |   |   |
|   | 500 | 1300 | 650000 | 2  | C60 | 7712 | 3794 | 303 | 5.8 | 2.0 |   |   |   |   |
|   | 500 | 1300 | 650000 | 3  | C60 | 6857 | 3698 | 296 | 4.6 | 1.6 |   |   |   |   |
|   | 500 | 1300 | 650000 | 4  | C60 | 6348 | 3611 | 289 | 4.0 | 1.4 |   |   |   |   |
|   | 500 | 1300 | 650000 | 5  | C60 | 5896 | 3472 | 278 | 3.5 | 1.2 |   |   |   |   |
|   | 500 | 1300 | 650000 | 6  | C55 | 5501 | 3370 | 271 | 3.1 | 1.1 |   |   |   |   |
standard value of tensile strength $F_{tk}$ of various types of concrete and the design value of tensile strength $f_t$, the average relationship is $F_{tk}=1.4f_t^5$. The sub-factor coefficient of seismic action is 1.3, and the design value in $N/A\geq 4f_t$ is changed to the standard value, that is, $1.3N/A\geq 4F_{tk}/1.4$, and $N/A\geq 2.2F_{tk}$ is obtained. According to the Technical Specification for Concrete Structures of High-rise Buildings, under frequent earthquakes, when the standard value of the average tensile stress generated by the axial force of the full section of the wall exceeds $2.2F_{tk}$, the shear resistance of the concrete part is no longer considered shear capacity, but the shear resistance of steel bars can be considered.

The "Technical Points" stipulates that under the action of a moderate earthquake, the average unyielding tensile stress of the wall should not exceed $2f_{tk}$ to ensure that the shear wall's shear resistance meets the performance requirements of the moderate earthquake. This requirement is essentially the same as the second paragraph of Article 7.2.11 of the "High Regulations". It is to ensure the shear resistance of the shear wall, but the technical requirements are stricter. It can be considered that the over-limit high-rise is more stringent than the ordinary high-rise design requirements.

4.2.2. Explain "The steel content of section steel exceeds 2.5%, and the nominal tensile stress can be appropriately relaxed in proportion". For concrete with strength grades C25~C80, the ultimate compressive strain is $(3.0~3.3)\times 10^{-3}$, and the ultimate tensile strain is $(0.7~1.0)\times 10^{-4}$. When the concrete crack width reaches the standard limit of 0.2~0.3mm, the stress of the steel bar is about 150~200MPa (smooth steel bar) or 200MPa~300MPa (threaded steel bar), so generally the tensile stress of the steel bar of the tension member is controlled not to exceed 200MPa. The original intention of the “Technical Points” stipulation is to control the development of cracks in tension members (to avoid through cracks), and to ensure that the members have sufficient load-bearing capacity under the reciprocating action of tension-shear and compression-shear under moderate and large earthquakes. The measure to control the development of horizontal cracks is to control the tensile stress of the steel bar to about 200 MPa.

When the section steel content does not exceed 2.5% and the nominal tensile stress does not exceed $2F_{tk}$, the tensile stress of the controllable steel bar is below 200MPa. When the section steel content is 3.8%, the average nominal stress can be relaxed to 3$F_{ta}$.

4.3. Optimum guidelines for section steel configuration
Principle 1: ($F_T > 2F_{ta}$) According to the opinions of over-limit experts, the shear wall pier at the bottom of the over-limit high-rise shall meet the requirement that the non-yielding tensile stress of moderate earthquakes should not exceed $2F_{tk}$. This requirement is a section control condition. When this condition is not met, the section steel can be configured to ensure the mid-earthquake shear resistance of the wall. The total area ($A$) of the required section steel and reinforcement is calculated according to the equivalent elastic modulus method (3). When the section steel content $\rho_s>2.5\%$, $F_{t}<\rho_s/2.5\%\times F_{tk}$.

$$F_T = \frac{F_e - F_{tk} - 0.5F_{t}}{B_sH_u - A + \frac{F_{et}}{F_{te}}} \leq 2.0F_{tk}$$

(3)

Principle 2: ($F_{tk} < F_T \leq 2F_{tk}$) The "Technical Points" stipulates that when the tensile stress of the wall limb exceeds the standard value of the concrete tensile strength, all the tensile force shall be borne by the section steel. Therefore, the area ($A_s$) of the matched section steel should also satisfy the formula (4):

$$\frac{F_e - F_{ta} - 0.5F_{t}}{F_{tk}} \leq A_s$$

(4)

Where $F_{tk}$ represents the standard value of tensile strength of section steel.
Principle 3: ($F_T > F_{tk}$) The tensile stress of the steel bar is controlled at about 200MPa.
5. Conclusions and recommendations
(1) For the shear wall at the bottom of the over-limit high-rise building in high-intensity areas, the average unyielding tensile stress of the wall limbs under moderate earthquakes is common.

(2) The original intention of the “Technical Points” stipulation is to control the development of cracks in tension members (to avoid through cracks), and to ensure that the members have sufficient load-bearing capacity under the reciprocating action of tension-shear and compression-shear under moderate and large earthquakes.

(3) Through an in-depth interpretation of the "Technical Points", the section steel optimization criteria were clearly put forward for the project, and a large amount of section steel was saved, providing a reference for other engineering practices.

(4) Whether the small eccentric tension member is equipped with section steel under the action of moderate earthquake is a design problem of the member, which can be determined by the designer according to its specific force. When the configuration of reinforcing steel bars can meet the requirements, it can be equipped with reinforcing steel bars.

(5) The seismic performance of the shear wall under small eccentric tension requires further theoretical and experimental research.

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