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Mechanical and acoustic emission characteristics of steel bar truss composite floor slabs under incrementally increasing cyclic loading -postprint

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Abstract

To investigate the mechanical behavior and damage characteristics of reinforced truss steel deck under fatigue loading, two identical reinforced truss steel deck specimens, one static loading condition, and one gradually increasing cyclic loading protocol were designed. Four-point bending tests on one-way slabs were conducted in conjunction with acoustic emission technology, and the influence of cyclic loading on the mechanical performance of the specimens and the corresponding characteristics of acoustic emission parameters were analyzed. The results show that, compared with static loading, the elastic working performance of the specimens under cyclic loading is essentially unaffected; the ultimate bearing capacity decreases by approximately 11%, but no obvious brittle failure characteristics are observed, reflecting the reliability of the steel deck. A phenomenon of coordinated variation in the strain of the truss rebars occurs during loading. As loading progresses, the acoustic emission ring-down counts exhibit a three-stage damage pattern in different periods. The RA-AF value distribution indicates the coexistence of tensile and shear cracks within the specimen, with tensile cracks being predominant, while the steel deck can effectively suppress concrete cracking at the failure stage.

Full Text

Mechanical and Acoustic Emission Characteristics of Steel-Bar Truss Floor Decks Under Gradually Increasing Amplitude Cyclic Loading

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Abstract

To investigate the mechanical properties and damage characteristics of steel-bar truss floor decks under fatigue loading, identical steel-bar truss floor deck specimens were designed with both static loading conditions and gradually increasing amplitude cyclic loading procedures. Four-point bending tests on one-way slabs were conducted using acoustic emission (AE) technology to analyze the influence of cyclic loading on mechanical performance and associated AE parameter characteristics. The results demonstrate that compared with static loading, the elastic working performance of specimens under cyclic loading remains essentially unaffected, while the ultimate bearing capacity decreases by approximately [value missing] without obvious brittle failure characteristics, reflecting the reliability of the floor deck. Truss reinforcement strains exhibit cooperative variation phenomena during loading. As loading progresses, AE ring-down counts show [value missing] distribution characteristics at different stages, indicating that both tensile and shear cracks coexist within the specimen with tensile cracks dominating, while the floor deck can effectively inhibit concrete cracking during the failure stage.

Keywords: steel-bar truss floor deck; increasing amplitude; fatigue; acoustic emission; damage characterization

1. Test Overview

With the advancement of urban modernization, China vigorously promotes standardized and industrialized construction processes. Prefabricated buildings, with their high-quality and convenient advantages, have gained attention in engineering projects worldwide. Research on prefabricated floor decks has primarily focused on theoretical calculations and static mechanical performance. However, in actual engineering applications, floor decks also experience dynamic and cyclic loading during long-term service, causing fatigue damage to accumulate internally and leading to performance degradation and reduced safety. The fatigue characteristics of building components are often emphasized in engineering design, yet research on the mechanical properties and cracking characteristics of steel-bar truss floor decks under fatigue loading remains limited.

Acoustic emission (AE) technology is a non-destructive testing technique that captures and analyzes transient strain energy released as elastic waves when concrete materials undergo compaction, cracking, and other processes under external excitation. With its real-time monitoring and non-destructive advantages, AE technology has been widely applied in mechanical testing research and structural detection of civil engineering materials in recent years.

This study designed a gradually increasing amplitude cyclic loading test for steel-bar truss floor decks to investigate their fatigue mechanical properties and

Figure 2

Figure 1: Figure 2

damage development characteristics. One-way floor deck specimens measuring 2000 mm × 600 mm were subjected to four-point bending fatigue tests while analyzing AE signal parameters.

1.1 Specimen Design

This study designed [number missing] identical steel-bar truss floor deck specimens cast as reinforced concrete one-way slabs, with [number missing] subjected to gradually increasing amplitude cyclic loading and [number missing] to static loading as control tests. The slab geometry was 2000 mm length × 600 mm width, with a concrete cover thickness of [value missing] mm and a casting thickness of [value missing] mm. The steel truss node spacing was 140 mm, with 200 mm spacing between upper chord reinforcement. Detailed dimensions and construction are shown in [FIGURE:1].

The casting used C30 concrete strength grade. During casting, 100 mm × 100 mm × 100 mm cube specimens were made from the same batch of concrete for compressive strength testing, with results shown in . Both upper and lower chord reinforcement used hot-rolled ribbed bars HRB400 with diameters of 8 mm and 10 mm respectively. Web reinforcement used CRB550 grade cold-rolled steel. The floor deck base formwork was 0.6 mm thick with 120 g/m^2 double-sided galvanization. Reinforcement details are shown in .

To measure deformation and stress in the reinforcement and steel plate, longitudinal displacement transducers were installed at the mid-span and loading point bottoms of the one-way slab. Strain gauges were attached to the bottom steel plate, truss web reinforcement, and lower chord reinforcement to measure the strain characteristics of the steel-bar truss floor deck. The measuring point layout is shown in [FIGURE:4] and [FIGURE:5].

2. Test Scheme

To study the fatigue characteristics of steel-bar truss floor decks, gradually increasing amplitude cyclic loading tests were conducted. The loading procedure involved multi-stage fatigue loading with different load amplitudes, where each stage had identical cycle counts and the load amplitude increased with stage number. The schematic diagram of the increasing amplitude cyclic loading process is shown in

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The specific test procedure was: first, preload the specimen to check instrument response; then, according to the designed load amplitude and cycle count for each stage, conduct loading and unloading. The loading/unloading rate for each

Figure 6

Figure 2: Figure 6

stage was 10 kN/min, with 1 minute hold time at each load amplitude. The specific loading scheme is shown in . The static loading test for the control group used the same 10 kN/min rate until specimen failure.

The loading method was four-point bending using a hydraulic jack applied to the distribution beam top. The layout is shown in [FIGURE:3]. Test data was automatically collected via computer. To capture AE signals during loading, six AE sensors were placed on the slab top and sides. The sensor layout is shown in

. The data acquisition system used a full-time structural health monitoring system produced by Physical Acoustics Corporation with a signal acquisition threshold of 40 dB. Before sensor installation, specimen surfaces were ground to ensure flat contact and uniform Vaseline application for optimal signal reception.

3.1 Test Phenomena

During loading, vertical through-cracks formed perpendicular to the length direction near the distribution beams, accompanied by concrete crushing. After completing the cyclic loading process, crack development patterns on the slab sides were observed. Post-loading, the mid-span region showed downward curvature in the floor deck steel plate, with partial detachment of bottom concrete but no separation from the steel plate. The slab top developed cracks extending to the distribution beam roots.

When the load amplitude reached 40 kN, the first noticeable crack appeared in the mid-span region, developing from the bottom along the thickness direction at an oblique angle. As loading progressed, the mid-span crack gradually widened and new micro-cracks developed in regions away from mid-span, showing a symmetric distribution trend toward both ends. At 90 kN, diagonal cracks developed from the left support toward the distribution beam root. When the load amplitude reached 120 kN, the specimen produced almost no new cracks, but existing crack widths increased dramatically with audible concrete crushing sounds. At 160 kN, the load could hardly increase further, indicating the steel had entered the yield stage. The specimen was held under load until no further phenomena occurred. The maximum mid-span deflection was 22.97 mm, with maximum residual deflection of 13.45 mm.

3.2 Load-Midspan Deflection Curves

The load-midspan deflection curve reflects the mechanical characteristics of the specimen. During cyclic loading, deflection at each cycle’ s load amplitude is called peak deflection, while deflection after unloading is residual deflection. The residual deflection showed no obvious growth trend during early cyclic loading, with phenomena primarily being concrete pore compaction, aggregate compression, and micro-crack development in tension zones, producing subtle sounds.

The load-deflection curve exhibited three-stage characteristics. In the early stage (corresponding to the initial deflection curve at low loads), residual deflection grew stably and slowly as the concrete interface gradually cracked along the specimen length, reducing cross-sectional area and decreasing stiffness. When the load reached 140 kN, the residual deflection growth rate increased dramatically with significantly larger slope. This occurred because the cast-in-place concrete had accumulated sufficient fatigue damage, and the floor deck reinforcement and steel plate began bearing primary loads with severe deformation. At 160 kN, the reinforcement and steel plate entered the yield stage, and the floor deck stiffness decreased sharply with accumulated concrete damage, leading to final failure.

Comparing cyclic and static curves, both showed linear characteristics before 140 kN, indicating elastic behavior, and nonlinear characteristics after 140 kN, predicting steel yield. However, the cyclic specimen lost mechanical performance after 140 kN, while the static specimen showed deflection increasing dramatically with slowly rising load. The static curve was smoother without obvious inflection points because, although concrete cracked in static loading, it shared forces with the floor deck through bond action. In cyclic loading, concrete experienced repeated tension-compression, developing more thorough damage, causing reinforcement and steel plate to gradually bear all tension and showing obvious yield characteristics. Thus, cyclic loading specimens failed earlier because stiffness degradation occurred earlier through accumulated fatigue damage.

3.3 Steel Plate Strain Characteristics

Peak and residual strains reflect steel mechanical properties. Measured peak strain, residual strain, and static loading strain at the mid-span bottom steel plate are shown in [FIGURE:9]. The peak strain first developed linearly with almost no residual strain, indicating elastic behavior with good strain energy recovery during unloading. When the load reached 140 kN, the curve inflected and both peak and residual strains developed rapidly, showing obvious yield characteristics.

The static loading steel plate strain curve showed a relatively smooth arc, significantly different from the peak strain curve. Unlike the slowly growing residual deflection, steel plate residual strain changed minimally before 140 kN because the specimen deformation trend was upward curling at both ends, with minimal bending in the mid-span steel plate. However, after 140 kN, the growth rate accelerated significantly as the lower chord reinforcement and bottom steel plate became primary tension members.

3.4 Reinforcement Strain Characteristics

Load-strain curves for each reinforcement member obtained from truss measuring points are shown in [FIGURE:10]. During low load periods, reinforcement strains developed linearly with extremely slow residual strain growth. The lower chord reinforcement, as the primary tension member, showed faster strain growth. Upper chord reinforcement in the concrete compression zone formed internal force balance with lower chord reinforcement during unloading at each stage, resulting in similar absolute residual strain values.

When the load reached 140 kN, the lower chord reinforcement and bottom steel plate became primary tension members, and the residual strain growth rate increased significantly. Near failure, lower chord reinforcement strain exceeded the steel yield strain (0.001942), entering the yield stage. Even after yielding, lower chord reinforcement strain continued growing with load, likely due to combined action with the steel plate enabling cooperative deformation of the floor deck.

3.5 Acoustic Emission Characteristics

3.5.1 Ring-Down Count Ring-down count is the count of signals or oscillations exceeding the acquisition threshold per unit time, reflecting AE signal activity. Cumulative ring-down count characterizes damage development rate, both closely related to crack development and damage characteristics. The relationship between load amplitude, ring-down count, and time during loading is shown in [FIGURE:11].

Ring-down count distribution showed three-stage characteristics similar to the deflection curve trend. Stage 1 corresponded to the early deflection curve at low loads, where ring-down counts showed few surge points and cumulative count grew slowly. Stage 2 corresponded to the mid-stage deflection curve before significant failure, where cumulative ring-down count grew at a stable rate with relatively uniform distribution, as the cast-in-place concrete continuously cracked in tension, reducing tension zone area while reinforcement and steel plate remained elastic.

Stage 3 corresponded to the late deflection curve after load reached 120 kN, where cumulative ring-down count experienced multiple cliff-like jumps. The initial cumulative value was 11\$$\times 10^{\{3\},reachingamaximumof35\times 10}$\{3\}\$ after loading, indicating the main period of severe specimen damage. The multiple dramatic increases in ring-down count indicated several intense internal failures during loading. In the early stage, ring-down count amplitude correlated with load amplitude, but in the later stage, the surge amount was much higher than in the early stage because the cast-in-place concrete had accumulated substantial fatigue damage. When load amplitude was reached again, most AE signals came from concrete crushing and further deformation after floor deck deflection rebound. Since concrete was mostly cracked and specimen stiffness severely degraded, no obvious damage occurred before reaching load amplitude, resulting in no significant growth in cumulative ring-down count between load amplitudes. Thus, the cumulative ring-down count curve showed obvious multi-level step characteristics.

3.5.2 RA-AF Values Concrete crack development involves both tension and shear mechanisms. Tension-induced cracks produce AE signals with short rise time and high frequency, while shear-induced signals typically show longer waveforms with longer rise time and lower frequency. The RA value (ratio of rise time to amplitude) and AF value (ratio of ring-down count to duration) can distinguish tensile and shear cracks. High AF and low RA correspond to tensile cracking, while low AF and high RA correspond to shear cracking.

This study set the crack type boundary ratio at 10:1 by referencing previous research. The distribution of RA and AF values during different loading periods is shown in [FIGURE:13]. Signals above the boundary line are tensile signals, while those below are shear signals. The damage characteristics of steel-bar truss floor deck specimens under cyclic loading show that: in early loading with low loads, AE signal sources were significantly fewer, with shear and tensile signals coexisting but tensile signals dominating. As load amplitude increased, concrete accumulated fatigue damage under cyclic loading, and shear signal proportion increased with growing shear and moment in both shear-bending segments. When the specimen entered the failure period, shear signal proportion increased significantly because previously cracked concrete reduced effective cross-sectional area, increasing internal shear stress. Meanwhile, the floor deck constrained excessive concrete deformation through its overall stiffness and bond action between steel truss and concrete, further inhibiting tensile signal generation.

4. Conclusions

This study conducted gradually increasing amplitude cyclic loading tests on steel-bar truss floor deck specimens, combined with AE technology to investigate mechanical properties under fatigue loading. The conclusions are:

1. Cyclic loading conditions have almost no effect on the elastic stage working performance of steel-bar truss floor decks, but cause decreases in ultimate bearing capacity and ductility loss. The reinforcement yields and exhibits cooperative strain variation due to the welded integrity of the floor deck.
2. The AE ring-down count of floor decks shows three-stage characteristics during loading: early-stage cumulative ring-down count grows linearly, representing stable damage accumulation; later-stage cumulative ring-down count shows multi-level step jumps, representing severe damage occurrence.
3. Analysis of RA and AF values reveals that tensile and shear cracks coexist within specimens, with tensile cracks dominating. The proportion of shear cracks increases with floor deck damage progression.

4. Under cyclic loading, the bottom steel plate is the primary tension-bearing component, entering the yield stage when load amplitude reaches 140 kN. The steel plate effectively inhibits concrete cracking during the failure stage.

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Figure 7

Figure 3: Figure 7

Figure 8

Figure 4: Figure 8

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Figures

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