Mechanical characteristics and comfort evaluation under pedestrian vertical excitation of a thin-walled cold-formed steel footbridge

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Abstract. Thin-walled cold-formed steel (TWCFS) structures have a great application prospect and are widely adopted in low-rise residential constructions due to great mechanical and economical properties. Since TWCFS member has low flexural rigidity and is hard to weld, it is rarely used in bridge engineering. In Poyang Lake Wetland of Jiangxi Province, China, a TWCFS footbridge was built. In this paper, Serial tests including static loading test, natural frequency test and pedestrian comfort test were conducted. At the same time, finite element analysis (FEA) models were built with Midas/Civil software. The data shows that although natural frequency is far greater than the range of pedestrian walking frequency and will not lead to the resonance of bridge, the peak acceleration under crowd load reflects poor pedestrian comfort. The structure was modified for improving pedestrian comfort and the results after modification verify that the comfort can be ensured.

1. Introduction
Thin-walled cold-formed steel (TWCFS) structural members are normally fabricated from structural steel sheets by cold-rolling and brake-pressing methods [1]. TWCFS structures are becoming increasingly popular in construction industry, either as a complete or partial structure [2]. There are many advantages such as high ratio of strength-to-weight, convenient construction and less pollution [2-3]. Due to excellent mechanical and economical properties, TWCFS members have seen wide applications in low-rise residual structures [4-5]. However, due to the thin-walled characteristics, the flexural rigidity of TWCFS members is worse than that of general hot-rolled steel members, which makes the application of TWCFS in bridge design very rare.

Aluminum alloy is another material used in lightweight structures, but the elastic modulus of aluminum alloy material is only 1/3 of that of steel [6], and the flexural rigidity of members is significantly worse than that of steel members with sections of the same size. Nevertheless, there are still some aluminum alloy bridges in the world. The first aluminum alloy bridge was built in 1946 in New York, USA [7]. Canada built the first long-span highway aluminum alloy bridge around the world with a span of 88.4 m in 1949 [8]. In 2006, Hangzhou, China, built a number of aluminum alloy footbridges represented by the overpass at intersection of Qingchun Road and Yan'an Road [7]. Compared with aluminum alloy, the application of TWCFS in bridge field is almost zero.

In Poyang Lake Wetland of Jiangxi Province, China, a 13-span continuous footbridge made of TWCFS was built, designed as a bird-watching platform. The main structure of each span of this TWCFS footbridge weighs about 600 kg and is 11.2 m in length. Among them, the total weight of
supporting members is about 186 kg, and the total weight of other members is 414 kg. Figure 1 shows some photos of this footbridge.

Figure 1. The TWCFS footbridge in poyang lake wetland.

Structures need to consider safety and serviceability at the same time. Safety refers to the ability to bear various possible loads and other actions, and serviceability refers to good performance under normal working conditions, which includes deflection, comfort and so on. For lightweight footbridges, even the safety requirements are met, people may feel uncomfortable due to excessive vibration when passing [9]. Xiebai footbridge, built in Hangzhou in 1993, was demolished in 2012 because of excessive vibration responses under pedestrian load [10]. The Millennium Bridge in London, which was built in 2001, was required to be stopped for use shortly after completion due to obvious vertical and lateral vibration. Some costly retrofitting had to be adopted to solve the vibration problems [11]. Therefore, the study of human-induced vibration is very necessary for the applicability of footbridges.

The problem of human-induced vibration of footbridges began to be of concerned in the 1970s. The national and regional codes mainly limit human-induced vibration by frequency adjustment method or combining it with dynamic response analysis method. The method of frequency adjustment is to avoid the possibility of resonance by adjusting the structure frequency not to fall into the range of pedestrian walking frequency. This method is simple and clear, adopted by the codes of most countries. The walking process of people consists of continuous steps, and the normal walking frequency is about 2 Hz [12]. In British bridge code BS5400 [13], the problem of human-induced vibration is not considered when the vertical natural frequency is greater than 5 Hz. The value reduces to 4 Hz in code OHBDC of Ontario, Canada [14]. Chinese code CJJ69-95 [15] requires that the natural frequency of the structure should be greater than 3 Hz.

In some codes, the dynamic response analysis method is adopted for footbridges which cannot meet the requirements of frequency adjustment method. This method evaluates the comfort of pedestrians directly by acceleration response, and the comfort limit value adopts peak acceleration or root mean square acceleration. However, there exists no necessity to analyze the acceleration response of the structure of which the natural frequency is beyond the range of pedestrian walking frequency. It is directly considered that avoiding resonance can meet the requirements of pedestrian comfort.

For the TWCFS footbridge located in Poyang Lake Wetland, some pedestrians reflected the lack of vertical comfort in use. Since each span of the structure is independent of each other, the static loading test and the natural frequency test were carried out based on a single-span structure, and the comfort response test was conducted at site. Finite element analysis (FEA) with Midas/civil software, which is a kind of FEA software and widely used in the field of bridge design in China was also used for studies during the process of tests.

2. Study of static and dynamic characteristics
Due to the lack of vertical comfort reflected by some pedestrians, tests for static and dynamic characteristics including the bearing capacity, deformation and natural frequency of the footbridge were conducted. According to Chinese code CJJ69-95, the ratio of deflection to span should be lower than 1/800; besides, the natural frequency shall be greater than 3 Hz. In the test, the independence of each span of the footbridge was considered, and the test was carried out based on a single-span structure.
2.1. Component composition and physical dimension

The TWCFS footbridge located in Poyang Lake Wetland is composed of 13 independent spans. During construction, each span structure shall be assembled firstly, and then transition trusses to realize the laying of additional wooden footpath boards connect the adjacent two spans. Each span is composed of one footpath truss and two handrail trusses on both sides. 12 connectors made of 3mm thick steel plate are used to enhance the connection strength of handrail truss and footpath truss. 8 among them are connected with the support rod at the same time. The other end of the support rod is fixed with the shallow foundation. The length of footpath truss of each span is 11.2 m, and the upper chord of each handrail truss with a length of 12.8 m is in contact with each other but not connected. Figure 2 and Table 1 show the component composition of a single-span and member section size.

2.2. Pre-study with FEA model

Before tests, analyses for static and dynamic characteristics were performed based on FEA model. In the model, only the TWCFS frame of the main structure was considered, and there was no additional wooden footpath board. Each component was rigidly connected at the position with bolts according to the design drawings. Referring to Chinese code CJJ69-95, the standard crowd load for bridges of which length is larger than 20 m is specified as:

\[ W = \left( 5 - 2 \times \left( \frac{L - 20}{80} \right) \right) \times \left( \frac{20 - B}{20} \right) \text{ (kPa)} \]  

(1)

Herein, \( W \) refers to the value of crowd load. \( L \) is the total length, while \( B \) is the bridge width. In formula (1), \( L \) should be taken as 100 m when the total length is greater than 100 m. According to the actual structure, \( L \) and \( B \) were taken as 100 m and 1.295 m respectively, and the calculated value of \( W \) was 2.8 kN/m², which was applied to double chords of footpath truss.

The structural static analysis showed that the maximum tensile stress was 34.6 MPa in the middle of upper chord, and the value of maximum compressive stress was 29.7 MPa in the middle of lower chord. Since the yield stress of Q235 steel is 235 MPa, all members have sufficient safety reserve. At the same time, the maximum deflection appeared at both ends of the structure with a value of 4.42 mm. Compared with the maximum limit value for the ratio of deflection to the span of 1/800 which is regulated in Chinese code CJJ69-95, that value of single-span model was 1/2534, which is completely satisfied.

The FEA model used subspace iteration method for modal analysis. The first-order frequency of the structure was calculated as 16.10Hz, which is much higher than the 3 Hz required by Chinese code CJJ69-95. At the same time, it completely avoids the high-order range of pedestrian step frequency, so resonance is almost impossible. Figure 3 shows the first four vibration modes.

2.3. Static and dynamic characteristic test

In static mechanical test, since the calculated crowd load was 2.8 kN/m², the total loaded weight was about 4 tons. Concrete blocks with an average weight of 75 kg were loaded in stages as shown in Table 2. Figure 4 shows the arrangement of strain gauges and displacement meters. Two displacement meters were arranged on both sides, and another was set in the middle.

As shown in Table 3, the stress and deflection under standard crowd load are compared with the value of former calculation with FEA model. In the test, the maximum tensile and compressive stresses were 37.70 MPa and 30.28 MPa, which appeared in the middle of upper chord and lower chord respectively. At the same time, the ratio of deflection to span was 1/2295 which is larger than 1/800 regulated in Chinese code CJJ69-95. The data obtained from test and Midas/Civil software were basically consistent, reflecting that the structure has a large reserve in bearing capacity and structural deformation.

For the actual multi-span structure, each span is connected to each other by transition truss. Thus, the free vibration of any single span is further limited, and it is safe to use the natural frequency of single-span structure to represent the lower limit natural frequency of the whole structure.
Figure 2. Component composition of one span footbridge.

Table 1. Section size of each member individually.

| Component       | Rod name        | Section size (mm) | Edge size (mm) | Member length (mm) |
|-----------------|-----------------|-------------------|----------------|--------------------|
| Footpath truss  | Chord           | 80×50×2           | 8              | 11200              |
|                 | Web member      | 74×50×2           | 17             |                    |
| Handrail truss  | Upper chord     | 80×50×2           | 8              | 12800              |
|                 | Lower Chord     | 80×50×2           | 8              | 11200              |
|                 | Web member      | 74×50×2           | 17             | 1475               |
| Support Bar     | --              | 75×4.5            | --             | 3000               |

Figure 3. The first four vibration modes of the single-span footbridge.

Table 2. Graded loading system.

| Load level          | Concrete block mass (t) | Equivalent load (kN) | Loading ratio (%) | Rest duration (min) |
|---------------------|--------------------------|-----------------------|-------------------|---------------------|
| Preloading          | 1                        | 10                    | 25%               | 5                   |
| First level loading | 2                        | 20                    | 50%               | 10                  |
| Second level loading| 2                        | 20                    | 100%              | 10                  |
Figure 4. Layout of measuring points in static and dynamic tests.

Table 3. Results of static characteristic research.

| Data source | Maximum tensile stress (MPa) | Maximum compressive stress (MPa) | Deflection (mm) | Deflection/Span |
|-------------|-----------------------------|----------------------------------|-----------------|-----------------|
| Midas/Civil | 34.60                       | 29.70                            | 4.42            | 1/2534          |
| Test        | 37.70                       | 30.28                            | 4.88            | 1/2295          |
| Difference (%) | +8.96                    | +1.95                            | +10.41          | -9.43           |

As shown in Figure 4, in the arrangement of acceleration sensors of single-span structure, four acceleration sensors were symmetrically arranged at both ends of the structure. Five workers took a rope connected to one end of the bridge and jumped continuously to fully vibrate the single-span structure.

For analyzing the frequency spectrum of measurement points, Fast Fourier Transform (FFT) method was applied to the transformation of the spectrum from the time domain to frequency domain. Figure 5 shows the frequency spectrum of measurement point at the end of bridge with the most obvious vibration as a representative.

The measured spectra of other 3 measurement points are similar to Figure 5, and it is clear that the first two frequencies are 16.65 and 22.75 Hz respectively. In addition, these data are basically consistent with the first two frequencies of 16.10 and 23.80 Hz calculated by FEA model, which both illustrates that resonance will not occur for the TWCFS footbridge.

Figure 5. Frequency spectrum of the measurement point at the end of bridge.

3. Study of acceleration response of multi-span structure

The studies based on single-span structure show that the bearing capacity and natural frequency of the TWCFS footbridge meet the requirements of Chinese code, and there will be no structural damage and resonance. However, in view of some uncomfortable phenomena in actual use, acceleration response is used as direct criteria to evaluate the pedestrian comfort of the footbridge.

3.1. Test design and results

The single-span structure with a span of 11.2 m can be divided into 4.8 m long supported section and 3.2 m long side span on both sides. In the multi-span structure, the adjacent two spans only connect the two footpath trusses through a transition truss. It is considered that each span is independent of each other, and the side span position of each span is the most sensitive position of the structure. The
test was carried out based on the multi-span structure at site, and two acceleration sensors were set at the position as shown in Figure 6 due to the restriction of test condition.

![Figure 6. Schematic diagram of acceleration measurement test.](image)

The footbridge is designed for scenic spot observation and photography, and is mainly loaded by pedestrians, and one walking cycle includes two steps. The landing of two feet on both sides corresponds to two peak walking forces, and the center of gravity is the lowest at this time. In the process of landing from the front foot to the back foot, the walking force reduces as the center of gravity is increased. With the increase of walking speed, the interval between the two peaks decreases until it overlaps in running state.

The actual walking process is random, and the walking frequency, peak value and phase vary from person to person. Random structural response cannot be directly superposed, and the maximum instantaneous step force of the total structure is less than the sum of peak values. According to the theory of random vibration, the dynamic response of \( N \) excitations with the same amplitude but the random phase is equal to \( (N)^{1/2} \) times of the input response of a single excitation to a linear structural system. Therefore, there is a conversion relationship between the number of random walkers and the number of consistent walkers as shown in formula (2). When the crowd density is large (more than 1 person/m\(^2\)), pedestrians cannot walk according to their wishes and habits. The structural response should be magnified by 1.85 times to obtain the conversion relationship as shown in formula (3) when the crowd density is greater than 1 person/m\(^2\) [16].

\[
N_p = (N)^{1/2} \tag{2}
\]

\[
N_p = 1.85(N)^{1/2} \tag{3}
\]

Among them, \( N_p \) is the number of pedestrians walking in consistent, while \( N \) is that of random pedestrians. The value of \( N \) can be calculated from \( N = S \times L \times B \), in which \( S \), \( L \) and \( B \) are the crowd density, the length and width of the bridge respectively.

In test, three walkers with similar weight and an average value of 60 kg walked across the bridge, and ensured that all pedestrians walked with the same phase. The measured walking speed is \( V = 1.4 \) m/s on average. Combined with the relationship \( V = 0.71f_p \) proposed by Aikaterini [17], the walking frequency \( f_p \) is approximately 2 Hz, which is the same as that proposed by Bachmann et al. [12]. Figure 7 shows the acceleration time history response, and Table 4 is the peak data.

There are many different codes for pedestrian comfort indicators. EN03 [18] in Germany divides comfort level by peak acceleration as shown in Table 5, which is widely used.

The relatively large value between the absolute value of the maximum acceleration and the minimum acceleration is selected, and the pedestrian comfort is evaluated with reference to Table 5. Under the condition of single pedestrian walking, the peak acceleration is 0.34 m/s\(^2\), which is at a very comfortable level. The peak accelerations of double and triple pedestrians walking are 0.69 and 0.87 m/s\(^2\) respectively, which are in the medium comfortable range, but the peak acceleration of triple pedestrians walking is close to the lower limit of uncomfortable level 1.00 m/s\(^2\). Thus, the comfort of the TWCFS footbridge under more pedestrian conditions remains to be studied.

3.2. Pedestrian load and FEA model

Based on the periodicity of pedestrian load, Fourier series pedestrian load model is widely used, which can be expressed by formula (4).
\[
F(t) = G + G \sum_{i=1}^{\infty} \alpha_i \sin(2 \pi f_i t - \phi_i)
\]  

(4)

**Table 4.** Measured peak acceleration.

| Number of pedestrians | Peak Acceleration at measuring point 1 (m/s²) | Peak Acceleration at measuring point 2 (m/s²) |
|-----------------------|-----------------------------------------------|-----------------------------------------------|
|                       | Maximum value | Minimum value | Maximum value | Minimum value |
| 1                     | 0.33          | -0.34         | 0.21          | -0.24         |
| 2                     | 0.67          | -0.69         | 0.50          | -0.48         |
| 3                     | 0.87          | -0.83         | 0.59          | -0.61         |

**Figure 7.** Measured acceleration time history.

**Table 5.** Comfort level specified in EN03 code [18].

| Grade | Comfort level | Vertical acceleration limit (m/s²) |
|-------|---------------|-----------------------------------|
| 1     | Very comfortable | <0.50                             |
| 2     | Medium comfortable | 0.50-1.00                       |
| 3     | Uncomfortable   | 1.00-2.50                        |
| 4     | Unbearable      | >2.50                             |

Among them, \(G\) is the weight of pedestrians, \(t\) is the time, and \(f_i\) is the walking frequency. \(\alpha_i\) is the \(i\)th-order dynamic load factor, which is the ratio of harmonic amplitude value to pedestrian weight, and the \(i\)th-order harmonic phase angle is represented by \(\phi_i\).

Bachman et al. [12] proposed that the first-order dynamic load factor was 0.37 at 2 Hz walking frequency. Young [19] thought that the first-order dynamic load factor at 2 Hz walking frequency was 0.43. IABSE [20] stated that the first-order dynamic load coefficient was 0.4, while \(\alpha_2\) and \(\alpha_3\) were 0.1 at the walking frequency of 2 Hz. In addition, \(\phi_1 = 0, \phi_2 = \phi_3 = \pi/2\).

According to the actual structure, a three-span model shown in Figure 8 was established on the basis of the single-span model. The number of pedestrians on the bridge deck was controlled to be 1-16 in sequence, and the pedestrians kept the same amplitude and phase to walk from left to right with space of 0.8 m between the front and rear pedestrians. The pedestrian weight \(G\) was taken as 700 N to simulate the acceleration response of the structure under pedestrians walking. Table 6 shows the calculation results, in which \(N_P\) is the number of pedestrians. The acceleration at the position of measurement point 1 in test was taken as the structural response, and \(a_{\text{max}}\) represented the peak value.

**Figure 8.** Three span FEA model.
Table 6. Maximum structural response of 1-16 pedestrians walking synchronously.

| N_p | \( a_{\text{max}} \) (m/s²) | N_p | \( a_{\text{max}} \) (m/s²) | N_p | \( a_{\text{max}} \) (m/s²) | N_p | \( a_{\text{max}} \) (m/s²) |
|-----|----------------------------|-----|----------------------------|-----|----------------------------|-----|----------------------------|
| 1   | 0.409                      | 2   | 0.808                      | 3   | 1.140                      | 4   | 1.462                      |
| 5   | 1.696                      | 6   | 1.828                      | 7   | 1.920                      | 8   | 1.902                      |
| 9   | 1.808                      | 10  | 1.689                      | 11  | 1.636                      | 12  | 1.714                      |
| 13  | 1.761                      | 14  | 1.779                      | 15  | 1.795                      | 16  | 1.783                      |

The results show that the maximum acceleration response value of the structure appeared when 7 pedestrians walked, and when the number of people was more than 9, the response value decreased gradually. Figure 9 shows the pedestrian position corresponding to the peak acceleration under various working conditions. When there were few pedestrians, the peak acceleration had a linear superposition relationship, which is similar to the measured acceleration response in test. It is reasonable that the peak acceleration calculated by the model is slightly larger than the measured results, considering that the absolute synchronous walking cannot be realized and the average weight of 60kg is lower than that in model. When the number of pedestrians increased to 7, the acceleration response at measurement point 1 continued to increase. All pedestrians were just full at the unsupported section when there were 11 pedestrians. When the number of pedestrians was more than 11, the peak acceleration response was stable between 1.7-1.8 m/s², reflecting that the pedestrian load in the supported section has no significant contribution to the response of the sensitive position of the side span of the structure.

The unsupported section is taken for independent analysis, which is composed of two side spans of 3.2 m long and 1.6 m long transition truss, with a total length of 8m. Under the condition corresponding to the most unfavorable load of the structure, all pedestrians walk in the unsupported section, therefore, \( L \) is taken as 8 m in the calculation of \( N \) in formulas (2) and (3). EN03 stipulates the largest crowd density is 1.5 person/m² [18], which is considered as a typical high-density crowd walking condition. According to formula (3), it is converted to 7.4 persons walking condition. In Figure 9, the interval between adjacent pedestrians is 0.8 m, which is more conservative than that of 7.4 people uniformly distributed along 8 m. Based on the calculation results of 7 and 8 pedestrians, the peak acceleration of 7 pedestrians walking at 0.8m interval is 1.920 m/s², corresponding to the uncomfortable interval in EN03 code. Under the density of 0.5 and 1.0 person/m², it is converted to 2.28 and 3.21 pedestrians walking uniformly and synchronously along the length 8 m according to formula (2). For safety, the acceleration response of 3 people and 4 people walking at 0.8m interval in Figure 9 is used to represent the random walking condition of 0.5 and 1.0 person/m². The corresponding maximum acceleration response is 1.140 and 1.462 m/s², which are slightly higher than the upper limit of the medium comfortable range and at an uncomfortable level.

Figure 9. Corresponding pedestrian position under peak acceleration of each working condition.

According to the comprehensive measurement of test and FEA, the natural frequency of the footbridge is beyond the range of walking frequency. Although the possibility of resonance is avoided, it is not enough to completely evade the uncomfortable phenomenon of pedestrians. The design of independent supports for each span is adopted, and the supported section is not sensitive to human-
induced vibration. However, the problem of insufficient vertical stiffness may exist in the section without support, resulting in the uncomfortable phenomenon of pedestrians.

4. Structural reinforcement
The control system represented by a tuned mass damper (TMD) is usually used when the pedestrian comfort is not satisfied for footbridges [21]. For the lightweight and simple TWCFS footbridge, TMD is not easy to arrange. Formula (5) is the forced vibration equation of damped multi-degree of freedom system.

\[
[M]\dddot{y}+[C]\dot{y}+[K]y = \{P\}
\] (5)

Under the premise of constant external load \( \{P\} \), since the mass matrix \( [M] \) and damping matrix \( [C] \) are not easy to change, the method of adding four support bars to the unsupported section as shown in Figure 10 was adopted for reinforcement to improve the vertical stiffness and reduce the pedestrian discomfort of structure.

IABSE load model was used to calculate the acceleration response of 1-16 pedestrians walking synchronously at 0.8 m spacing. The pedestrian weight \( G \) was taken as 700 N, and Table 7 shows the calculation results. \( N_p \) is the number of pedestrians walking, and the maximum acceleration response is represented by \( a_{max} \).

The relationship between the peak acceleration of the structure and the number of pedestrians before and after the support reinforcement is similar. It reached the peak value when 8 pedestrians walked on the reinforced supported section and reached the minimum value when 12 pedestrians were covered between the two supported sections. After that, the peak acceleration slightly increased with the increase of the number of pedestrians in the supported section. According to the formulas of conversion from random walking people to the consistent walking crowd, the peak acceleration of 3 people walking synchronously is 0.610 m/s², which represents the response of crowd density 0.5 person/m², and it is in the medium comfortable range according to EN03 code. The peak acceleration response of crowd density 1.0 and 1.5 person/m² corresponds to the acceleration of 3 and 8 people walking synchronously, and they are also at the medium comfortable level.

![Figure 10. Arrangement of reinforced support bars.](image)

| \( N_p \) | \( a_{max} \) (m/s²) | \( N_p \) | \( a_{max} \) (m/s²) | \( N_p \) | \( a_{max} \) (m/s²) | \( N_p \) | \( a_{max} \) (m/s²) |
|---|---|---|---|---|---|---|---|
| 1 | 0.236 | 2 | 0.439 | 3 | 0.610 | 4 | 0.763 |
| 5 | 0.874 | 6 | 0.936 | 7 | 0.956 | 8 | 0.961 |
| 9 | 0.926 | 10 | 0.881 | 11 | 0.850 | 12 | 0.837 |
| 13 | 0.843 | 14 | 0.858 | 15 | 0.872 | 16 | 0.882 |

5. Conclusions
Based on the static and dynamic tests of the single-span TWCFS footbridge, it was considered that the bridge has no problems of bearing capacity and possibility of resonance. Then, the acceleration response of 1-3 pedestrians walking synchronously was measured, and the comfort level was evaluated to be at a very comfortable or medium comfortable level. At the same time, the FEA model was built with Midas/Civil software. The calculation results in the single-span model were quite consistent with the experimental data, which provided a basis for
further acceleration simulation of multi-span footbridge. After that, the acceleration response analysis of 1-16 people walking synchronously with equal spacing of 0.8 m was carried out. According to the conversion relationship of pedestrian number between synchronous walking state and random walking state, the comfort levels of 0.5, 1.0 and 1.5 person/m² crowd density reached uncomfortable level.

In order to improve the pedestrian comfort on the basis of the original structure, the method of support reinforcement in the unsupported section was adopted. The acceleration response of 1-16 pedestrians walking synchronously is similar to that before reinforcement. However, the acceleration response of the structure under the crowd density of 0-1.5 person/m² reached a medium comfortable level, which shows the feasibility of the reinforcement scheme.

After the structural reinforcement, the structure forms a continuous support, which not only compressed the space under the bridge, but also was not conducive to the development of this structure. Then, the supports were changed to a continuous simple support structure, and the vertical bending stiffness of the structure was modified by changing the height of the handrail truss. The comparison of three kinds of handrail truss height showed that the vertical stiffness improvement was effective for the improvement of pedestrian comfort.

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