

Article

Dynamic Test and Analysis of the Structure of the Stadium Stand in Suzhou Industrial Park

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Abstract: The test object is the grandstand of the Suzhou Industrial Park Stadium. First, the structure was tested for dynamic characteristics, including natural frequency, mode test, and damping test. Then, the structure was subjected to a human-induced vibration response test. Lastly, the finite-element modeling of the structure was carried out, and the simulated and measured values were compared. The test results of the dynamic characteristics of the structure are consistent with the calculated values, and the finite-element modeling of the structure was accurate and reasonable. The natural frequency of structural modal test areas 1 and 2 met the 3 Hz limit vibration specified by JGJ/T 441-2019, and the second-order resonance frequency was still within the range of pedestrian load frequency, which may still cause comfort problems. Test results show that the presence of people increases the damping effect of the structure.

Keywords: damping ratio; dynamic characteristics test; structural mode; human-induced vibration response



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1. Introduction

With the improvement of building material performance and building technology, the style of building structure is showing increasing diversity. Demand for building functions has led to the development of building structures towards larger spans and lighter weight [1–4]. This gives rise to comfort problems, especially for large buildings (large ballrooms, high-speed railway stations, etc.), long pedestrian bridges, and buildings with large overhanging spaces [5–12].

When considering the comfort problem caused by vertical vibration, the vertical vibration mode of the structure should normally be calculated first. When there are modes with frequencies below or slightly above 3 Hz, and the vertical mass participation coefficient of this model is large, the simplified or finite-element calculation of the vibration acceleration response under the crowd load is carried out mainly at this frequency. Modal analysis is the basis of comfort calculation. Multiple crowd load conditions should be arranged at the large amplitude of the modal vibration pattern [13]. Conditions should be designed according to the building use requirements and the actual possible situations [14–17]. The comfort standard should be determined according to building use requirements [8]. For cases that do not meet the comfort requirements, consideration should be given to changing the structural arrangement to improve the structural modal frequency or using multiple tuned mass dampers (MTMDs) for vibration damping control [3,18–20]. For a structure using MTMD, the acceleration response calculation of the structure before and after damping should be evaluated to reasonably select the tuned mass damper (TMD) parameters and arrangement position.

Thus, this paper takes the overhanging grandstand of the Suzhou Industrial Park stadium project as the research object, and the structural vibration response of this large building cover under random crowd excitation load mode is analyzed. At the same time,

by calculating the response at the maximal vibration response point of the grandstand under a specific crowd load, the dynamic characteristics and vibration response of the stadium were tested on-site. The total construction area of Suzhou Industrial Park Stadium is 88,770 m², the eave height of the building is 55 m, the east–west length of the main grandstand is about 240 m, and the north–south length is about 210 m. Both sides are large overhanging structures, with maximal overhang length of 10.4 m. After calculation, the vertical self-oscillation frequency of the first few steps there was in the range of 3–4.5 Hz. The excitation frequency generated by people walking was 1.5–3.2 Hz, so when intensive activities of people occur in the stands, it is easy to cause structural resonance.

2. Method

2.1. Floor Dynamic Test

To obtain the dynamic characteristics of the building floor, it was tested. The main test instruments include vibration pickup (941-B, designed by the Institute of Engineering Mechanics, China Seismological Bureau; transmission bands were 0.25–80 Hz; measurement resolution was 5×10^{-6} m/s², weight was 0.875 kg, size was 55 × 55 × 72 mm; Analyzer System-Instrument (AZ-308, Nanjing Analyzer Software Engineering Co., Ltd., Nanjing, China; laptop, China Hewlett-Packard Co., Ltd., Nanjing, China). The main testing equipment is shown in Figure 1.

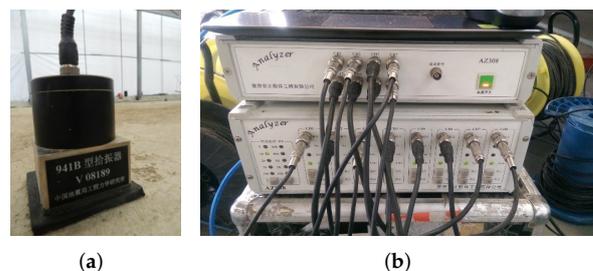


Figure 1. Instrument installation schematic. (a) Vibration pickup; (b) Analyzer System Instrument.

2.2. Natural Frequency, Modal Shape, and Damping Ratio Test

The main methods of structural dynamic property testing are the artificial excitation method and the environmental random vibration method. Among them, the structural response signal to environmental excitation can be used to identify the modal parameters of the system. This method requires the excitation signal to be white noise or an ergodic process. The excitation signals of actual projects can generally meet the requirements. The ambient random vibration method is one of the more used methods in building structure testing, and its biggest advantage is that it does not require artificial excitation and is especially suitable for measuring the dynamic characteristics of the whole structure [19,21].

The modal test uses dynamic testing under ambient vibration excitation. The test network was established according to the site conditions and stadium shape and divided into two regions. There are 66 measurement points in this test, of which each region is set up with the reference point of measurement point 35. In this building, we only considered the acceleration comfort in the vertical direction, so vertical acceleration was used to identify the modal parameters. Dynamic characteristic parameters were tested using the modal parameter identification method with ambient random excitation, and the modal parameter identification was performed on the collected time-range data of the vertical acceleration of the structure surface in each region. Figure 2 shows the two modal test area measurement points.

Damping testing methods are divided into time- and frequency-domain methods, such as the logarithmic-reduction, resonance-frequency, transfer-function, and half-power bandwidth methods. Because the time-domain method has strict requirements on the waveform, the traditional half-power bandwidth method in the frequency domain method is the most commonly used in engineering [22–24]. In this test, the natural vibration

frequency of the structure was determined according to the dynamic characteristic test, and the force hammer was used to excite the 23 points. Then, the free attenuation signal in single mode was extracted through the bandpass filter, and the damping ratio of the first vertical vibration mode of the floor was lastly obtained by using the logarithmic-reduction method.

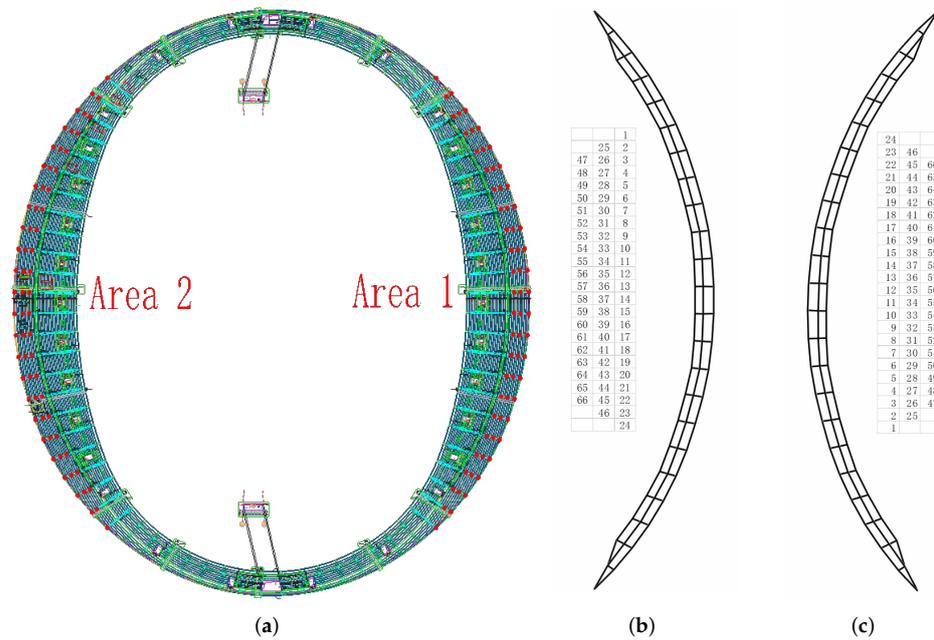


Figure 2. Area and number of dynamic characteristics measurement points. (a) Modal test area; (b) Area 1; (c) Area 2.

2.3. Crowd Load Response Test

According to the structural design requirements, a vibration test under crowd load excitation was carried out on the stands. It should be ensured that there is no construction in the test and surrounding areas, environmental interference is small and can be ignored, and the actual test results are considered to be true and reliable. The vibration pickup was fixed on the floor with strong glue (acrylic adhesive, Beijing Chemical Plant Co., Ltd., Beijing, China) to ensure testing accuracy and data reliability.

The measurement is carried out in different regions because of the large structure of the Suzhou Industrial Park Stadium. During the test, the average weight of the walking experimenter was close to 70 kg. To allow for the participants to have the same walking frequency, many training sessions were carried out.

The test conditions are shown in Table 1, including jumping and walking frequencies. The jumping experiment was performed with all the people jumping at the same time. Normal walking was all 50 people walking in randomly in the structure. Quick stepping was all 50 people walking at the same step in the same direction in the structure. Two sets of tests were carried out for each working condition, and measuring points were arranged into 6 measuring points (see Figure 3).

Table 1. Field test conditions.

Condition Number	Condition Type	Number of People	Frequency (Hz)	Test Time (s)
T1	Jumping	50	1.5	32
T2	Normal walking	50	1.5	32
T3	Normal walking	50	1.7	32
T4	Quick stepping	50	1.5	32

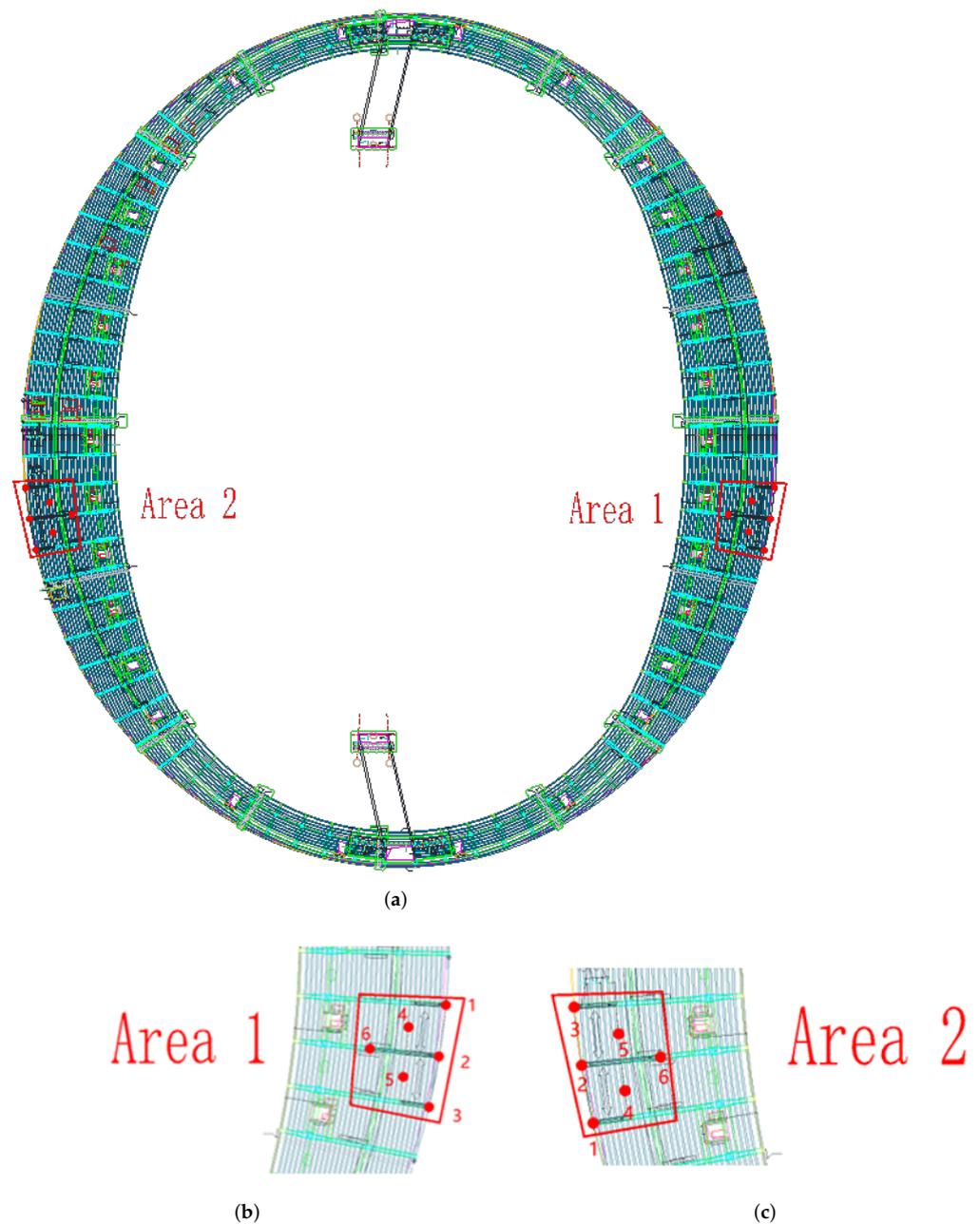


Figure 3. Distribution map of measuring points. (a) Area of crowd load response test; (b) Area 1; (c) Area 2.

3. Results and Analysis

3.1. Natural Frequency, Modal Shape, and Damping Ratio Test Results

Figure 4 shows the shape of the first-order frequency mode of each measurement area of the modal area. Figure 5 shows the measured power spectrum of the measuring points in the modal area.

The actual measurement showed that the vertical mode of the two-mode measurement area of the original structure of the grandstand was 3.75 and 3.95 Hz (Figure 5), which is likely to cause second-order resonance within the frequency range of a crowd walking.

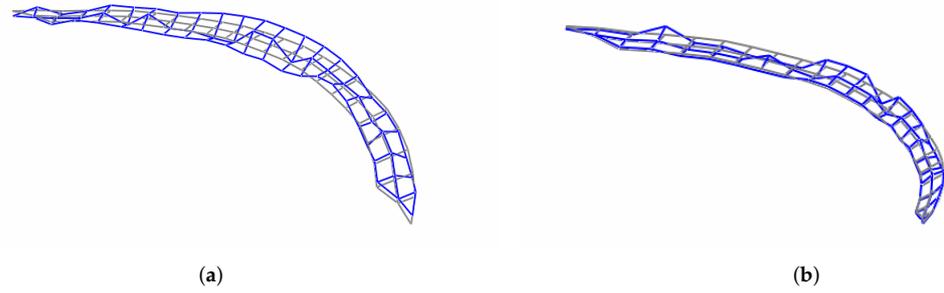
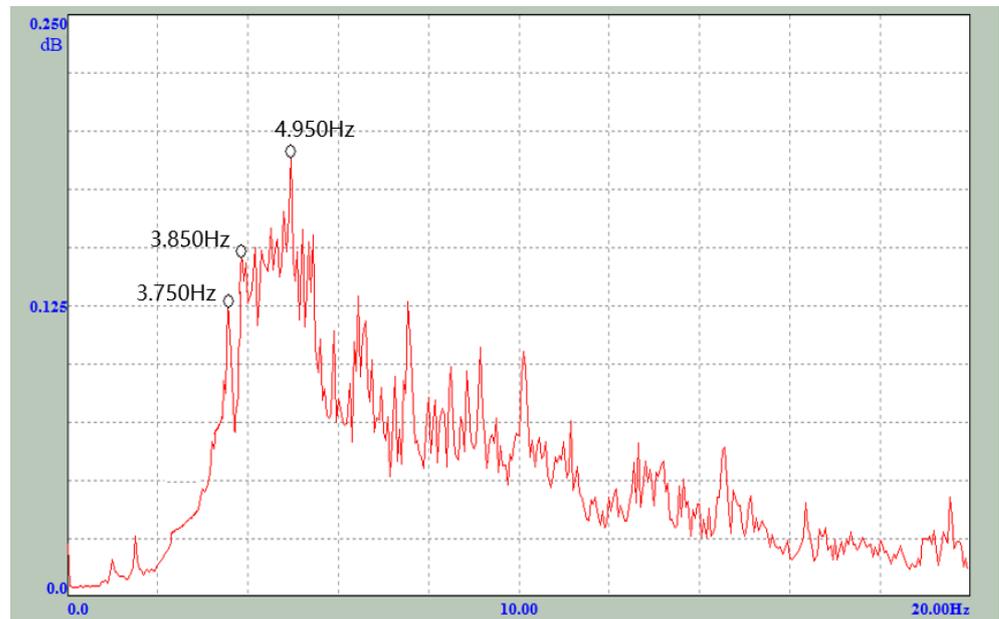
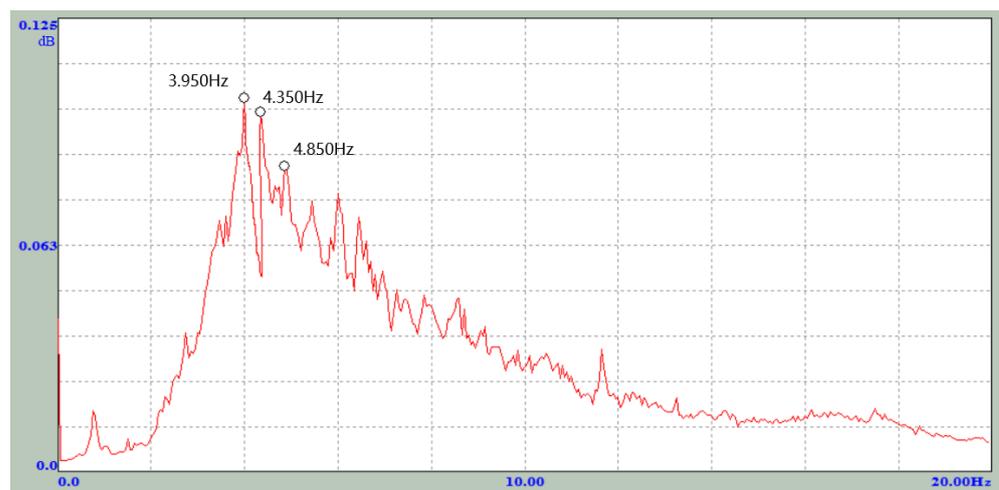


Figure 4. First-order vertical mode shape of (a) Area 1 and (b) Area 2.



(a)



(b)

Figure 5. Self-power spectrum of each modal measurement area. (a) Area 1; (b) Area 2.

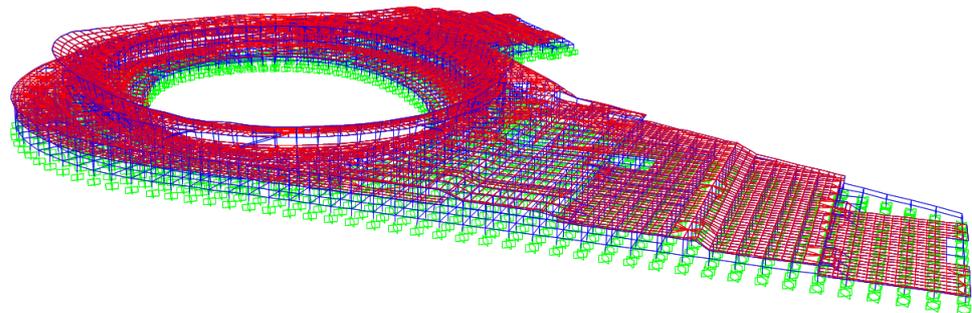
Taking the average value as the damping of the structure, Table 2 shows that structural damping is 1.22%.

Table 2. Damping test results.

Measuring Point	Damping Ratio (%)	Measuring Point	Damping Ratio (%)	Measuring Point	Damping Ratio (%)
1	0.73	9	1.69	17	0.98
2	1.25	10	2.92	18	1.23
3	0.83	11	1.42	19	0.75
4	1.59	12	1.63	20	1.40
5	0.79	13	0.96	21	1.41
6	1.94	14	1.40	22	0.98
7	0.29	15	1.74	23	0.83
8	1.31	16	1.17	-	-
average value	1.22				

3.2. Finite-Element Dynamic Characteristics Analysis

According to the drawings of the structure, SAP2000 was used to analyze the dynamic characteristics of the structural floor before and after vibration reduction. Calculation results were analyzed according to the three-dimensional spatial structure, and the structure was modeled using SAP2000 as shown in Figure 6. The used material properties were according to the specification [24], and the damping ratio of the structure was taken to be 1.22% (see in Table 2). According to Technical Standard for Vibration Comfort of Building Floor Structures JGJ/T441-2019 [13], the elastic modulus of reinforced concrete floor slabs was enlarged by 1.2 times. The equivalent uniform live load of people in the stands was 1.5 kN/m^2 . When performing structural modal analysis, the quality source was selected: 1 time (constant load) + 1 time (live load at the stand) + 0.5 times (live load in other areas). Since the grandstand structure was not fully completed during the test, there was no decoration surface, no seats, walkways, and low live load. Therefore, the load conditions of the original calculation model were modified according to actual conditions.

**Figure 6.** Structure model diagram.

Due to the complex structure, the natural vibration period of the structure is very dense. The 100-order mode shape was selected to meet the requirements of the specification for vertical mass parameter coefficients. Table 3 shows the period of the first 30 modes and their participating masses. Because the problem that affected the comfort of the structure was the cantilever part of the stand, by observing the shape of each order, the vibration mode with the larger vertical vibration of the 4th order cantilever part is selected. The natural frequencies were 3.056, 3.470, 3.747, and 4.215 Hz, and the local mode diagrams are shown in Figure 7.

Table 3. First 30 modes and their quality of participation.

Modal Number	Frequency (Hz)	Z-Direction Mode Quality of Participation (%)
1	1.854	6.80×10^{-5}
2	2.138	3.52×10^{-8}
3	2.192	2.15×10^{-7}
4	2.429	2.47×10^{-4}
5	2.598	1.12×10^{-3}
6	2.832	1.09×10^{-7}
7	2.873	2.32×10^{-4}
8 *	3.056	1.48×10^{-3}
9	3.084	3.19×10^{-5}
10	3.128	1.14×10^{-3}
11	3.272	5.18×10^{-4}
12	3.295	1.25×10^{-7}
13	3.369	5.07×10^{-8}
14	3.392	8.63×10^{-4}
15 *	3.470	1.14×10^{-3}
16	3.518	6.99×10^{-7}
17	3.573	6.00×10^{-4}
18	3.627	7.39×10^{-3}
19	3.653	8.38×10^{-7}
20 *	3.747	1.53×10^{-4}
21	3.857	2.65×10^{-4}
22	3.968	3.49×10^{-6}
23	4.034	3.28×10^{-7}
24	4.102	8.23×10^{-9}
25	4.119	2.74×10^{-6}
26	4.155	2.48×10^{-8}
27	4.165	2.54×10^{-4}
28 *	4.215	2.07×10^{-3}
29	4.229	5.04×10^{-5}
30	4.251	2.14×10^{-4}

* this is the order and frequency of interest.

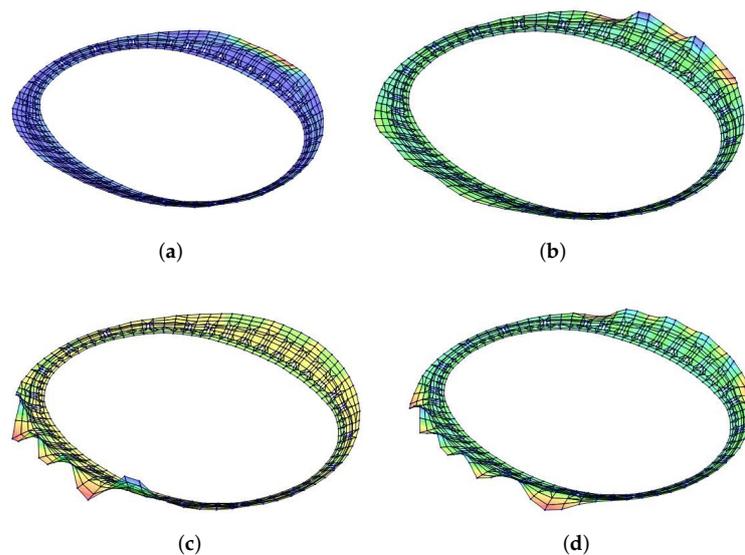


Figure 7. Local vibration diagram of cantilevered part: (a) 8th mode is 3.056 Hz; (b) 15th mode shape is 3.470 Hz; (c) 20th mode is 3.747 Hz; (d) 28th mode is 4.215 Hz.

Finite element software showed: the 8th order was the overall vibration in the middle part of measurement area 1; the 15th order was the wave-shaped local vibration at the outer

edge of measurement area 1; the 20th order was the wave-shaped local vibration at the edge of measurement area 2; in the 28th order, wave-shaped vibration occurred in both measurement areas.

We analyzed the modal assurance criteria (MAC) of the finite-element model-calculated mode shape to check the similarity of each order mode shape. Table 4 and Figure 8, show that, in the estimation of the mode vector of different modes, the MAC is close to 0, indicating that the two are mutually independent and orthogonal, and the modal identification results of each order were clear and reliable.

Table 4. Finite-element calculation of the MAC data of the first 4 main modes in the vertical direction.

Measurement Area	MAC Value	8	15	20	28
Area 1	8	1	0.00059	0.06989	0
	15	0.00059	1	0.22992	0.00009
	20	0.06989	0.22992	1	0.1723
	28	0	0.00009	0.1723	1
Area 2	8	1	0.22701	0.00001	0.03524
	15	0.22701	1	0.00732	0.00737
	20	0.00001	0.00732	1	0.01458
	28	0.03524	0.00737	0.01458	1

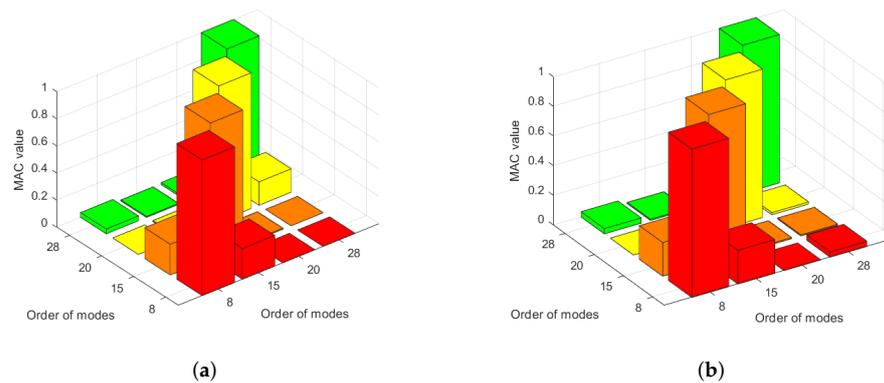


Figure 8. MAC of main vertical mode (a) Area 1; (b) Area 2.

3.3. Comparison and Analysis of Dynamic Characteristics

To examine the matching, the MAC value of actual measurement and finite element calculation was solved.

Figure 9 and Table 5 show that the first-order matching finite-element calculation of the measured modal in measurement area 1 is the 15th order, and the first-order matching finite element calculation of the measured modal in measurement area 2 is the 20th order. It can be considered that they correspond to each other.

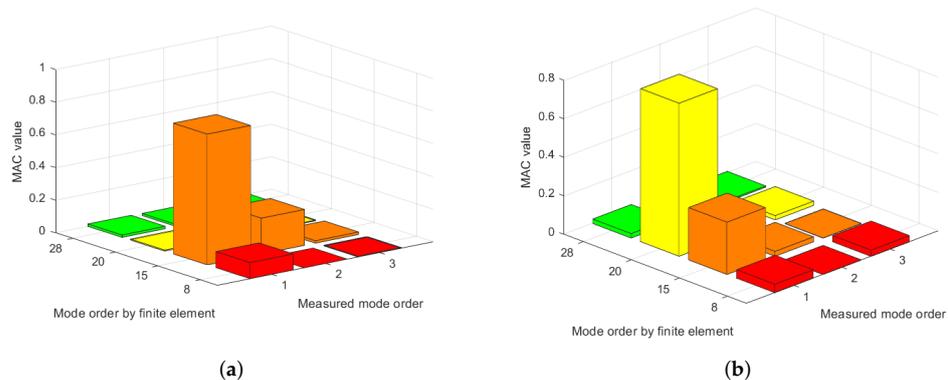


Figure 9. MAC histogram of measured mode and finite-element calculation mode. (a) Area 1; (b) Area 2.

Table 5. MAC of actual measurement and finite-element calculation.

Mode Order Calculated by Finite Element				
Area 1 (MAC)				
Measured mode order	8	15	20	28
1	0.0983	0.8005	0.0063	0.0145
2	0.0001	0.2196	0.0084	0.0124
3	0.0041	0.015	0.0042	0.0311
Area 2 (MAC)				
Measured mode order	8	15	20	28
1	0.0436	0.2717	0.7947	0.0231
2	0.0004	0.0256	0.0064	0.0222
3	0.0324	0.001	0.0223	0.0065

Table 6 shows that the maximal error between the theoretical and measured values of the mode shape was 7.47%, which was within a reasonable range. The measured model had higher stiffness, which may have been caused by an error between the load of the actual structure and the load of the calculation model, and there was a certain error between the measured and theoretical vertical frequencies. Weak rigid support at the outer edge of the cantilevered end of the structure had a certain impact on the dynamic characteristics of the structure (see Figure 10).

Table 6. Comparison of calculated and measured values of first-order vertical frequency.

Measurement Area	Frequency (Hz)		
	Finite-Element Calculation	Measured Value	Error
Area 1	3.470	3.750	7.47%
Area 2	3.747	3.950	5.14%



Figure 10. Radial tilt support around stands at the test site.

3.4. Crowd Load Response Test Results and Analysis

Table 7 shows the test results where the response was the greatest in each group of tests. Figure 11 shows the acceleration time history curve at the maximal response point of response measurement area 1 under the 1.5 Hz jump condition.

Test results showed that, under condition T1 (jumping), the maximal response was 374 mm/s²; under condition T2 (walking), the maximal response of the test area was 168 mm/s²; under condition T3 (walking), the maximal response of the testing area was 84.25 mm /s²; under condition T4 (quickstep), the maximal response of the test area was 279.15 mm/s².

Table 7. Statistical time-domain results of vertical vibration acceleration.

Condition Number	Test Group	Peak Value	Root Mean	Peak Value	Root Mean
		(m/s ²)	Square	(m/s ²)	Square
		Area 1		Area 2	
T1	Group 1	0.33400	0.06352	0.25267	0.04582
	Group 2	0.37400	0.05960	0.27900	0.06580
T2	Group 1	0.11379	0.01550	0.16800	0.02510
	Group 2	0.14000	0.02194	0.15900	0.02695
T3	Group 1	0.08000	0.02051	0.05133	0.01448
	Group 2	0.08425	0.01813	0.04800	0.01318
T4	Group 1	0.27915	0.03956	0.16235	0.02357
	Group 2	0.22733	0.03347	0.15112	0.02558

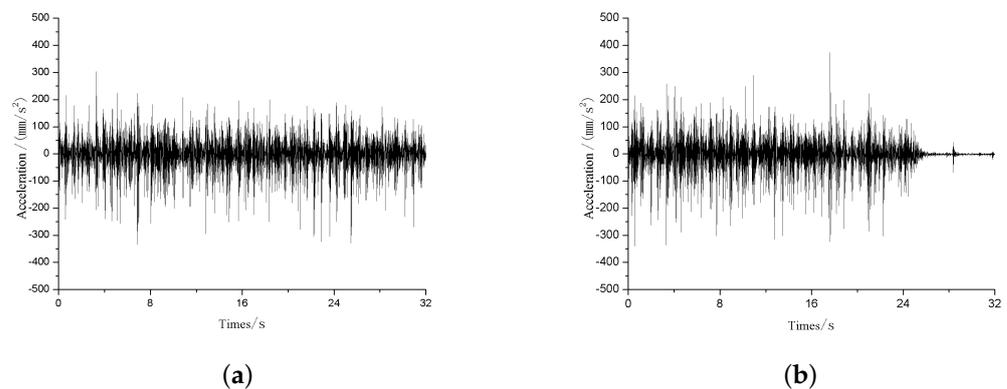


Figure 11. Area 1, maximal response under condition T1. (a) Test group 1; (b) Test group 2.

The theoretical value was consistent with the actual measured value. The position where the acceleration response is larger in the field test is similar to the maximum response position in the finite element calculation result, and the test value was selected for comparison with the finite-element calculation value of the response (Table 8).

Table 8. Theoretical value and maximal value of measured value of the response.

Area Number	Conditions	Vertical Vibration Acceleration		
		Theoretical Value of Original Structure (m/s ²)	Measured Value (m/s ²)	Difference
Area 1	T1	0.42800	0.37400	14.44%
	T2	0.13400	0.14000	−4.29%
	T3	0.06020	0.07425	−18.92%
	T4	0.31160	0.27915	11.62%
Area 2	T1	0.26290	0.27900	−5.77%
	T2	0.19710	0.16800	17.32%
	T3	0.04230	0.05133	−17.59%
	T4	0.19790	0.16235	21.90%

Table 8 shows that, except for the T3 working condition, the error between the theoretical and measured values was basically within ±15%. The possible reason is that the response value itself under T3 working conditions was relatively small, which led to a large percentage difference due to small changes. Considering the coupling effect of the crowd, the theoretical value of the crowd load response obtained by the finite-element calculation was reduced.

The reasons for the impact:

- auxiliary steel is not horizontal and has an inclination angle (Figure 10);
- bending stiffness has a certain effect on vibration;
- end circumferential wall also affects the vibration response of the structure.

4. Conclusions

In this paper, the dynamic characteristics and crowd load response of the stadium stands in Suzhou Industrial Park were tested on-site, combined with the numerical calculation of the corresponding finite-element model, and the following conclusions were obtained:

(1) The test results of the dynamic characteristics of the structure were consistent with the calculated values, indicating that the finite-element modeling of the structure was accurate and reasonable.

(2) The measured vertical fundamental frequency of structural modal test area 1 was 3.750 Hz, and that of modal test area 2 was 3.950 Hz, which was greater than the limit of 3 Hz given by Technical standard for human comfort of the floor vibration [7]. However, its second-order resonance frequency was still within the range of pedestrian load frequency, which may still cause vibration comfort problems. When the surface layer is added, the decoration is completed, and when the stadium holds an event, the constant and live load increase, especially the increase in the crowd load caused by the audience, and the vertical vibration frequency of the floor structure decreases. Therefore, from the perspective of vertical vibration frequency, the problem of vibration comfort of the structure should still be paid attention.

(3) Comparing the structure finite-element model and the obtained acceleration response peak with the test results showed that the presence of people increased the damping effect of the structure.

(4) Experience and data references for the platform construction of large stadiums should be provided in the future. In design and construction, the structural frequency should be avoided as far as possible from the pedestrian step frequency and the double frequency of the step frequency; the measured damping is small, and other methods should be used to increase the structural damping.

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