Research on vibration comfort of steel structure pedestrian bridges in slow moving systems

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Abstract. The current urban life needs a public, very attractive, leisure and entertainment space that can enjoy ecological services. Based on the above requirements, the slow traffic system bridge plays a role of "stringing beads into a chain" in the ecological park around the city. Its design complexity and construction difficulty are increasing, and its vibration comfort problem is becoming increasingly prominent. This paper analyzes the bridge load conditions of the slow traffic system, and summarizes the walking load and pedestrian comfort standards. The finite element model of a long-span steel structure slow travel system bridge in Chengdu was analyzed, and its vibration frequency was calculated to be 2.29Hz. In this paper, 8 sets of TMDs with a single weight of 0.5 tons are arranged in the middle span of the bridge overpass, and 6 sets of TMDs with a single weight of 0.5 tons are arranged on each side span, with a total mass of 10 tons. The model damping design shows that TMD can play a good damping effect, which can control the vibration caused by pedestrian load within the specification requirements, so that the maximum vertical acceleration under the most unfavorable working conditions is reduced from 65.3cm/s2 to 17.1 cm/s2, which is less than the 50 cm/s2 required by the comfort standard, and effectively improves the comfort of bridge use.

Keywords: slow moving system bridge; vibration frequency; comfort.

1 Introduction

In urban development, pedestrian overpasses are increasingly used for pedestrian crossing needs. These pedestrian overpasses usually have a large span, ranging from 40 to 50 meters, and the structural form is a single span simply supported beam. The structural beams and columns are mostly box beams or multi-layer trusses. Due to the similarity between the natural vibration frequency of the structure and the frequency of normal human walking, resonance is prone to occur, which affects the safety and normal use of the structure; At the same time, vibrations may exceed the limit of human comfort, which can easily cause psychological panic for pedestrians.

In modern design, due to functional requirements, there may sometimes be structures with large cantilevers or spans. The lack of constraints at the far end of the cantilevers

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results in lower frequencies of the cantilever structure. When the frequency of the cantilever structure is close to that of the pedestrian load, resonance may also occur. Long span structures, with a mid span position far from the constrained end, have high flexibility and low frequency, and are also prone to vibration problems.

In response to such vibration problems, TMD is currently widely used in the field of bridge engineering for vortex induced resonance and human induced vibration control. Xiang Haifan analyzed the optimal parameters and vibration reduction efficiency of TMD control for vortex induced resonance based on the Scanlan nonlinear vortex induced force model [1]. Gu Jinjun studied the control effect of TMD on the vortex vibration phenomenon of the H-shaped suspension rod of the Jiujiang Yangtze River Bridge [2]. Yu Mei studied the optimal parameters for controlling vortex induced vibration with TMD and found that when the mass ratio is 1%, TMD can significantly suppress vortex induced vibration [3]. Wang Libin and Li Xiaowei respectively applied TMD for human induced vibration control on the pedestrian bridge crossing Huaining Road in Hefei and the connecting bridge of a Chinese art museum [4,5] Deng Hartog derived the optimal frequency ratio and damping ratio parameters for STMD (Single Tuned Mass Dampers) without considering the damping of the controlled structure [6]. Rana used numerical methods to analyze the optimal parameters of TMD and pointed out that frequency ratio is the main factor affecting the vibration reduction efficiency of TMD. When designing TMD, if the damping of the controlled structure itself is not considered, its vibration reduction efficiency will be reduced [7].

2 Research objects and methods

2.1 Project overview

The research object is a steel structure bridge with a slow moving system in Chengdu. The bridge connection adopts a 38+56+38m variable height continuous steel truss bridge, and the approach bridge adopts an equal height continuous steel truss beam. The truss beam is in the form of an inverted triangular cross-section. The lower structure is all V-shaped pier steel piers, and the foundation is made of drilled cast-in-place piles. The total length of the project is 682.549m. A 150cm wide pedestrian railing is set on the bridge deck. The seismic fortification measures are classified as 8 degrees, and the seismic fortification of bridges is classified as Class C.

Pedestrian walking may generate vertical, lateral, and longitudinal forces. The lateral load is approximately 1/10-1/8 of the vertical load, and it will not cause resonance of connecting bridges, and is generally not considered in comfort analysis. According to the Technical Standards for Vibration Comfort of Building Floor Structures, high-density pedestrian areas such as walkways and bridges in buildings can be designed with reference to overpasses due to the high density of pedestrians and the possibility of synchronous walking.

The crowd density load excitation per unit area of the corridor and indoor overpass is calculated according to the following formula:

$$p_1(t) = P_b r' \psi \cos\left(2\pi \overline{f}_{s1} t\right) \tag{1}$$

$$p_2(t) = P_b r' \psi \cos\left(2\pi \overline{f}_{s2} t\right) \tag{2}$$

among them,

$p_1(t)$——The vertical load per unit area corresponding to the frequency of the first order vertical crowd load (kN/m2);

$p_2(t)$——The vertical load per unit area corresponding to the frequency of the second order vertical crowd load (kN/m2);

P_b——The vertical force generated by a single pedestrian walking on the corridor and indoor overpass can be taken as 0.28kN;

$\overline{f}_{s1}$——First order vertical crowd load frequency;

$\overline{f}_{s2}$——Second order vertical crowd load frequency;

$\overline{f}_{s1}$, $\overline{f}_{s2}$ can confirm as follows:

$$\overline{f}_{s1} = \begin{cases} 1.25 & \dfrac{f_1}{n} < 1.25 \\ \dfrac{f_1}{n} & 1.25 \le \dfrac{f_1}{n} \le 2.50 \\ 2.50 & \dfrac{f_1}{n} > 2.50 \end{cases} \tag{3}$$

$$\overline{f}_{s2} = 2\overline{f}_{s1} \tag{4}$$

The formula for calculating the equivalent population density is:

$$r' = \frac{10.8\sqrt{\xi N}}{A} \tag{5}$$

among them,

ξ——The structural damping ratio used in the comfort analysis is steel floor slabs, and the damping ratio value for walking conditions is 0.005;

N——The total number of pedestrians, with a value of;

m——Equivalent number of pedestrians

The vertical load reduction coefficient should be calculated according to the following equation:

$$\psi=\begin{cases} 0 & \bar{f}_{s1}\le 1.25 \\ \dfrac{\bar{f}_{s1}-1.25}{1.7-1.25} & 1.25<\bar{f}_{s1}\le 1.7 \\ 1 & 1.7<\bar{f}_{s1}\le 2.1 \\ 1-\dfrac{\bar{f}_{s1}-2.1}{2.3-2.1} & 2.1<\bar{f}_{s1}\le 2.25 \\ 0.25 & 2.25<\bar{f}_{s1}\le 2.5 \\ 0.25 & 2.5<\bar{f}_{s2}\le 4.2 \\ 0.25\left(1-\dfrac{\bar{f}_{s2}-4.2}{4.6-4.2}\right) & 4.2<\bar{f}_{s2}\le 4.6 \\ 0 & \bar{f}_{s2}>4.6 \end{cases} \tag{6}$$

2.2 Comfort evaluation criteria

Numerous studies and experiments have shown that human comfort and sensitivity can be evaluated using the vibration acceleration response of floors, and for stairs, acceleration evaluation can also be used. At present, there are many evaluation indicators for vibration acceleration, including peak acceleration, root mean square acceleration, weighted root mean square acceleration, weighted acceleration level, fourth power vibration dose level, etc. According to the Technical Standards for Vibration Comfort of Building Floor Structures, in this article, the peak acceleration is used as the evaluation index for walking excitation, and the vertical acceleration limit is taken as 50cm/s^2.

2.3 Bridge finite element model

Based on the overview of bridge engineering, establish a finite element analysis model for the bridge as shown in Figure 1:

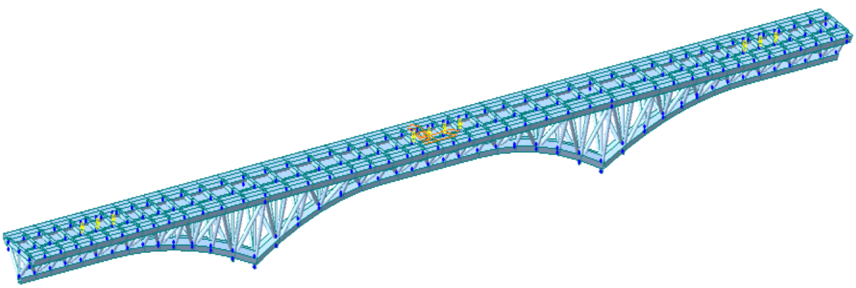

Fig. 1. Finite element model of bridge.

3 Simulation results and discussion

3.1 Modal analysis of undamped structures

Perform linear modal analysis on the overall model. The first 20 vibration mode periods and mass participation coefficients obtained are shown in Table 1. The vibration mode results show that the first vertical vibration frequency of the pedestrian bridge is 2.29Hz, with a participation mass of 1.1%. The vertical vibration mode of the overpass is concentrated in the third order, with a frequency of 4.89 and a participation mass coefficient of 66.9.

Table 1. Vibration modes and mass participation coefficients of non damping structures

Vibration mode	frequency	cycle	UX	UY	UZ	RX	RY	RZ
	Hz	s	%	%	%	%	%	%
1	2.293	0.436	0.1	0.0	1.1	0.0	0.0	0.0
2	3.821	0.262	0.3	0.0	0.0	0.0	50.5	0.0
3	4.893	0.204	0.7	0.2	66.9	0.0	0.0	0.0
4	5.096	0.196	0.0	12.3	0.3	7.2	0.0	0.4
5	6.184	0.162	0.0	2.5	0.0	20.4	0.0	0.9
6	7.255	0.138	2.2	0.1	0.0	0.0	22.4	2.1
7	7.383	0.135	0.2	4.5	0.0	1.9	1.3	17.9
8	7.436	0.134	0.0	15.7	0.0	26.5	0.1	2.0
9	8.267	0.121	65.3	0.0	0.7	0.0	0.2	0.0
10	8.740	0.114	0.0	19.6	0.0	6.6	0.0	25.8
11	9.077	0.110	0.0	26.8	0.0	7.2	0.0	19.9
12	10.719	0.093	0.1	0.0	0.4	0.0	0.0	0.0
13	10.740	0.093	0.0	0.0	0.0	0.0	0.0	1.3
14	11.265	0.089	0.0	0.0	0.0	0.0	0.1	0.0
15	12.486	0.080	0.7	0.0	9.7	0.0	0.0	0.0
16	13.577	0.074	0.0	0.0	0.0	0.0	0.0	7.6
17	14.741	0.068	0.0	0.0	0.0	0.2	0.0	0.0
18	14.782	0.068	0.0	0.0	0.0	0.0	0.0	0.1
19	15.661	0.064	0.0	0.0	0.0	8.0	0.0	0.0
20	17.262	0.058	0.9	0.0	0.0	0.0	0.6	0.0

The pedestrian walking frequency considered in Table 3. 1 is between 1.25 and 4.6 Hz, which can be biased towards conservative consideration of the impact of pedestrian load.

3.2 Calculation parameters

When conducting nonlinear time history analysis of the structure, pedestrian loads are applied to the floor elements according to the surface load, causing vibration of the bridge deck. For steel floors, the vibration mode damping ratio under walking excitation is taken as 0.005. The definition of time history working conditions is shown in Table 2, and the calculated time history working conditions are defined as follows. The area A obtained from the calculation of equivalent density is the single span area. The equivalent area of the corridor is approximately 500m^2.

Table 2. Definition of time history working conditions

Load case	Walking frequency	Area	Equivalent density	Vertical load
	Hz	m^2	Person/m2	Reduction factor
TC1	1.63	500	0.024149534	0.84
TC2	2.29	500	0.024149534	0.25
TC3	2.44	500	0.024149534	0.25
TC4	2.50	500	0.024149534	0.25
TC5	3.26	500	0.024149534	0.25
TC6	4.58	500	0.024149534	0.01

Apply the load uniformly to the nodes of the bridge in the form of nodal dynamic loads in finite element software, as shown in Figure 2. The nonlinear mode superposition method is used for time history analysis, with a dynamic analysis duration of 70s and a time increment of 0.01s. Adopting modal damping, all modal damping ratios are 0.005.

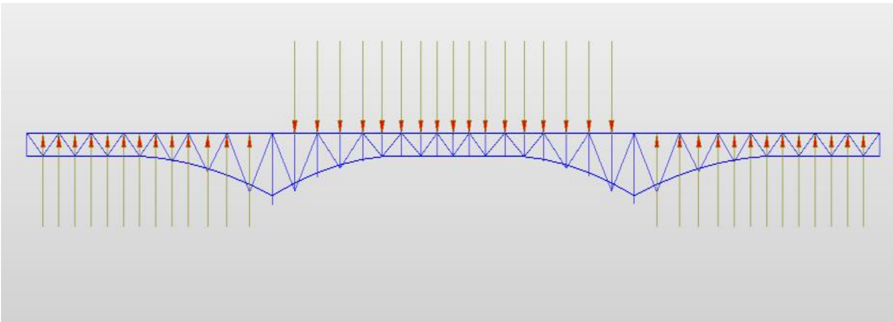

Fig. 2. Uniform arrangement of dynamic load application

3.3 Calculation results

Perform a time history analysis of the original structure under various operating conditions without TMD arrangement, and the results are shown in Table 3. From the results in the table, it can be seen that the maximum vertical acceleration of the pedestrian bridge under resonance conditions is 65.3cm/s^2, which exceeds the comfort standard.

Table 3. Dynamic response of undamped structures (cm/s^2)

Load case	Walking frequency	Before vibration reduction	
	Hz	Mid span	Side span
TC1	1.63	6.9	3.8
TC2	2.29	65.3	34.6
TC3	2.44	10.0	5.3
TC4	2.50	7.5	4.0
TC5	3.26	2.1	1.3
TC6	4.58	0.1	0.1

Based on the structural vibration mode characteristics, a preliminary arrangement of TMD is carried out, as shown in Figure 3. A total of 20 TMDs are arranged, with a single mass of 0.75 tons and a frequency of 2.29Hz. 8 sets are arranged for the mid span, and 6 sets are arranged for each of the two side spans. Although the acceleration results before vibration reduction show that the vertical acceleration in the mid span of the side span meets 50 cm/s^2, the first mode of vibration of the overpass and the three spans interact with each other. TMD can still control the vibration of the mid span and improve the comfort of the side span. Therefore, TMD is still installed in the side span of this project.

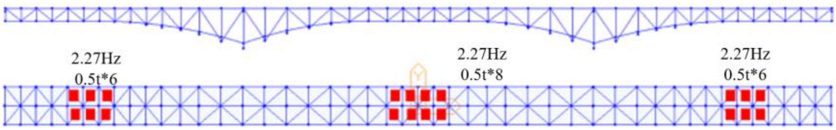

Fig. 3. Layout of TMD on bridge deck

The mass ratio is defined as the ratio of the mass of the TMD to the mass of the structural vibration mode. When the mass ratio of the TMD arranged to the mass of the vibration mode is between 1% and 5%, both the vibration reduction effect and the economy can be considered. In this project, the three span vibration directions in the first order vibration mode of the pedestrian bridge are staggered, resulting in mutual cancellation of the vibration mode mass participation coefficients. The final UZ vibration mode coefficient is only 1.1%, but the actual vibration is not small. Here, the overall mass of the structure is estimated, and the total mass of the pedestrian bridge in this project is 1089 tons. The TMD mass ratio calculated based on the total mass is 1%.

The TMD design parameters are shown in Table 4. The optimal frequency and damping coefficient design formula is as follows, where is the mass ratio.

$$f_{opt} = \frac{f_1}{1+\mu}, \xi_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}} \tag{7}$$

Table 4. TMD parameters

TMD parameters	
TMD mass/kg	500
Natural frequency/Hz	2.27
Spring stiffness/ kN/m	25.4x4
Spring stiffness	±15%
Damping ratio	0.06
Damping coefficient/ kN•s/m	0.86
Damping index	1.0
Damping index	20

Perform time-history dynamic analysis on the structure after TMD placement, and obtain its dynamic response as shown in Table 5. After vibration reduction, the vertical vibration acceleration can meet the comfort standard.

Table 5. Vertical acceleration after vibration reduction (cm/s^2)

Load case	Walking frequency	Before vibration reduction		After vibration reduction	
	Hz	Mid span	Side span	Mid span	Side span
TC1	1.63	6.9	3.8	6.6	4.4
TC2	2.29	65.3	34.6	7.1	3.7
TC3	2.44	10.0	5.3	17.1	9.2
TC4	2.50	7.5	4.0	9.7	5.2
TC5	3.26	2.1	1.3	2.1	1.4
TC6	4.58	0.1	0.1	0.1	0.1

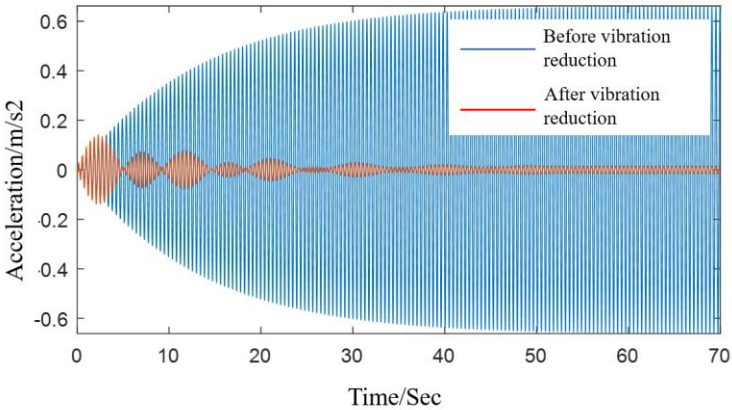

Fig. 4. Comparison of acceleration time histories before and after mid span vibration reduction under 2.29Hz walking conditions

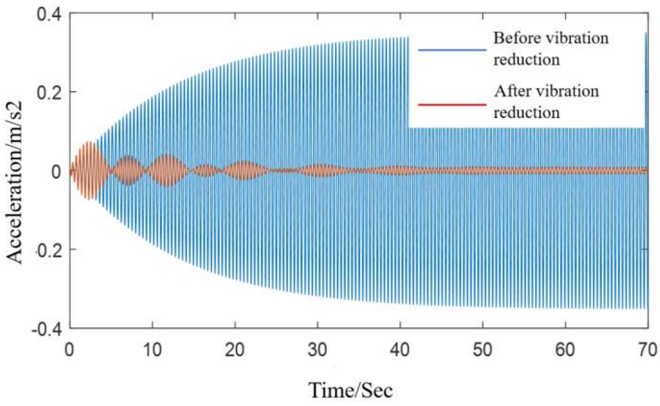

Fig. 5. Comparison of acceleration time histories before and after side span vibration reduction under 2.29Hz walking conditions

The comparison of vibration reduction effects between edge span and mid span under 2.29Hz walking conditions is shown in Figures 4 and 5.

4 Conclusions

(1) The vertical vibration frequency of pedestrian overpasses is 2.29Hz, which is close to the frequency range of 1.2-4.6Hz for pedestrian loads specified in the "Technical Standards for Building Floor Vibration Comfort". It may cause resonance and cause discomfort or even panic among pedestrians. Therefore, it is necessary to conduct comfort analysis and arrange TMD for vibration reduction control when necessary;

(2) Conduct dynamic time history analysis on the structure, and the results indicate that under resonance conditions, the vertical acceleration in the span of the corridor exceeds the comfort standard requirement of 50 cm/s^2, requiring vibration reduction design;

(3) 8 sets of TMDs with a single weight of 0.5 tons are arranged in the middle span of the overpass, and 6 sets of TMDs with a single weight of 0.5 tons are arranged on each side span, with a frequency of 2.27 Hz. The total mass of TMDs is 10 tons. The vibration reduction design of the model shows that TMD can play a good role in reducing vibration, and can control the vibration caused by pedestrian loads within the specified range.

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