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Evaluation of Load-Bearing Performance of Existing Cast Steel Node

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Abstract

This paper presents a preliminary evaluation of the load-bearing performance of an existing cast steel node in a constructed tennis stadium using numerical simulations and non-destructive field tests. Given the absolute stress values of the existing cast steel node were immeasurable, the accuracy of the numerical simulations were verified by comparing the stress increments derived from numerical simulations and non-destructive field tests. During the experiment, the existing cast steel node was loaded indirectly by moving the retractable roof to three different positions (i.e. closed, semi-opened and fully-opened configurations); thus, only the stress increments were recorded. Three simplified truss models and one solid finite-element model were developed to simulate the stress distributions with the corresponding roof positions. A comparison suggests that the stress increments simulated with the developed finite-element models were in good agreement with experimental results. Therefore, the simulated stress distributions can be used to judge the load-bearing performance of the existing cast steel node.

Keywords: Existing Cast Steel Node; Stress Increment; Numerical Simulation; Non-Destructive Field Test.

1. Introduction

To date, cast steel nodes are widely used as connecting joints in large span spatial structures, such as stadiums, airport terminals and public transportation hubs, given their advanced mechanical performance and flexible forms [1]. Due to the complex configurations and stress distributions of cast steel nodes [2], engineers often combine numerical simulations and full-scale destructive tests to ensure sufficient load-bearing capacity. Essentially, a finite-element model is developed in advance to predict the ultimate load-bearing capacity, while a full-scale experiment is performed to mimic the real-world loading conditions. The experimental results are often considered a baseline to validate the accuracy of the developed finite-element model.

However, gaps exist between laboratory tests and practical applications. First, the laboratory test method is not suitable for existing cast steel nodes. Second, given that cast steel nodes are often connected to a number of asymmetric trusses, load combinations and boundary conditions are difficult to simulate in a laboratory. Thus, to the best of the author's knowledge, field tests cannot be replaced completely.

Currently, there are four primary constraints that prevent field tests from being applied to existing cast steel nodes and experimental results from being compared to numerical simulations.

- Compared to laboratory tests, loads cannot be directly applied to an existing cast steel node.
- If loads are applied indirectly, e.g. loads applied to neighbouring structural components, the experimental results are meaningless unless the load distributions are calculated correctly.
- Traditional human-operated measurement instruments cannot be installed and operated unless there is an aerial work platform.

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- The initial parameters (e.g. initial strain, stress, etc.) of an existing cast steel node may not have been recorded; thus, experimental results may be incomparable to those obtain by numerical simulation.

This paper reports an effort to overcome the above constraints by studying the load-bearing performance of an existing cast steel node in a constructed tennis stadium using numerical simulations and non-destructive field tests. Given that initial stress and strain data have not been recorded, which make the absolute stress values immeasurable, the accuracy of numerical simulation is examined by comparing the predicted stress increments to those derived from field tests.

During field tests, the existing cast steel node was loaded indirectly by moving the retractable roof to three different positions (i.e. closed, semi-opened and fully-opened configurations).The stress increments were measured by using the fibre-optic strain gauges. To better mimic the experimental conditions, three simplified truss models simulating the entire stadium were developed with the roof positioned at corresponding places. The calculated truss forces were then converted to uniform pressures and treated as external loads in a solid cast steel node model. The stress increments predicted by the solid model were used for comparison.

The experimental results together with the results of the numerical simulation suggested that the stress increments simulated with the developed finite-element models were in good agreement with experimental results. Therefore, the predicted maximum stress can be used to judge the load-bearing capacity of the existing cast steel node.

2. Related Work

A cast steel node was first developed to replace welded connections in steel offshore platforms and has been used extensively in building structures since 1980s [3]. Many researchers have investigated its static strength and ultimate capacity before 2000, as summarized in Oliveira, 2006 [4]. Recently, researchers in this domain have paid more attention to its fatigue property. Sturm et al., 2003 evaluated the fatigue behaviour of several cast steel nodes in bridge structures by experimental investigations [5]. Oliveira, 2006 studied the fatigue performance of a cast steel connector by doing seismic analysis [4]. Wang et al., 2013 and Han et al., 2016 investigated the welds between a node and its neighbouring trusses by using fracture mechanics and finite-element analysis [1, 6].

On the other hand, years of sport stadium constructions, particularly after Beijing Olympics, provide a rich set of literatures on the load-bearing capacity of cast steel node in China. However, most researchers in China evaluated the load-bearing capacity by doing destructive tests in laboratory [7-9]. In other words, their research focused on examining the ultimate strength of designed cast steel nodes prior to construction rather than analysing the performance of existing cast steel nodes.

3. Project Background

The Guanggu tennis stadium is located in Wuhan City, China, and has a seating capacity over 15,000. As illustrated in Figure 1(a), the stadium has a fixed roof and a retractable roof to cover the seating and playing areas, respectively. The fixed and retractable roofs are supported by 16 cast steel nodes located at the four corners of the stadium, as indicated in Figure 1(b). Considering the symmetrical arrangement of the stadium, only one cast steel node was selected in this study. Detailed AutoCAD drawings of the cast steel node are shown in Figure 2.

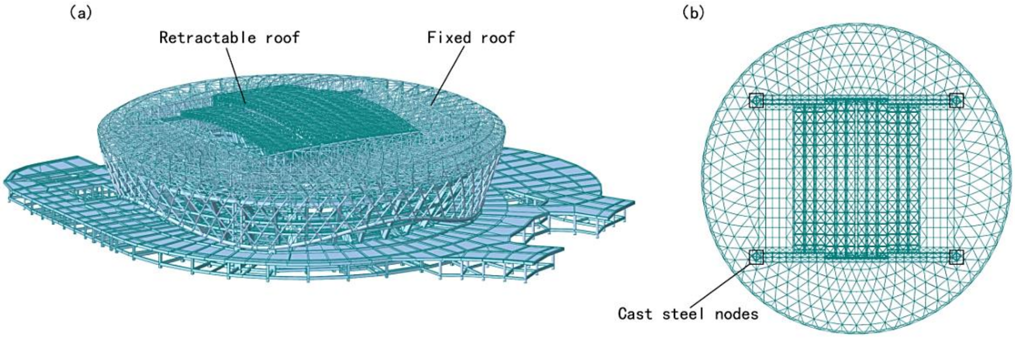

Figure 1. Schematic of the Guanggu tennis stadium: (a) three-dimensional view of the entire structure and (b) plan view showing the positions of the cast steel nodes

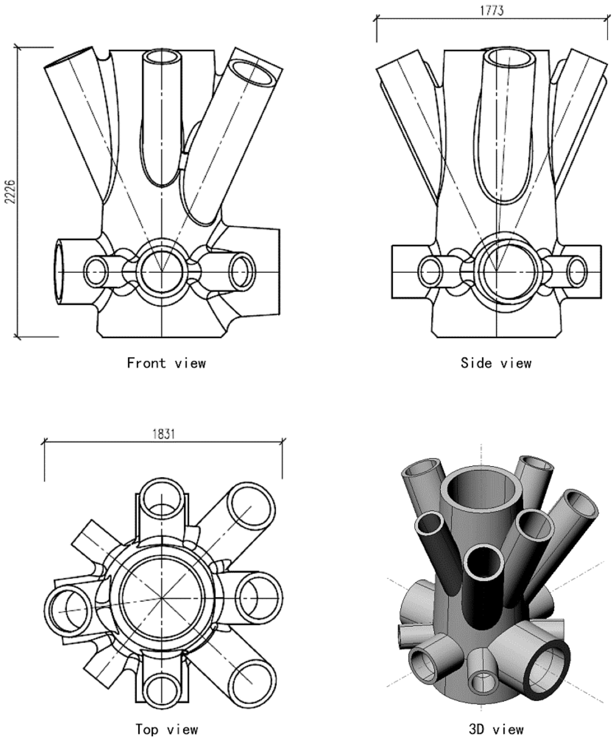

Figure 2. AutoCAD drawings of the cast steel node

The cast steel node is made of ZG420-530 alloy steel. Based on the Chinese GB 50017 and DG/T J08-52 standards, its material properties were assumed to be elastic with density ρ = 7800 kg/m^3, Young's modulus E = 200,000 MPa, Poisson's ratio μ = 0.3, yield stress σ_s = 420 MPa and ultimate stress σ_b = 530 MPa. These material properties are important in the development of finite-element models in numerical simulations and converting from measured strain increments to stress increments in field tests [10, 11].

4. Numerical Simulations

Two commercial finite-element software packages, i.e. Midas Gen and Midas FEA, were adopted in this study [12]. Even though both of them are operated by MIDAS IT, the Midas Gen is more suitable for linear elastic calculations, while the Midas FEA supports advanced nonlinear simulations. As illustrated in Figures 3(a, b and c), the entire tennis stadium was simplified to three truss finite-element models with different roof configurations (i.e. closed, semi-opened and fully-opened) under Midas Gen. These simplified truss models were utilized to determine the axial force in each neighbouring truss of the cast steel node. A solid model, shown in Figure 3(d), was then developed under Midas FEA to simulate the cast steel node, where the axial forces derived from Midas Gen were treated as external loads. A four-node three-dimensional solid element (Tet4) was used for meshing the cast steel node. The average dimension, minimum and maximum size of the meshed elements were summarized in Table 1. In addition, the boundary conditions for the solid model consisted of fixing the bottom of the cast steel node and equally distributing the axial forces transmitted from neighbouring trusses, as illustrated in Figures 3(d) and 4(b).

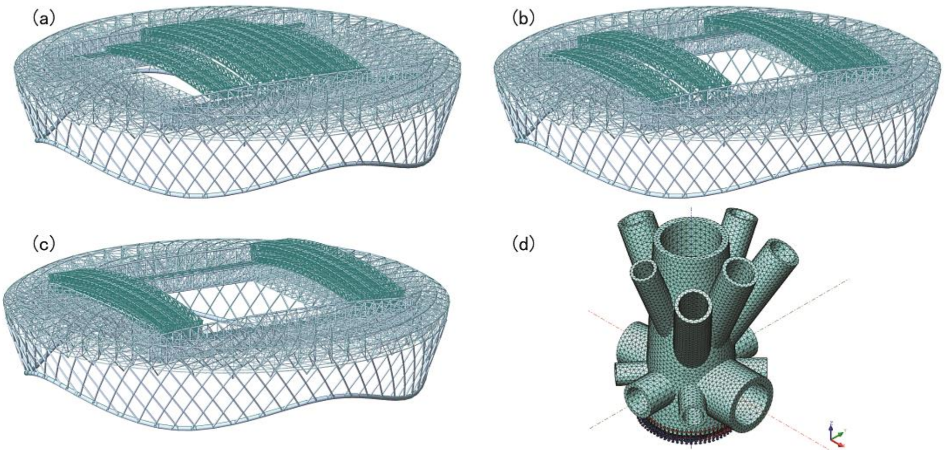

Figure 3. Finite element models developed in this study: (a) simplified truss model of the closed roof configuration; (b) simplified truss model of the semi-opened roof configuration; (c) simplified truss model of the fully-opened roof configuration; (d) meshed solid model of the cast steel node

Table 1. Mesh characteristics of the solid finite-element model

Element shape	Number of elements	Average dimension	Minimum size	Maximum size
Tetrahedron	68,104	50mm	Approximately 20 mm	Approximately 90 mm

Taking the closed roof configuration as an example, the axial forces in neighbouring trusses were first calculated using the simplified truss model, as shown in Figure 4(a). They were then converted to uniform pressures and applied to the cast steel node in the solid model, as illustrated in Figure 4(b). Finally, the stress diagram was generated (Figure 4(c)), and the maximum stress was determined to be 162 MPa. Following the same methodology, stress diagrams of the semi-opened and fully-opened roof configurations were generated (Figure 5), where the maximum stresses were 145 MPa and 130 MPa, respectively. Given that the yield stress of the cast steel node is 420 MPa, which is much greater than the calculated maximum stress, the cast steel node is considered to be in good condition. However, the accuracy of numerical simulation needs to be experimentally validated. Hence, the stress increments estimated by the finite-element models are reported later in comparison to the results of the field test.

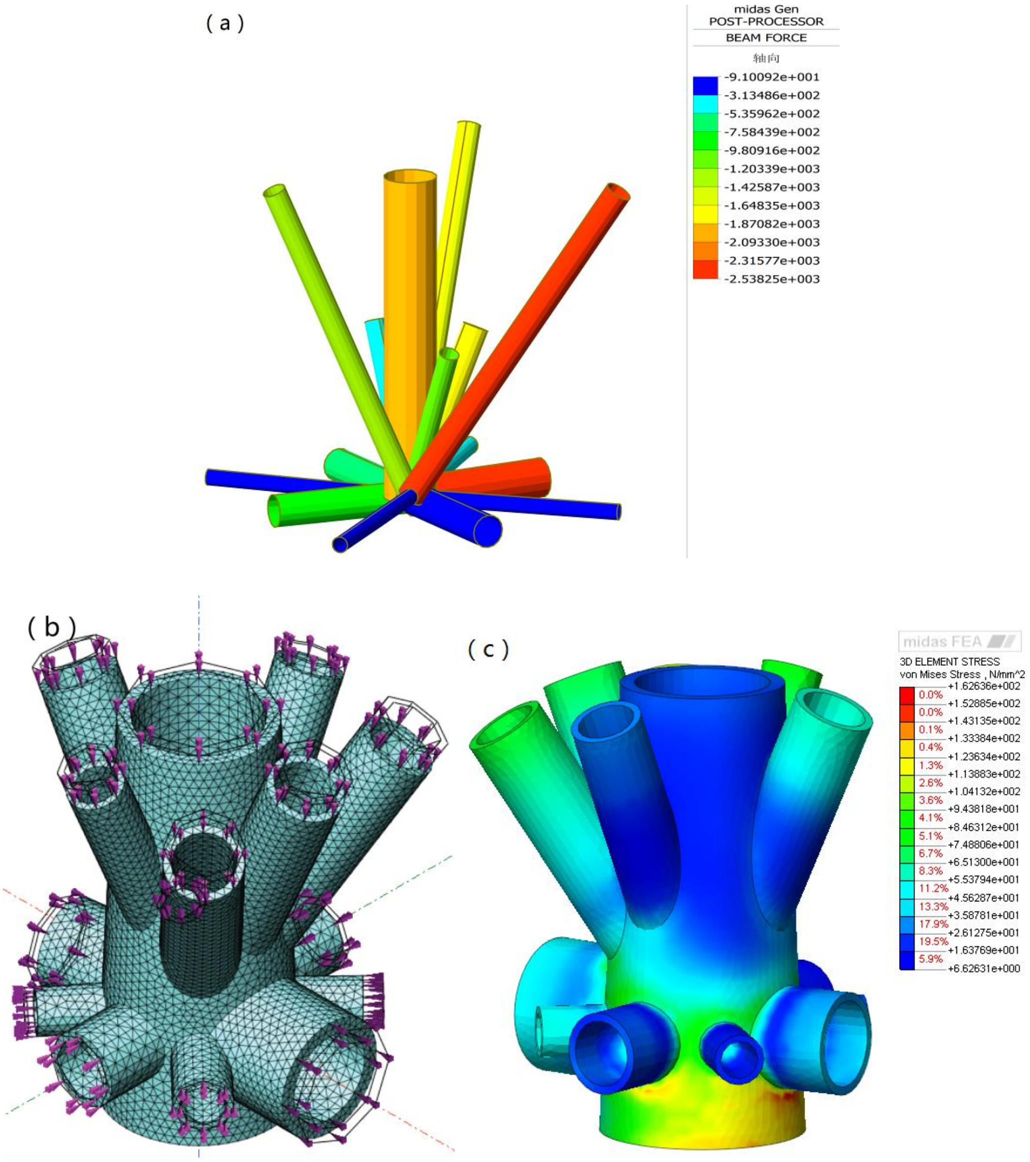

Figure 4. Illustration of the numerical simulation: (a) axial force diagram derived from the simplified truss model; (b) uniform pressures in the solid model; (c) stress diagram of the cast steel node for closed roof configuration

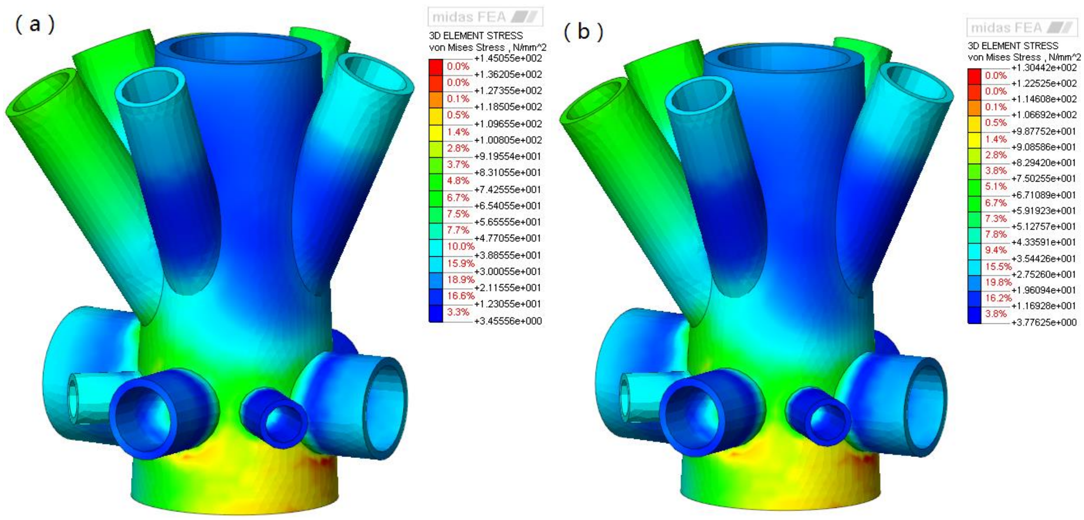

Figure 5. Stress diagrams of the cast steel node for (a) semi-opened roof configuration and (b) fully-opened roof configuration

5. Non-Destructive Field Tests

One set of non-destructive field tests was systematically organized and carried out to measure the stress increments of the cast steel node with the previously described roof configurations. The instruments utilized in the field tests were an onsite laptop, one data acquisition toolbox, and some fibre-optic strain gauges. As illustrated in Figure 6, the fibre-optic strain gauges were fixed to the node's surfaces along the axial directions of the neighbouring trusses. Note that the paint on the steel surface was polished in advance following the recommendations by Geokon, Inc [13]. The laptop and data acquisition toolbox were connected to the fibre-optic strain gauges by fibre-optic cables.

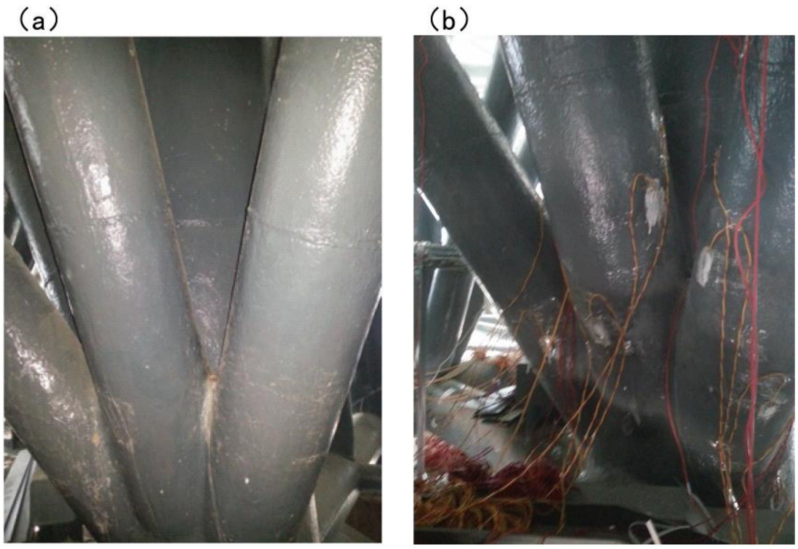

Figure 6. Cast steel node: (a) before the test and (b) during the test

Considering the feasibility of installing the fibre-optic strain gauges, the experiments only covered the upper half of the cast steel node, as indicated in Figure 7. The purpose of the field tests was to validate the accuracy of numerical simulations rather than thoroughly inspect the load-bearing capacity; therefore, the author assumed that this incompletion did not cause significant error in the comparison of numerical simulations and experimental results. The tests were organized according to the Chinese CECS 333 standard [14]. To minimize the dynamic effect, data were collected only when the roof stopped moving completely. In addition, tests were conducted on a cloudy day with temperatures slightly fluctuating between 28 to 30 degrees Celsius with wind gusts less than 5 m/s. Therefore, the external factors were assumed to have little effect on the experiments.

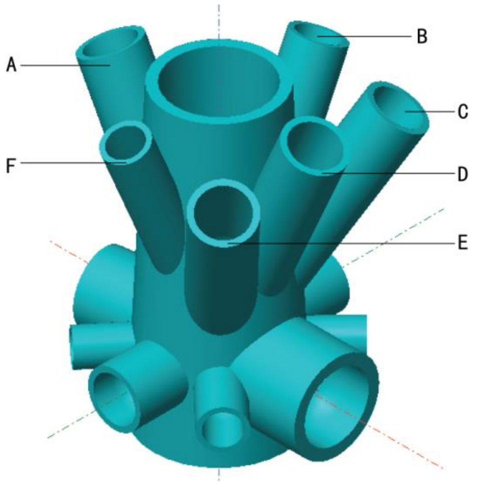

Figure 7. Labelled positions of the cast steel node covered in the field tests

6. Comparison Between Numerical Simulations and Experimental Results

Prior to the comparison, stress increment must be defined. In this study, stress increment one ($\Delta\sigma_1$) is calculated by subtracting the stress of the closed roof configuration from that of the semi-opened configuration, and stress increment two ($\Delta\sigma_2$) is calculated by subtracting the stress of the semi-opened configuration from that of the fully-opened configuration. The deviations are determined by subtracting the simulation results from the experimental results. Table 2. summarizes the stress increments obtained from the numerical simulations and the experiments. Overall, the predicted stress increments match the experimental results fairly well. Among the calculated deviations, only stress increment one ($\Delta\sigma_1$) at Position B reached 10 MPa. This agreement suggests that the developed finite-element models are suitable for evaluating the load-bearing performance of the cast steel node.

Table 2. Comparison of stress increments determined from numerical simulations and field tests ($\Delta\sigma_1$ and $\Delta\sigma_2$ denote stress increments one and two, respectively)

Labelled positions (Figure 7)		Simulation results	Experimental results	Deviations
A	$\Delta\sigma_1$	15.4	15.2	−0.2
	$\Delta\sigma_2$	7.0	6.9	−0.1
B	$\Delta\sigma_1$	−4.7	5.4	10.1
	$\Delta\sigma_2$	3.7	2.1	−1.6
C	$\Delta\sigma_1$	−6.1	−9.4	−3.3
	$\Delta\sigma_2$	8.2	2.9	−5.3
D	$\Delta\sigma_1$	−1.1	0.8	1.9
	$\Delta\sigma_2$	4.0	3.8	−0.2
E	$\Delta\sigma_1$	13.0	10.4	−2.6
	$\Delta\sigma_2$	8.0	6.5	−1.5
F	$\Delta\sigma_1$	15.2	9.8	−5.4
	$\Delta\sigma_2$	6.3	3.9	−2.4

7. Conclusion

Cast steel nodes are widely used as connecting joints in large span spatial structures; therefore, their load-bearing performance must be evaluated carefully. This paper has presented a preliminary evaluation of an existing cast steel node using numerical simulations and non-destructive field tests. By comparing the stress increments, the simulation results are considered in good agreement with the experimental results. Therefore, the existing cast steel node is considered to have sufficient load-bearing capacity. Several limitations associated with this preliminary study should be mentioned. First, given that the absolute stresses of the cast steel node were immeasurable, the author had no choice but to use stress increment comparisons, which may somewhat affect the accuracy of the evaluation. Moreover, the existing cast steel node was loaded by moving the retractable roof to different positions. For cast steel nodes in other projects, this methodology may not be suitable.

8. Acknowledgments

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