

Application of Concrete Filled Steel Tube Column Structural System in Calcination Furnaces

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Abstract

The complex nature of calciner construction and its resulting height places significant structural requirements on its support system. In recent years, the concrete-filled steel tubular column (CFSTC)-steel braced frame (SBF) structural system has been applied widely in calciners due to its excellent mechanical performance and ease of construction. This paper details the development of a layout scheme for the CFSTC-SBF structural system based on the calciner's structural characteristics and loads from equipment, wind, and earthquakes. With the aid of finite element analysis software, internal stress and relevant design analyses are carried out to examine the impacts of different parameters including steel tubular column cross-sectional dimensions and concrete strength grades on the performance and cost-effectiveness of the structural system, as well as the applicability of these parameters. The results show that the CFSTC-SBF structural system offers high bearing capacity, high levels of both stiffness and ductility, and can effectively withstand horizontal loads and seismic actions, thus meeting the safety and cost-effectiveness requirements of calciners. The design results of the CFSTC-SBF structural system are detailed, which, along with practical examples, provide a reference for similar engineering projects.

Keywords: Concrete-filled steel tubular column (CFSTC), Steel bracing, Frame structure, Aluminum hydroxide calcination, Towering structure.

1. Research Background and Significance

Due to continuously rising production capacity, expanding equipment size and increasing load, traditional H-shaped or cross-shaped column-SBF structures could no longer meet the stress and cost-effectiveness requirements. As a result, CFSTCs were used in high-rise buildings as early as the mid-1980s. Fully utilizing the high tensile strength of steel and the high compressive strength of concrete, CFSTCs offer significant advantages in mechanical performance and cost-effectiveness. In terms of strength, they increase compressive strength by 30–50 % (hoop effect) because of steel tubes constraint on the core concrete [1] and raise bearing capacity by 20–40 % over pure steel columns through steel tube-concrete synergy [2]. In terms of cost-effectiveness, they reduce steel usage by 20–30 % and lower the gross cost by 15–25 % [3]. With regards to construction convenience, no formwork is needed after the hollow steel tube is hoisted and cast with self-compacting concrete, thus shortening the construction period by 30 % compared to that of concrete structures [4]. The fire resistance performance achieves a fire endurance of up to 2.5 hours and can well meet the requirement of *Code for Fire Safety of Steel Structures in Buildings* (GB51249-2017)[5], with no need for fireproof coating.

Technical Specification for Concrete Structures of Tall Building [6] lists CFSTCs as a recommended solution for high-seismicity areas. Calciners are characterized by a large single column reaction force (over 15 000 kN), high process equipment density, numerous planar

openings, a high floor height, non-continuous frame beams, and a wide column grid layout (generally with a span of 10–13 m). Given these characteristics, adopting the CFSTC-SBF structural system holds great practical significance.

2. Design Theory of CFSTC

The design theories recommended in GB 50936-2014 [7] are respectively the unified theory of steel-reinforced concrete and the experiment-based limit equilibrium theory. The unified theory of steel-reinforced concrete considers it as a composite material, analysing its composite working performance based on its properties such as uniformity, continuity, and correlation. Instead of focusing on a specific critical section of the CFSTC, the limit equilibrium theory examines the entire length of the CFSTC, or the so-called unit column, taking it as the fundamental element of the structural system. By applying the broader concepts of stress and strain in the limit equilibrium theory, the broader yield conditions of the unit column under the combined action of the two broader stresses, namely axial force N and column end bending moment M , are examined based on the experimental observations.

3. Case Study

A refinery adopts CFSTC-SBF as the structural system of its calciner and uses corrugated steel plate as the floor slabs. The main structure of the calciner is 58.5 m high, with a total of 8 primary floors. The column grid, typical floor layout and elevation drawing are shown in the figures below.

Main design parameters: the site is classified as Category II, with a seismic fortification intensity of 6 degrees (0.05 g) and a reference wind pressure of 0.75 kPa.

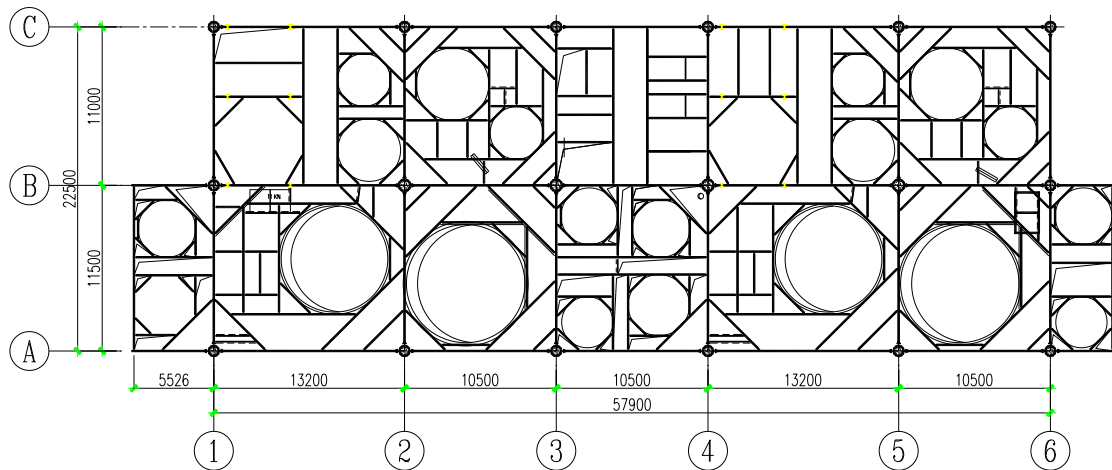


Figure 1. Typical plan layout 1.

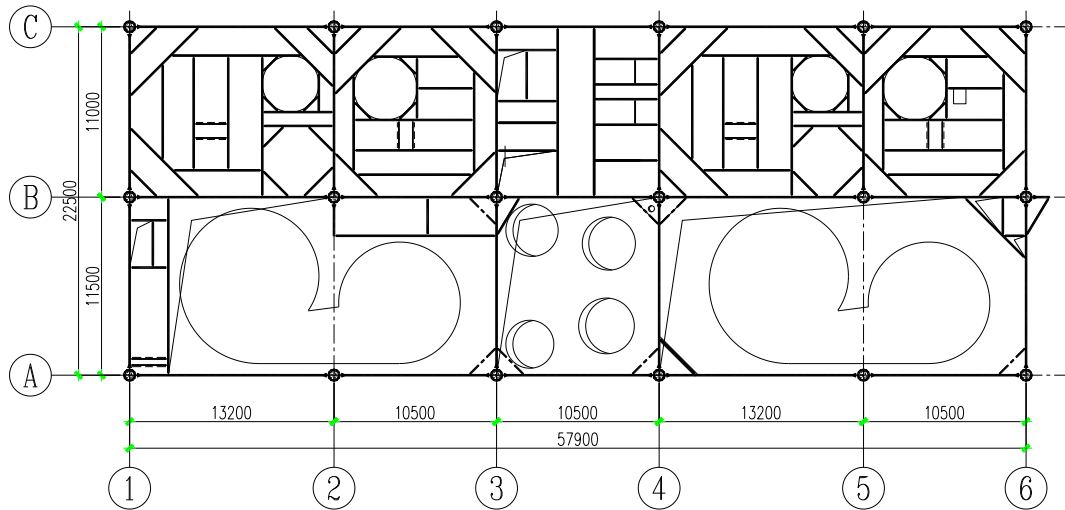


Figure 2. Typical plan layout 2.

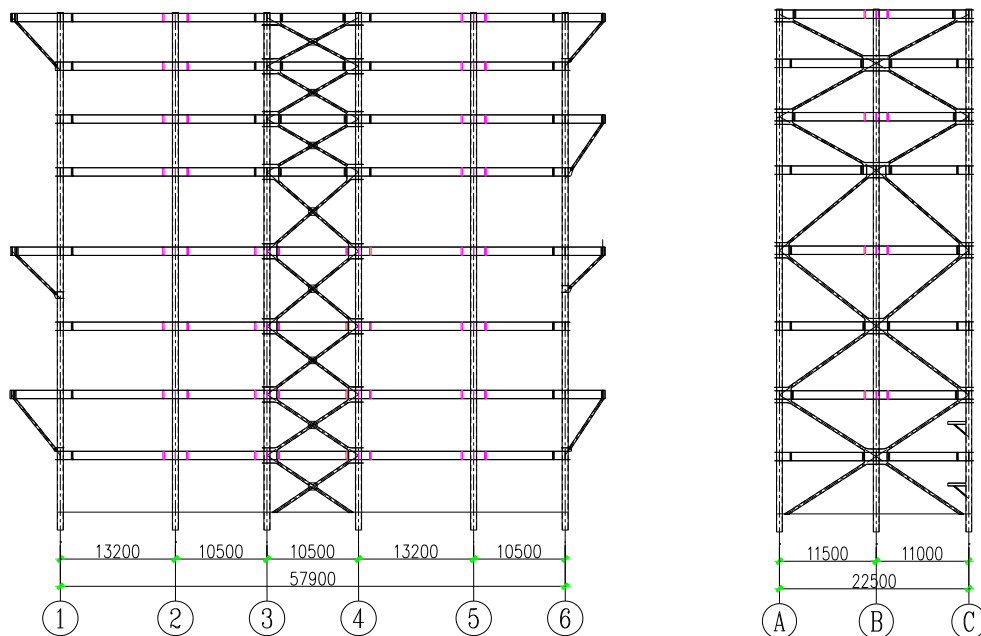


Figure 3. Typical elevation view.

4. Structural Design

4.1 Design Theory

The unified design theory is adopted for the CFSTC design of this project. Under the complex stress state resulting from the joint actions of compression, bending, torsion, and shear, the bearing capacity of the column is calculated using the following formulae (1-3):

When $\frac{N}{N_u} \geq 0.255 \left[1 - \left(\frac{T}{T_u} \right)^2 - \left(\frac{V}{V_u} \right)^2 \right]$:

$$\frac{N}{N_u} + \left(\frac{\beta_m \cdot M}{1.5 M_u \cdot (1 - 0.4N/N_E)} + \left(\frac{T}{T_u} \right)^2 + \left(\frac{V}{V_u} \right)^2 \right) \leq 1 \quad (1)$$

When $\frac{N}{N_u} < 0.255 \left[1 - \left(\frac{T}{T_u} \right)^2 - \left(\frac{V}{V_u} \right)^2 \right]$:

$$-\frac{N}{2.17 N_u} + \left(\frac{\beta_m \cdot M}{M_u \cdot (1 - 0.4N/N'_E)} + \left(\frac{T}{T_u} \right)^2 + \left(\frac{V}{V_u} \right)^2 \right) \leq 1 \quad (2)$$

with
$$N'_E = \frac{\pi^2 E_{sc} \cdot A_{sc}}{1.1 \lambda^2} \quad (3)$$

where:

N, M, T, V Respectively the design values of axial pressure, bending moment, torque, and shear force acting on the component

N_u, M_u, T_u, V_u Respectively the axial compressive, the flexural, the torsional and the shear strengths for a solid or hollow concrete-filled steel tubular component

β_m Equivalent bending moment coefficient [8]

N'_E Coefficient, which can be simplified to:

$$N'_E = 11.6 k_E f_{sc} A_{sc} / \lambda^2$$

Note from the ICSOBA Editor: Description of E_{sc} , A_{sc} and λ has not been provided on time, those variables appear in traditional buckling theory as Modulus of Elasticity (Young) of Steel in Composite Section, Cross-sectional Area of Steel in Composite Section and Slenderness Ratio (effective length to gyration radius) of the column.

4.2 Simplified Computational Model

This project uses the Pre-Dimensioning and Pre-Analysis Modular (PKPM) finite element software developed by China Academy of Building Research to establish a spatial model for computation and analysis. Vertical load at the supporting equipment is entered for computation, with consideration given to both the additional load at the centre of the equipment body and the seismic additional effect. The structural damping ratio is set as 0.035.

By adjusting the beam column section dimension, wall thickness, concrete strength grade, and hoop coefficient, the optimal component dimensions are obtained to form a structural system that meets the overall indicators set forth in relevant specifications and standards.

4.3 Design Results

4.3.1 Structural Natural Vibration Period and Mass Participation Coefficient

The first vibration mode is longitudinal translation with a period of 2.177 s, the second one is lateral translation with a period of 1.596 s, and the third one is torsional vibration with a period of 1.422 s. In the design, the supporting position is adjusted according to the equipment load to align the centre of mass with the centre of stiffness as much as possible, thereby ensuring the continuity of the structural system's stiffness. In total, 33 vibration modes were considered in the computations, resulting in an effective mass coefficient of 96.56 % for the first earthquake direction (EX) and 90.16 % for the second earthquake direction (E), both of which meets the relevant regulatory requirements.

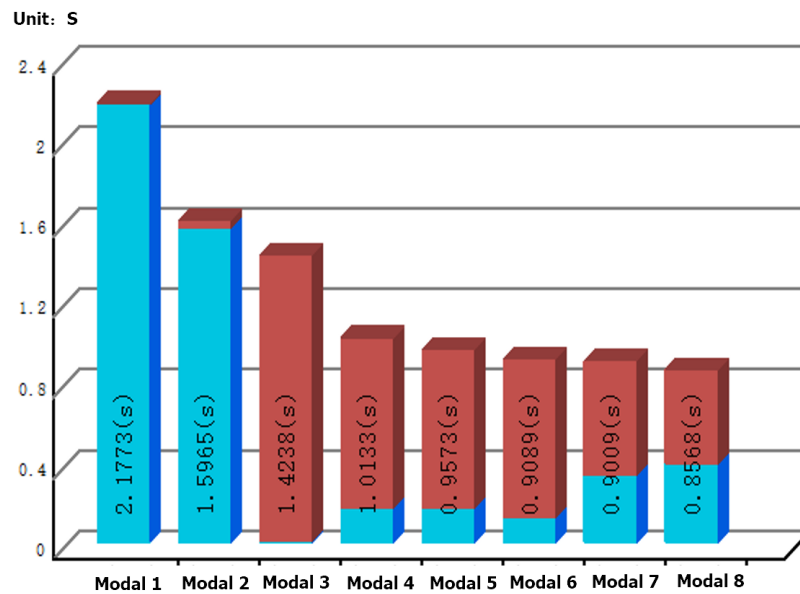


Figure 4. Natural vibration periods.

4.3.2 Shear Strength and Its Ratios for Different Floors

The shear strength and its ratios for all the floors meet the requirements specified in section 3.5.3 of the *Technical Code for Concrete-filled Steel Tubular Structure* [7], as shown in Table 1.

Table 1. Shear strength and its ratios for different floors.

Floor No.	V _x (kN)	V _y (kN)	V _x /V _{xp}	V _y /V _{yp}	Ratio Assessment
8	30 236.22	36 744.46	1.00	1.00	Compliant
7	25 700.79	30 994.31	0.85	0.84	Compliant
6	28 195.98	36 302.43	1.10	1.17	Compliant
5	22 978.90	30 130.66	0.81	0.83	Compliant
4	37 184.05	39 311.63	1.61	1.29	Compliant
3	31 636.39	42 965.75	0.85	1.09	Compliant
2	46 282.03	69 686.78	1.46	1.62	Compliant
1	40 814.84	72 350.57	0.88	1.04	Compliant

4.3.3 Deformation Verification

The horizontal force generated by wind load in this example is much greater than that from earthquake action. Table 2 only lists the deformation indicators under wind load in the X and Y directions. The maximum floor displacement in the X direction of the entire building was 43.08 mm ((occurring at Tower 1 on the 8th floor), and the maximum inter-floor displacement angle was 1/895 ((occurring at Tower 1 on the 3rd floor). The maximum floor displacement in the Y direction of the entire building was 68.44 mm ((occurring at Tower 1 on the 8th floor), and the maximum inter-floor displacement angle was 1/513 ((occurring at Tower 1 on the 5th floor). As shown in Table 2, both displacement values are less than 1/300 and meet the requirements.

Table 2. Displacements and displacement angles in X and Y directions under wind load conditions.

Floor No.	Maximum Displacement in X Direction (mm)	Maximum Inter-floor Displacement Angle in X Direction	Maximum Displacement in Y Direction (mm)	Maximum Inter-floor Displacement Angle in Y Direction
8	43.08	1/2553	68.44	1/970
7	42.07	1/2097	67.14	1/884
6	39.81	1/1682	64.27	1/849
5	36.55	1/928	60.51	1/513
4	27.27	1/1036	42.53	1/706
3	19.66	1/895	30.76	1/608
2	11.17	1/1104	17.69	1/679
1	5.20	1/1374	7.88	1/907

5. Conclusions

1. In the design of CFSTCs, the first step is to determine the preliminary section dimensions based on the floor height and aspect ratio. Then, the matching concrete strength should be selected according to steel strength grade, and the appropriate hoop coefficient (between 1.5 and 2.0) should be iteratively verified and adjusted to identify the optimal steel tubular section dimensions for the component;
2. Through real-world case study, this paper shows that CFSTCs can achieve favourable cost efficiency for calciners with large vertical loads and column grid spans, saving more than 40 % of steel compared to traditional H- and cross-shaped steel columns or cross shaped columns;
3. A matching bracing structural system is adopted for the steel tubular columns, which can fully leverage their compressive strength and improve the overall structural lateral resistance, thus ensuring good structural safety and economy.

6. References

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