Dependency of dynamic characteristics on environment temperature in cable-stayed bridge

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ABSTRACT: During the damage detection of structures, the methods based on structural dynamic characteristics, such as frequency and mode shape, are used most frequently. Accurate obtainment of structural dynamic characteristics is a challenging topic of structural healthy monitoring. Changes of environment temperature, due to season weather or radiation from sunshine, will lead to changes of characteristics of different members in cable-stayed bridge, which results in changes of dynamic characteristics of cable-stayed bridge. In this paper, effect of environment temperature on frequency and mode curvature of cable-stayed bridge is analyzed under season temperature difference and asymmetric temperature distribution, and these dynamic characteristics' changes due to temperature variation are compared with those due to damages of girder and cables in cable-stayed bridge.

1 INRODUCTION

Many techniques have been proposed to detect and localize structural damage using changes in dynamic characteristics of structures, such as natural frequency and mode shape. For the large structures, such as long-span bridges, damage detection is affected by environment factors. Significant damage may cause very small changes in dynamic characteristics, and dynamic characteristics' changes may go undetected due to changes in environmental and operational conditions. For long-span bridges, temperature differences may reach more than 50^0C due to season weather, and temperature differences in different part will reach 20^0C in one day due to radiation from sunshine. These changes have an influence on the dynamic characteristics of structures. Damage-induced dynamic characteristics' changes may be completely masked in changes due to environmental temperature.

Few researchers have addressed temperature effect on dynamic characteristics of bridges. Turner tested a three-span concrete foot-bridge which consisted of one central 16m and two 8m side spans, and results shows that the natural frequencies of this small bridge appeared unaffected by temperature changes [1]. Wood reported that the bridge responses were closely related to the structural temperature based on the vibration tests of five bridges in the United Kingdom [2]. Moorty and Roeder conducted a field test on the Sutton Creek Bridge in Montana, and the analytical analysis and the tests showed significant extensions of the bridge deck as temperature increased [3]. Hoon *et al* presented an adaptive filter that accommodated the changes in temperature to the damage of large-scale continued steel bridge [4]. Farrar *et al* attempted to develop system identification models that can separate the influences of temperature from damage on dynamic modal parameters [5]. Kim *et al* assessed variability of modal properties caused by temperature effects and adjusted modal data used for frequency-based damage detection in plate-girder bridges [6]. In this paper, frequency and mode curvature changes of cable-stayed bridge caused by average temperature changes and asymmetric temperature changes in different parts are studied, and these dynamic characteristics' changes are compared with those due to damages of girder and cables in cable-stayed bridge. For these dynamic characteristics' changes of cable-

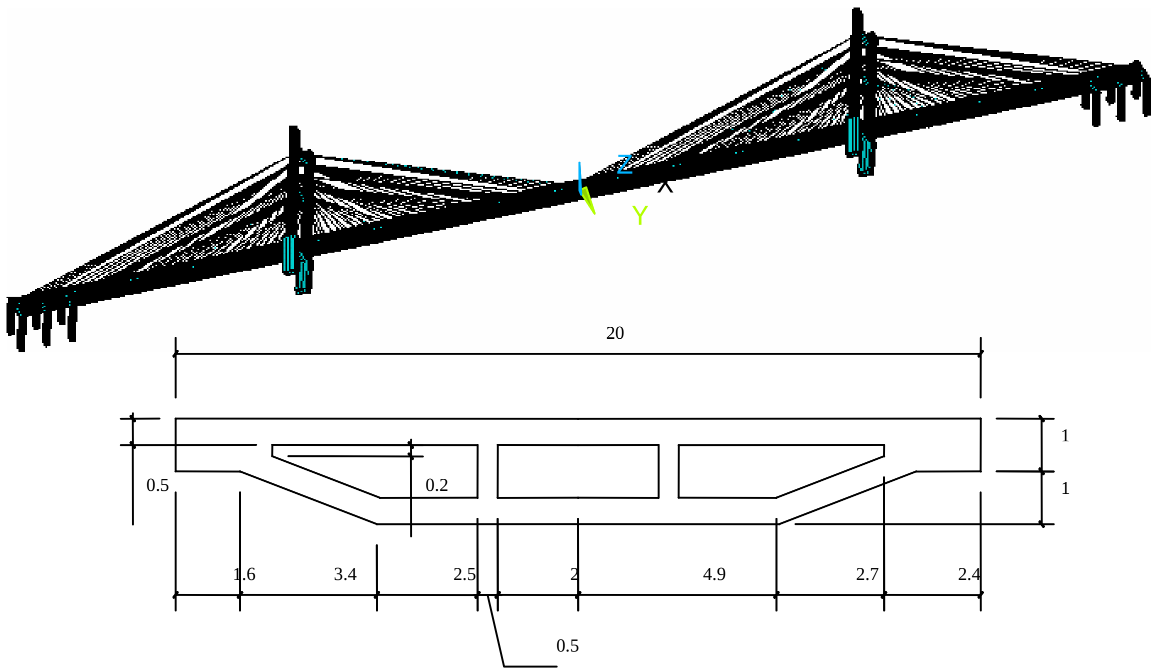


Fig.1 Schematic of cable-stayed bridge model

stayed bridge caused by temperature changes are analyzed based on the following several aspects: (1) changes of cable-stayed bridge's size and prestress in cables; (2) changes of elastic modulus of concrete; (3) changes of cables' sag effect. It can be shown from a numerical example that temperature variation will lead to visible frequency changes of cable-stayed bridge.

2 TEMPERATIRE'S EFFECT ON DYNAMIC CHARACTERISTICS OF CABLE-STAYED BRIDGE

Temperature's effect on dynamic characteristics of cable-stayed bridge is a complex topic. The mechanism on dynamic characteristics' changes of cable-stayed bridge under different temperature is delusive because of changes of many factors with varied temperature, especially for bridges with cracks. How to affect the appearance and extension of cracks by temperature is a challenge topic. In this paper, only cable-stayed bridges without cracks will be discussed.

The first factor of temperature's effect on dynamic characteristics of cable-stayed bridge is thermal effect of all parts, including cable, girder and tower. All members will change in size under different ambient temperature. For cable-stayed bridge, temperature's effect on cables is notable, cables will become longer and looser with increasing temperature. Here thermal effect will be considered by material thermal expansion coefficient α. In the following analysis, α_s =1.2 × 10^{-5} for cables and α_c =1.2×10^{-5} for concrete are adopted.

The second factor is changes of elastic modulus of concrete with the changing temperature. Under the ambient temperature, the relationship between elastic modulus of concrete and temperature can be written as follows [7]

$$E(T) = (100 - 0.0744T) \times 10^{-2} E(0) \tag{1}$$

where T is the calculated temperature, $E(T)$ and $E(0)$ are elastic modulus of concrete under the calculated temperature T ^{0}C and the referenced temperature 0^0C.

The other factor considered in this paper is temperature's effect on cables' sag effect. During analysis of cable-stayed bridges, nonlinear effect caused by self-weight of cables, namely cables' sag effect, usually must be considered. In the following analysis, the nonlinear effect is considered by Ernst (1965) formula [8].

$$E_{eq} = \frac{E_0}{1 + \dfrac{Aq^2 l^2 E_0}{12 S^3}} \tag{2}$$

Where E_{eq} is the equivalent elastic modulus of cables, E_0 is initial modulus of cables, A is the section area of cables, q is self-weight of cables per meter, l is the horizontal project length of cables, S is the tension force in cables. As known, prestress in cables is different under different temperature, i.e. the tension force in cables, S in equation (2) is different, thus the equivalent elastic modulus of cables, E_{eq} is different.

3 AVERAGE TEMPERATURE CHANGES

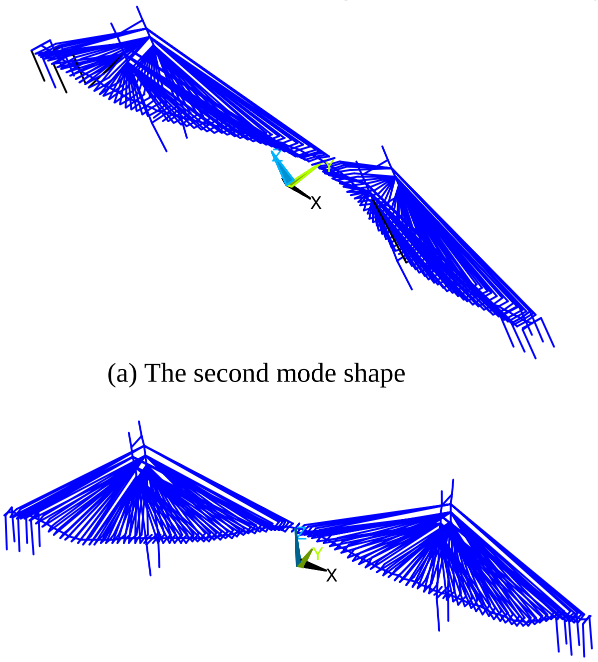
3.1 *3.1 Effect on frequency*

Changes of temperature are mainly caused by season weather or radiation from sunshine. Changes of temperature due to season weather are slow and average, which is usually supposed as average temperature difference.

In order to clarity dynamic characteristics' changes due to temperature difference, a numerical example on three-span (200m+400m+200m) cable-stayed bridge, as shown in Fig.1, is carried. The concrete box girder as shown in Fig.1 is adopted. The number of cables is 25×8, three kinds of section area are 0.010m^2, 0.012m^2 and 0.014m^2, and prestress in them are 420MPa, 500MPa and 480MPa, respectively. Isolation rubber bears are placed between towers and girders. Additionally, towers of the bridge are assumed to be attached to bedrock, and the effects of soil-structure interaction are neglected.

The reference temperature is set as 0^{0}C, three-dimension finite analysis about this bridge is carried. The finite element model employs beam elements, link elements, contact elements, spring elements and mass elements. The natural frequencies and the mode shapes of the cable-stayed bridge are determined after the static nonlinear analysis. The first mode of the bridge is float mode, the second mode is 1st symmetric lateral bending, as shown in Fig.2 (a), and the third mode is 1st symmetric vertical bending, as shown in Fig.2 (b).

When the environment temperature is changed to 40^{0}C and -20^{0}C, the natural frequencies of the bridge are solved. Firstly, only changes of cable-stayed bridge's size and prestress in cables are considered. Fig. 3 shows the first ten frequencies of the bridge under the three environment temperatures, and the detailed value can be seen in Table 1. It can be shown from Fig.3 and Table 1 that frequencies do not change obviously if only considering thermal size effect. The fifth and the eighth frequencies do not change under the three temperatures. Comparing



(a) The second mode shape

(b) The third mode shape

Fig.2 The main mode shapes of the bridge

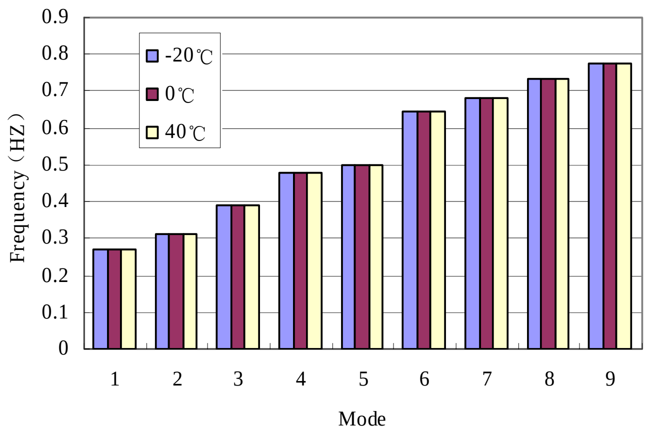


Fig. 3 Frequency change considering size thermal effect

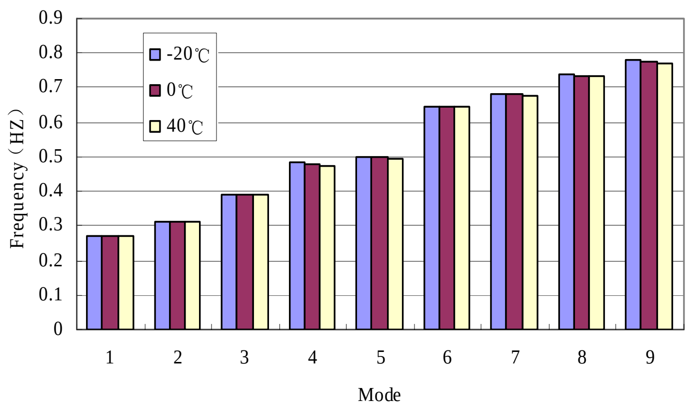


Fig. 4 Frequency change considering elastic modulus

frequencies under -20^{0}C and 0^{0}C the most obvious change happens the forth frequency, the forth frequency under -20^{0}C increases 0.0001HZ, the corresponding variation ratio is 0.03%. Also, under 40^{0}C the most obvious change happens the forth frequency, the frequency value decreases 0.0002HZ, the corresponding variation ratio is 0.06%. As we known, changes of ambient temperature will lead to changes of size of different parts in bridges, which will result in changes of prestress in cables. It is no doubt that these changes will result in changes of frequencies of cable-stayed bridges. While analysis results show that changes of frequencies due to size thermal effect are week.

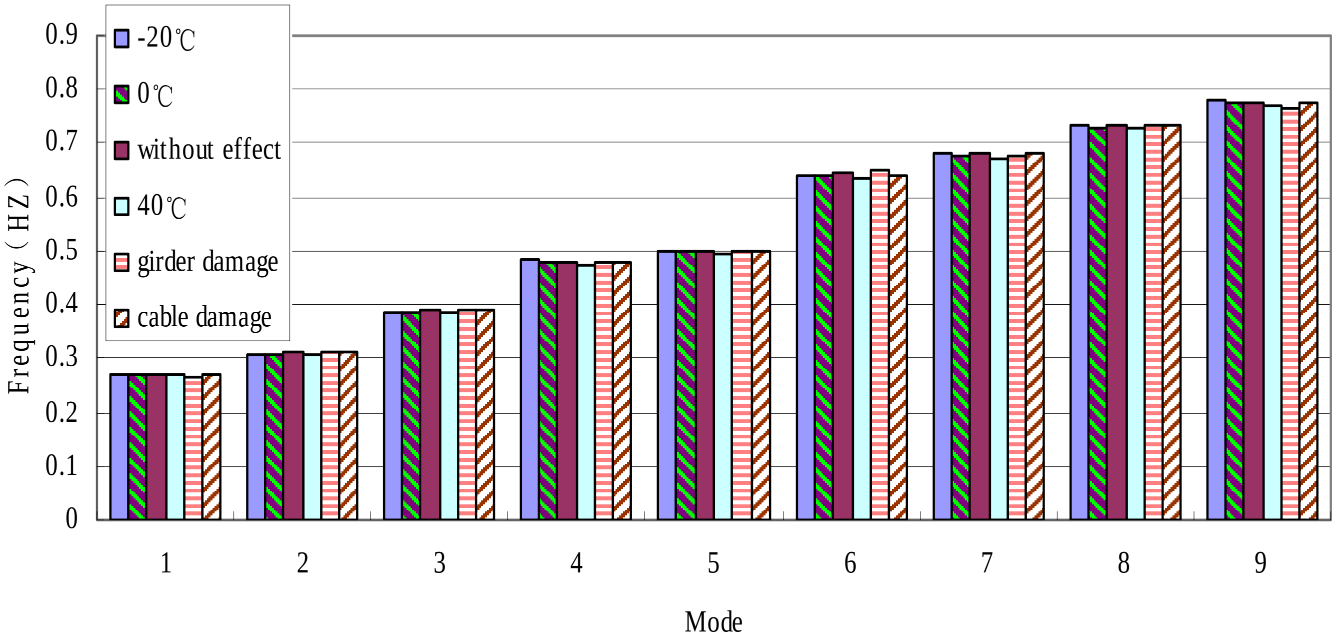


Fig. 5 Frequency comparison under different conditions

Table 1. Comparison of frequencies under the different conditions

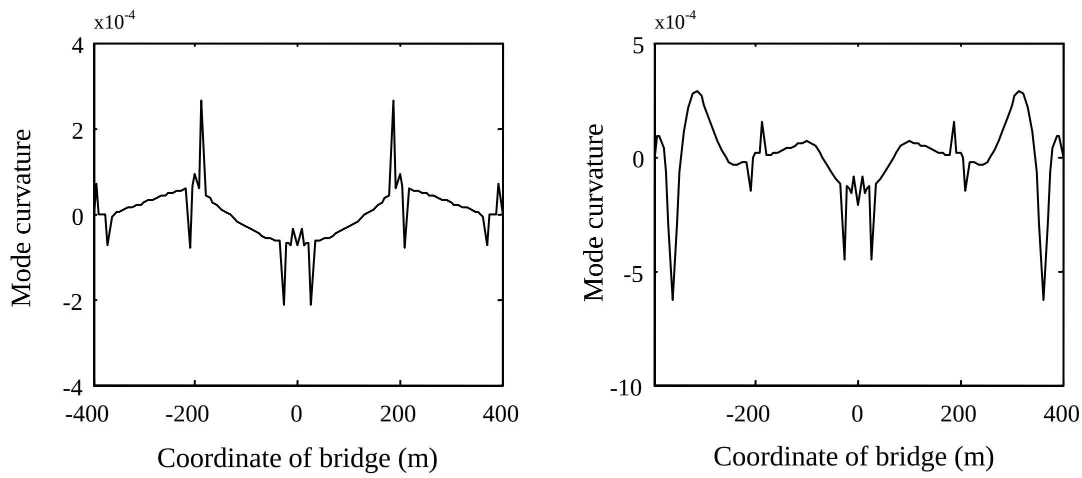
Mode No.	Mode Type	Without Effect	-20^{0}C (thermal size)	40^{0}C (thermal size)	-20^{0}C (elastic modulus)	40^{0}C (elastic modulus)	-20^{0}C (all effects)	0^{0}C (all effects)	40^{0}C (all effects)	Girder Damage	Cable Damage
1	float	0.092	0.092	0.092	0.092	0.091	0.092	0.092	0.091	0.092	0.092
2	1st lateral bending	0.2715	0.2715	0.2715	0.2728	0.2689	0.2728	0.2715	0.2689	0.2654	0.2715
3	1st vertical bending	0.3116	0.3116	0.3115	0.3119	0.3109	0.3072	0.3068	0.3061	0.3109	0.3111
4	2nd vertical bending	0.3910	0.3911	0.3908	0.3917	0.3896	0.3875	0.3867	0.3852	0.3897	0.3905
5	Total reverse sway	0.4802	0.4802	0.4802	0.4826	0.4754	0.4826	0.4802	0.4754	0.4801	0.4802
6	Total sway	0.4984	0.4984	0.4983	0.5008	0.4933	0.5008	0.4983	0.4933	0.4971	0.4983
7	3rd vertical bending	0.6460	0.6460	0.6458	0.6472	0.6435	0.6404	0.6391	0.6365	0.6409	0.6417
8	1st torsion	0.6793	0.6793	0.6793	0.6820	0.6738	0.6816	0.6789	0.6733	0.6761	0.6792
9	4th vertical bending	0.7353	0.7354	0.7351	0.7366	0.7327	0.7313	0.7300	0.7274	0.7341	0.7334
10	2nd lateral bending	0.7747	0.7748	0.7747	0.7783	0.7675	0.7784	0.7748	0.7674	0.7665	0.7747

When considering changes of elastic modulus due to temperature, elastic modulus can be calculated in accordance with equation (1) under different temperatures. Changes of frequencies due to elastic modulus can be seen in Fig.4 and Table 1. It can be seen that changes of frequencies due to elastic modulus' changes are more than those due to size thermal effect. Under -20^{0}C compared with 0^{0}C frequencies' change is between 0.001HZ and 0.004HZ, the corresponding variation ratio is 0.12%~0.5%. Under -40^{0}C compared with 0^{0}C frequencies' changes are between 0.001HZ and 0.008HZ, the corresponding variation ratios are between 0.3%~1.1%. Elastic modulus decreases with increasing temperature, thus frequencies of bridges will decrease.

Sag effect of cables will become obvious as the span of cable-stayed bridges increases. At the same time, sag effect of cables will become more obvious when cables are more far from tower. As known, temperature affects prestress in cables, and sag effect of cables is relative with prestress in cables. It is certainly that temperature will affect dynamic characteristics of cable-stayed bridges through sag effect of cables. Firstly, cable force should be calculated under different temperature. Secondly, the equivalent elastic modulus of cables can be got in accordance with equation (2). Then dynamic characteristics of the bridge can be got through revising finite element model. Fig.5 shows comparison of frequencies with and without considering all effects, including size thermal effect, elastic modulus' change and cables' sag effect. At the same time, in order to determine the effect of frequency changes on damage, frequency changes due to temperature variation are compared with those due to damages of girder or cables in cable-stayed bridge, as also seen in Fig.5. The detailed value can be seen in Table 1. Under 0^{0}C, frequencies considering cables' sag effect deing cables' sag effect, which is caused by decrease of cables' stiffness due to sag effect. The maximum frequency difference reaches 0.007HZ, and the maximum variation ratio is 1.6%. Obvious changes happen for the third, forth, seventh and ninth frequency. When considering all effects, comparing frequencies under -20^{0}C with those without considering temperature effect, the third, forth, seventh and ninth frequencies decrease, while the other frequencies increase. The maximum frequency difference reaches 0.006HZ, and the maximum variation ratio is 1.5%. Comparing frequencies under 40^{0}C with those without considering temperature effect, all frequencies decrease. The maximum frequency difference reaches 0.01HZ, and the maximum variation ratio is 1.8%. It can be found that temperature's effect on frequencies corresponding to bending mode type is more obvious, especially for vertical bending mode type. When two damages are happened in the girder of cable-stayed bridge (element 38, -284m~-276m, element 2, -13m~-8m, and 70% reduction in stiffness for each element), the maximum frequency difference reaches 0.009HZ, and the maximum variation ratio is 2.3%. When four damages are happened in the cables of cable-stayed bridge (element 247, -276m, element 253, -140m, element 185, -13m, element 541, 116m and 50% reduction in stiffness for each element), the maximum frequency difference reaches 0.005HZ, and the maximum variation ratio is 0.7%. As seen, frequency changes due to damages of girder are as almost as those due to temperature variation, and frequency changes due to damages of cables are smaller than those due to temperature variation. Obviously, frequency changes due to damages submerge in those due to temperature variation, especially for cables' damage.

3.2 *Effect on mode curvature*

Mode curvature is one of dynamic characteristics



(a) The second mode

(b) The third mode

Fig. 6 Mode curvature of bridge without considering temperature based on mode shape, which is usually used in structural dynamic damage detection. For girder in cable-stayed bridge, the mode curvature v_i'' in the position i can be written as

$$v_i'' = \frac{v_{i+1} - 2v_i + v_{i-1}}{h^2} \qquad (3)$$

where v_{i-1}, v_i and v_{i+1} are the offset in the position $i-1$, i and $i+1$, respectively, h is the distance between the adjacent measured positions. Fig. 6 is mode curvature of the cable-stayed bridge without considering temperature. For the second mode and the third mode, higher mode curvature value happens in the mid-spans and supports of the bridge.

under different temperature is calculated compared with the mode curvature without considering temperature. Fig.7 shows comparison of the mode curvature difference under -20^{0}C and 40^{0}C. The mode curvature difference of the second mode reaches 1.6 $\times 10^{-6}$, about 0.8% of the initial maximum mode curvature. The mode curvature difference of the third mode reaches 1.8$\times 10^{-6}$, about 0.3% of the initial maximum mode curvature. It can be concluded that changes of mode curvature due to size thermal effect are weak. It can be seen that more obvious mode curvature changes happen in mid-span, 1/4 span of the central span, and mid-span of the side span in the cable-stayed bridge.

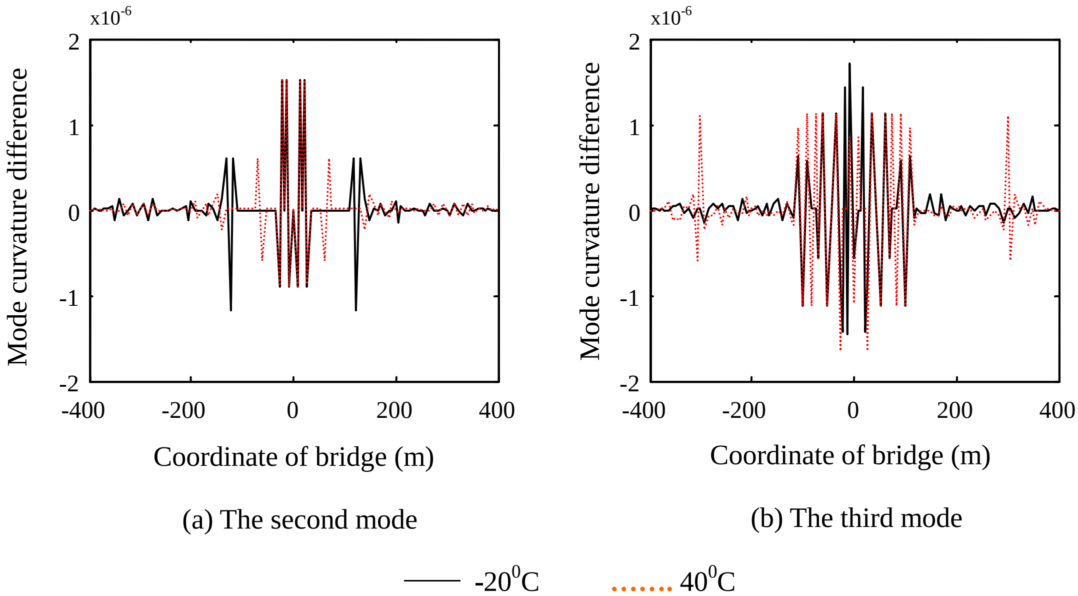


Fig. 7 Comparison of mode curvature difference considering size thermal effect

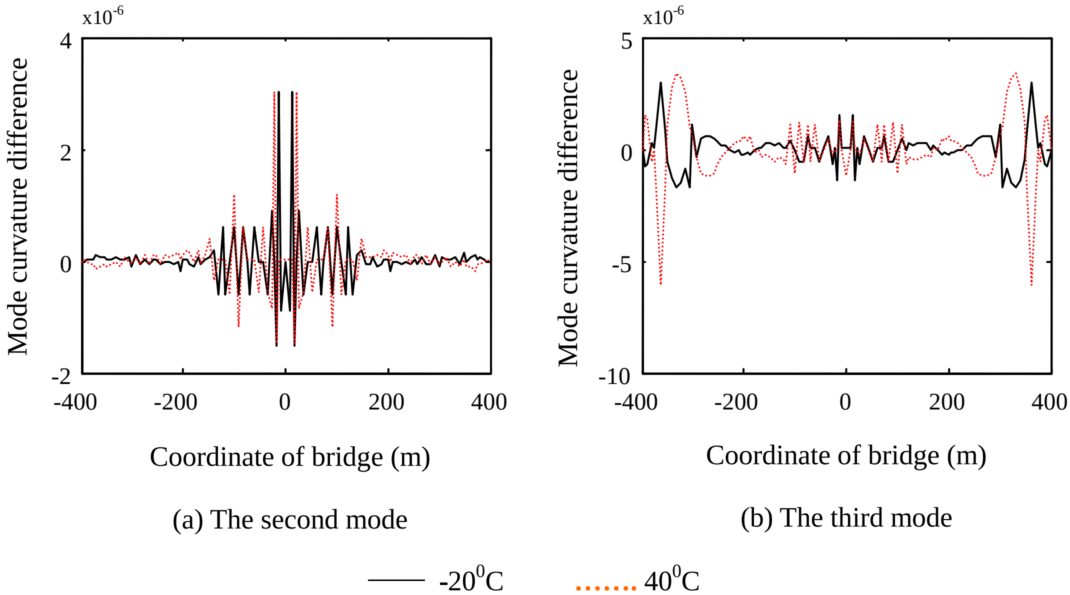


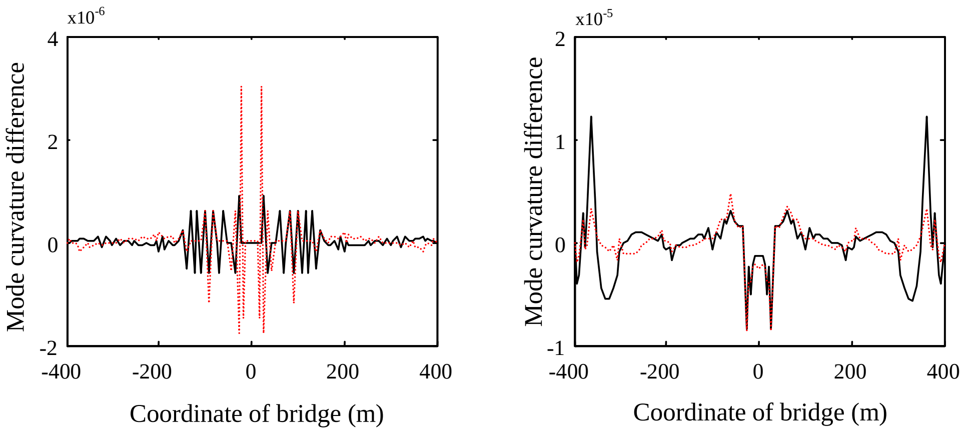
Fig. 8 Comparison of mode curvature difference considering elastic modulus

Similarly, when considering size thermal effect, the mode curvature of the girder in cable-stayed bridge is calculated. In order to display changes of the mode curvature, the mode curvature difference

When considering changes of elastic modulus due to temperature, mode curvature difference under -20^{0}C and 40^{0}C is plotted in Fig.8. The maximum mode curvature difference happens in mid-span and

1/4 span of the central span, and mid-span of the side span for the two modes. The maximum mode curvature difference of the second mode reaches 3.6 $\times 10^{-6}$, about 1.8% of the initial maximum mode curvature. The maximum mode curvature difference of the third mode reaches 6.3×10^{-6}, about 1.1% of the initial maximum mode curvature.

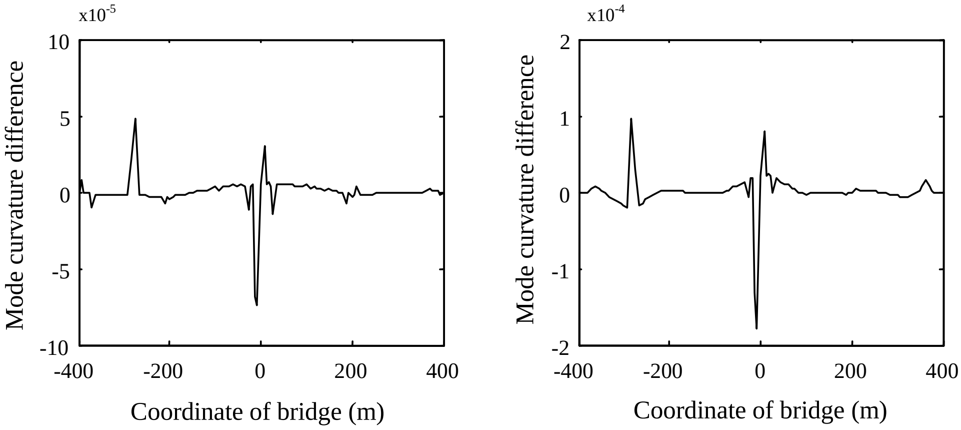
When considering all three temperature factors including size thermal effect, elastic modulus change, and cables' sag effect. Fig.9 shows mode curvature difference comparison considering all effects between -20^{0}C and 40^{0}C. The maximum mode curvature difference happens in mid-span and 1/4 span of the central span, and mid-span of the side



(a) The second mode (b) The third mode

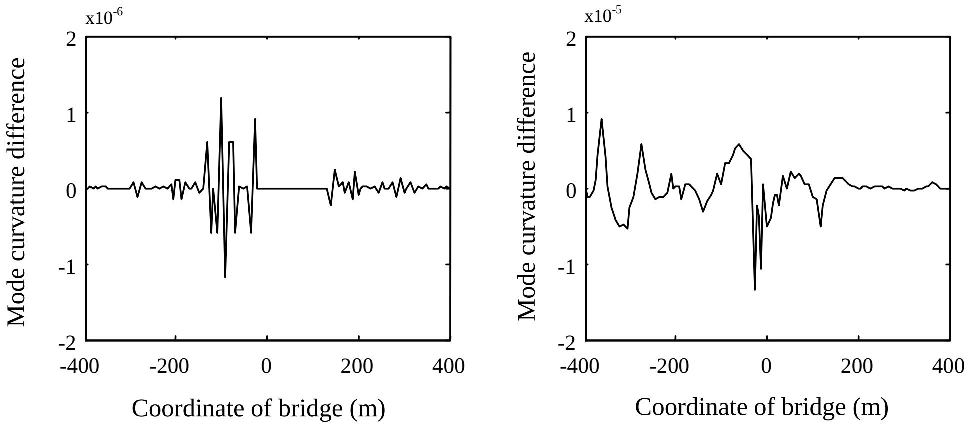
—— -20^{0}C .... 40^{0}C

Fig. 9 Comparison of mode curvature difference considering all effects



(a) The second mode (b) The third mode

Fig. 10 Mode curvature difference subject to girder damage



(a) The second mode (b) The third mode

Fig. 11 Mode curvature difference subject to cables damage span for the two modes. The maximum mode curvature difference of the second mode reaches 3.6×10^{-6}, about 1.8% of the initial the maximum mode curvature. The maximum mode curvature difference of the third mode reaches 1.2×10^{-5}, about 6% of the initial the maximum mode curvature. Numerical results shows that turn of the first ten mode shapes is not changed with ambient temperature, though the frequency and the mode curvature change.

In order to compare changes of mode curvature due to temperature to changes of mode curvature due to damage, mode curvature differences due to girder damage (element 38, -284m~-276m, element 2, -13m~-8m, and 70% reduction in stiffness for each element) and cables damage (element 247, -276m, element 253, -140m, element 185, -13m, element 541, 116m and 50% reduction in stiffness for each element) are plotted in Fig.10 and Fig.11, respectively. Results show that the maximum mode curvature difference of the second mode reaches 7.4×10^{-5} for the girder damage, about 37% of the initial the maximum mode curvature. The maximum mode curvature difference of the third mode reaches 1.8×10^{-4} for the girder damage, about 30% of the initial the maximum mode curvature. Girder damage will lead to obvious mode curvature changes, moreover, obvious mode curvature changes happen in positions of girder damage. Both for the second mode and for the third mode, for the girder damage positions (-284m~-276m and -13m~-8m), obvious mode curvature changes happen. The maximum mode curvature difference of the second mode reaches 1.2×10^{-6} for the cables damage, about 0.6% of the initial the maximum mode curvature. The maximum mode curvature difference of the third mode reaches 1.2×10^{-5} for the cables damage, about 2% of the initial the maximum mode curvature. As seen, changes of mode curvature due to cables damage submerge in changes of mode curvature due to temperature. Changes of mode curvature for the second mode can show cables damage position, while it is very difficulty to show cables damage position for the third mode curvature changes.

4 ASYMMETRIC TEMPERATURE CHANGES

4.1 *Effect on frequency*

Temperature difference caused by seasonal weather can be considered average temperature variation, while temperature of different parts will be different due to radiation of sunshine during one day, especially in summer. Usually, temperature in cables is highest, up-side of girder and tower take the second place, and temperature in down-side of girder and pier is lowest. In the following, different temperature distribution’s effect on dynamic characteristics of cable-stayed bridges will be analyzed.

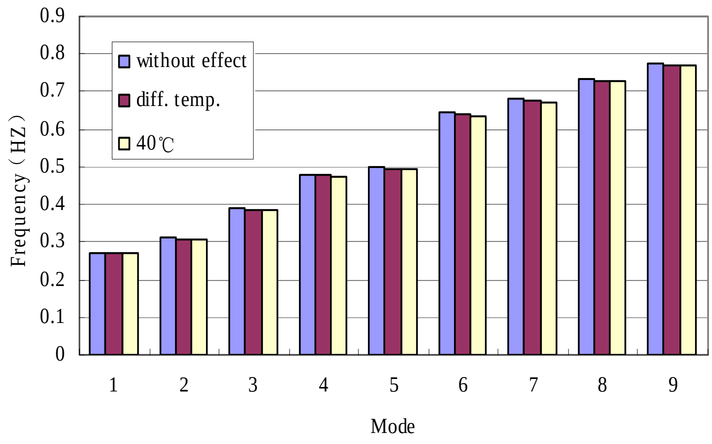
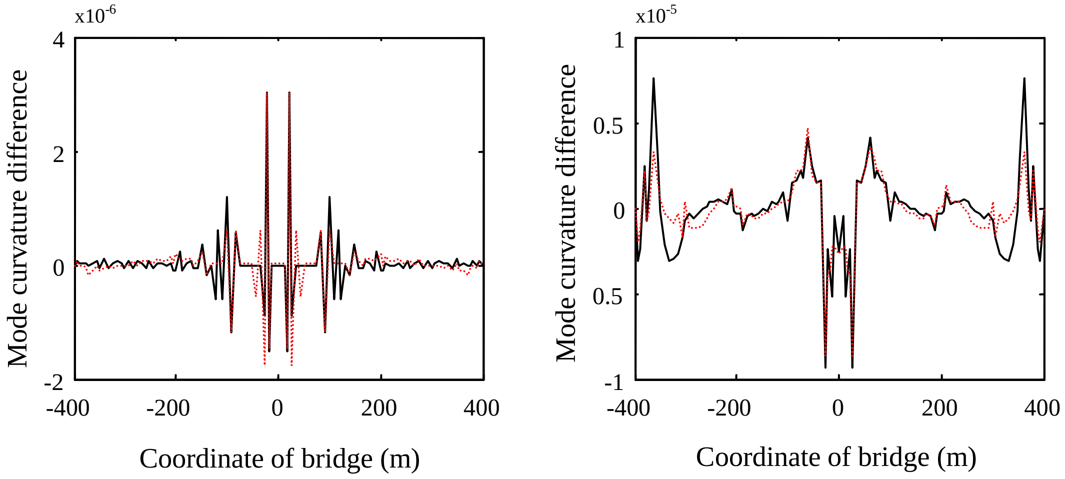


Fig. 12 Frequency comparison under different conditions

In the finite element analysis, supposed that temperature of cables is 40^0C, 35^0C for towers, 30^0C for girder, and 25^0C for piers. The described three temperature factors afore will all be considered. Fig.12 shows comparison of frequencies between considering different temperature distribution, 40^0C average temperature distribution and without considering temperature effect. The first tenth frequencies considering different temperature distribution are 0.092HZ, 0.2707HZ, 0.3065HZ, 0.3861HZ, 0.4782HZ, 0.4963HZ, 0.6382HZ, 0.6771HZ, 0.7292HZ, and 0.7725HZ. Compared with those without considering temperature effect, frequencies decrease. It can be found the major frequency changes still happen on the third, forth, seventh and ninth mode shape, i.e. vertical bending mode shape. The maximum frequency difference reaches 0.008HZ, and the maximum variation ratio is 1.7%. Compared with frequencies under 40^0C average temperature distribution, frequencies under asymmetric temperature distribution increase, the maximum frequency difference reaches 0.006HZ, and the maximum variation ratio is 0.7%. Frequencies variation laws caused by asymmetric temperature distribution are similar with those caused by average temperature change. Frequency variation ratio caused by asymmetric temperature distribution approaches to 1% compared with results of average temperature change.

4.2 *Effect on mode curvature*

Changes of mode curvature under asymmetric temperature distribution and 40^0C average temperature distribution are analyzed. Fig. 13 shows comparison of mode curvature difference between asymmetric temperature distribution and 40^{0}C aver-



(a) The second mode

(b) The third mode

—— asymmetric temperature distribution .... 40^{0}C

Fig. 13 Comparison of mode curvature difference under different temperature

age temperature distribution are analyzed. It can be seen that changes of mode curvature under two temperature distributions are similar for both the second mode and the third mode. Changes of mode curvature due to asymmetric temperature distribution reaches 3.4×10^{-6} for the second mode, about 1.7% of the initial the maximum mode curvature. Changes of mode curvature due to asymmetric temperature distribution reaches 9.8×10^{-6} for the second mode, about 1.6% of the initial the maximum mode curvature.

5 CONCLUSIONS

Ambient temperature is an important factors affected dynamic characteristics of bridges. In this paper, through numerical analysis on changes of frequency and mode curvature in cable-stayed bridge caused by temperature difference due to season weather and sunshine radiation, some conclusions can be got.

(1) Frequencies' variation ratio due to temperature will approach to 2%. Temperature's effect on frequencies corresponding to bending mode type is more obvious, especially for vertical bending mode type.

(2) Changes of mode curvature due to ambient temperature usually happen in mid-span and 1/4 span of the central span, mid-span of the side span, and supports in the cable-stayed bridge, and the corresponding variation ratio is between 1%~8%.

(3) Change of ambient temperature will affect dynamic characteristics of cable-stayed bridge, such as frequencies and mode shape values, but it does not change the turn of mode shapes.

(4) Frequencies variation laws caused by asymmetric temperature distribution are similar with those caused by average temperature change. The maximum frequency variation ratio caused by asymmetric temperature distribution approaches to 1% compared with results of average temperature change. Similarly, changes of mode curvature under asymmetric temperature distribution and average temperature distribution are similar.

(5) Frequency changes due to damages of girder are as almost as those due to temperature variation, and frequency changes due to damages of cables are smaller than those due to temperature variation. Obviously, frequency changes due to damages submerge in those due to temperature variation, especially for cables' damage.

(6) Girder damage will lead to obvious mode curvature changes, while variation ration of mode curvature due to cables damage is less than 3%, these changes are easy to submerge in changes of mode curvature due to temperature.

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