

Vibration serviceability of a seven span continuous cable-stayed bridge

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Abstract. This study aims to investigate the vibration serviceability and impact coefficient of a seven-span continuous cable-stayed bridge planned for the extension of a highway. In the design of the bridge, the deflection under the design live load of the continuous cable-stayed bridge would not meet the deflection limit specified in “Japanese Design Specifications for Highway Bridges.” The excessive deflection indicates the possibility of poor vibration serviceability. To clarify the vibration serviceability of the bridge, the dynamic responses of the bridge and passing vehicles are examined by means of the three-dimensional vehicle-bridge coupled vibration analysis. The ride comfort of vehicles on the bridge is examined in terms of vibration serviceability. Observations demonstrated that the vehicle response does not exceed the ride comfort limit under various vehicle, road, and running conditions. In other words, the influence of large deflection on driving safety is negligible. The impact factors were found to be less than 1.05 for the main girder, the base of the main tower, and the cable.

Keywords: cable-stayed bridge; impact factor; seven-span continuous bridge; vehicle-bridge interaction; vibration serviceability

1 Introduction

In bridge design, it is generally required to check the serviceability in addition to the load-bearing capacity and durability[1]. For long-span bridges such as cable-stayed bridges, the deflection under the design live load becomes quite large[2][3]. Excessive deflection will cause vibrations that cause discomfort to users. Moreover, repeated excessive deflections cause a high possibility of fatigue damage in the steel girders and slabs. For this reason, Japanese Design Specifications for Highway Bridges specify the deflection limit of bridges with a span length up to 200 m as “ $L/400$ ” where L is in meter[1]. The deflection limit ensures the necessary stiffness of the bridge so that repeated vibrations such as those caused by passing vehicles will not affect the bridge.

However, for long-span bridges with span lengths that greatly exceed 200 m, it needs

to investigate the influence of excessive deflection on driving safety. Cable-stayed bridges exhibit more flexible behavior than girder bridges and have more complex vibration characteristics due to their complex structure consisting of beams, cables, and pylons. Few studies examine the vibration characteristics of the cable-stayed bridge with an extremely long span. This study aims to investigate the vibration serviceability and impact coefficient of a seven-span continuous cable-stayed bridge with a maximum span length of 650m. The maximum deflection of the bridge under the design live load is expected to be about 4m.

2. Traffic-induced vibration analysis for seven span continuous cable-stayed bridge

2.1 Bridge model

The general layout and cross-sectional view of the target bridge are shown in Figure 1. The bridge is a seven-span continuous cable-stayed bridge. The total length is 2730m long and the span length is 130m+260m+3@650m+260m+130m long. The main tower consists of four A-shaped steel towers in the longitudinal direction of the bridge, and the foundation is a steel pipe sheet pile foundation. The bridge consists of three inbound lanes and three outbound lanes, for a total of six lanes as shown in Figure 1. The main girders are separated from the main tower.

2.2 Analytical method

Dynamic responses of the bridge are taken from three-dimensional traffic-induced vibration analysis[4]. It is based on the finite element method for the modal analysis using three-dimensional models for both vehicle and bridge. It is noteworthy that field experiments on a similar type of bridge were carried out to examine the validity of the vehicle-bridge interaction method for the cable-stayed bridge. The bridge is modeled using beam elements for girders, a truss element for cables, and a spring element with two nodes with the same coordinate for the elastomeric bearing. The bridge model is shown in Figure 2. The vehicle model is composed of the body, tires, and suspension systems. Details of the vehicle model are shown in Figure 3 and Table 1 along with the eight degrees of freedom to describe its movement[4][5]. Rayleigh damping was adopted in the three-dimensional traffic-induced vibration analysis of the bridge, and equations of motion for vehicles and bridge interactive system were solved by the Newmark- β method assuming $\beta = 1/4$. Two vehicle speeds, 80 km/h and 60 km/h, are considered in the traffic-induced vibration analysis. The analysis starts when the lead vehicle reaches at 10m before the bridge. The time interval in the numerical integration was 0.005 s. In the traffic-induced vibration analysis of the bridge, the roadway roughness on the bridge surface is simulated based on the PSD curves shown in Figure 4[6], in which the roadway roughness is assumed to be categorized as Class A according to the ISO estimate[6]. Two different road surface profiles are considered in the analysis.

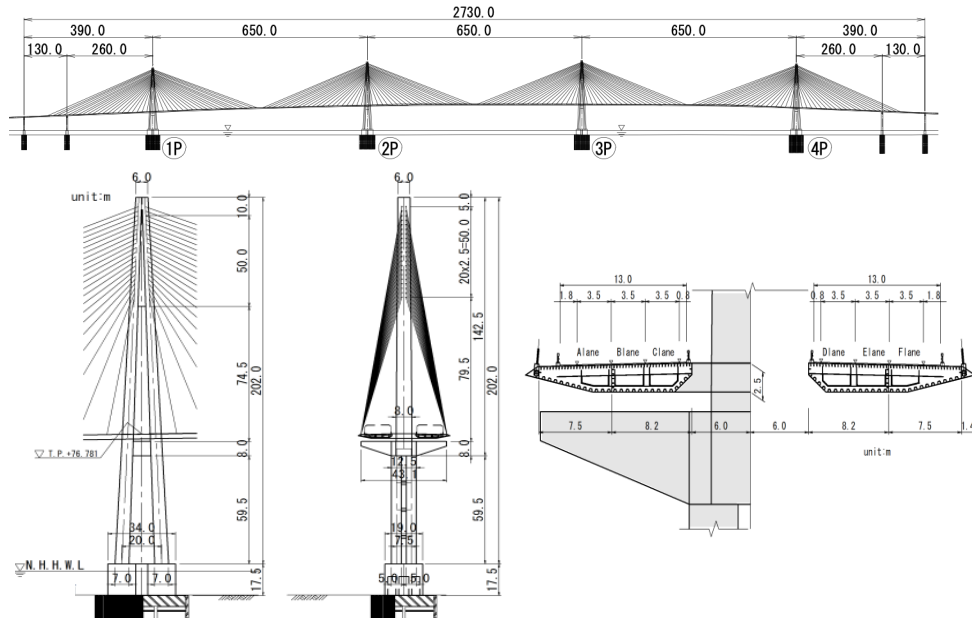


Fig. 1. The general layout of the examined a cable-stayed bridge (unit: m).

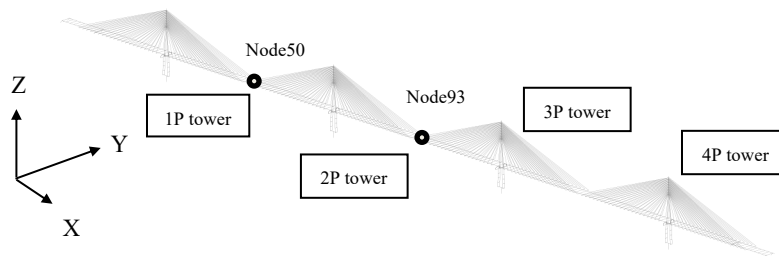


Fig. 2. FE models for bridges.

Table 1. Mechanical properties of vehicle.

Total weight (kN)	245.0
Axle weight (kN)	Front: 61.25 , Rear: 183.75
Damping ratio	Front: 0.66 , Rear: 0.33
Natural frequency (Hz)	Suspension at front (bounce): 2.0, Suspension at Rear (bounce): 3.5 Tyre (hop): 10 - 20
Damping coefficient of vehicle (kN·s/m)	Suspension at front axle: 20.87, Suspension at rear axle: 17.15 Tyre at front axle: 2.45, Tyre at rear axle: 8.62
Spring constant of vehicle (kN /m)	suspension at front axle: 919.2, suspension at rear axle: 7350 Tyre at front axle: 1352, Tyre at rear axle: 5390

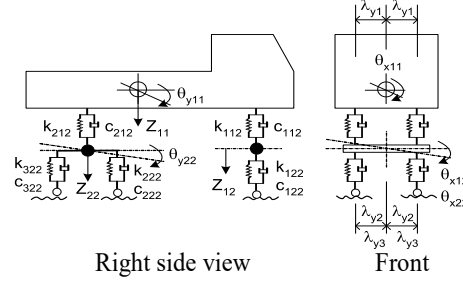


Fig. 3. 8DOF Vehicle.

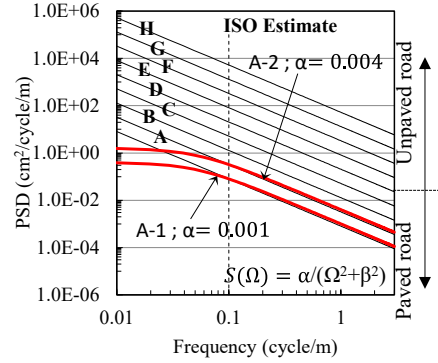
Fig. 4. PSD of road surface roughness⁶.

Table 2. Analysis scenarios for vibration serviceability and impact coefficient.

	Driving lane	Vehicle speed (km/h)	No of vehicles	Resonant headway (m)	Road surface roughness
Case1	A to C	80	4/lane	165	A-1; $\alpha=0.001$
Case2	A to C	80	4/lane	165	A-2; $\alpha=0.004$
Case3	A to C	60	4/lane	125	A-1; $\alpha=0.001$
Case4	A to F	A to C: 80, D to F: 80	A to C: 30/lane, D to F: 10/lane	A to C: 20, D to F: 165	A-1; $\alpha=0.001$
Case5	A to F	80	4/lane	165	A-1; $\alpha=0.001$
Case6	A to C	80	1/lane	165	A-1; $\alpha=0.001$

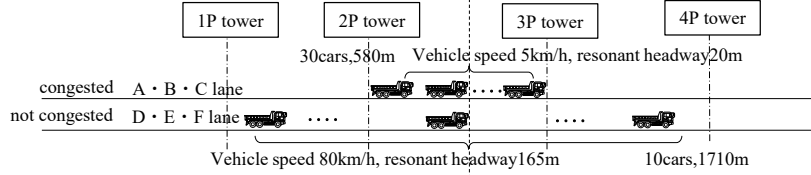


Fig. 5. Vehicle placement for Case 4.

3. Traffic-induced vibration analysis and ride comfort of vehicles

3.1 Methodology

Analysis scenarios are summarized in Table 2. In Table 2 the resonant headway indicates the distance between two vehicles which produces the most excessive vibration of the bridge. Two road surface roughness profiles denoted A-1 and A-2 are considered in the analysis. The difference between the A-1 and A-2 road surface roughness profiles is from the form of the amplitude of the profile, and the A-1 profile is smoother than the A-2 profile. Since the vertical bending mode of the main girder and the sway mode of the main tower are dominant, the resonant headway is set as $L=165\text{m}$ ($= v/f = 22.22(\text{m/s})/0.136(\text{Hz})$). The number of vehicles traveling on one lane is set to four so that four vehicles with a headway of 165m place on the deflection mode of the main girder. Case 4 is the scenario investigating the ride comfort of vehicles during a traffic

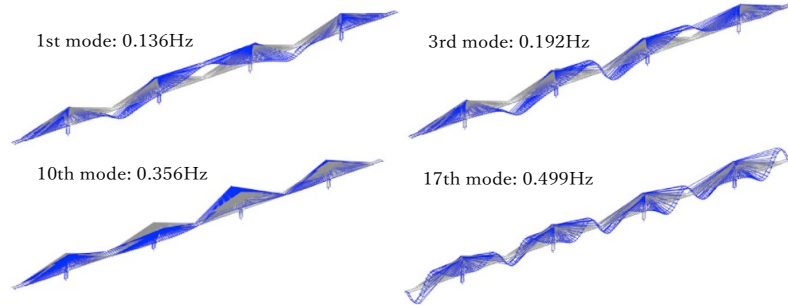


Fig. 6. Selected vibration modes from eigenvalue analysis.

jam. Figure 5 shows a schematic diagram of moving vehicles for Case 4 in which three lanes A, B, and C are congested while another three lanes D, E, and F are not congested and vehicles travel at the speed of 80 km/h.

3.2 Eigenvalue analysis

Selected mode shapes and frequencies of the bridge from the eigenvalue analysis are summarized in Figure 6. Therein, the bending mode of the bridge is observed in the 1st mode (0.136Hz). The 3rd mode (0.192Hz) is the 2nd bending mode of the main girder between the 2P tower and 3P tower. The 10th mode (0.356Hz) shows a sway mode of the main tower, and the 17th mode (0.499Hz) is the 2nd bending mode of all spans. It is noted that those dominant frequencies are far from the bounce motion of vehicles, 2.0 Hz to 3.5 Hz.

3.3 Dynamic responses of bridges and vehicles

Figure 7 shows vertical accelerations of the main girder at the span center between 2P and 3P towers. Case 2 has a worse road surface roughness than Case 1 and Case 3, resulting in a maximum amplitude of 27.21cm/s² of acceleration. On the other hand, Case 3, where the vehicle speed is lower than Case 1 and Case 2 and the road surface roughness is smoother than Case 2, shows the lowest peak amplitude of 8.24cm/s². Therefore, it is obvious that the dynamic response of the main girder is affected by both road surface conditions and vehicle speed.

Fourier spectra of those acceleration responses in Figure 7 are shown in Figure 8. The 1st bending mode of the bridge (see Figure 6) is the major source of the dominant frequency near 0.136Hz. At Fourier amplitudes around 0.136 Hz, the Fourier amplitude of Case 3 is smaller than the other two cases, but there is no clear difference between Case 1 and Case 2, even though Case 1 considers smoother road surface roughness. It is noted that the Fourier amplitude around 3.5 Hz is similar to the frequency for the bounce motion (3.5 Hz) at the rear axle of the vehicle, and Case2 shows the biggest Fourier amplitude which is excited by excessive vehicle vibrations caused by rougher roadway surface condition.

The vertical displacements and influence lines at the span center of the main girder between the 2P and 3P towers for the Case 1 and Case 3 scenarios are shown in Figure 9. As the number of vehicles is the same in Case1 and Case3, the maximum static

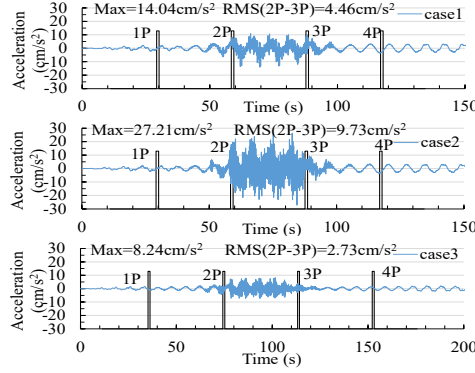


Fig. 7. Acceleration responses at the span center of the main girder between 2P and 3P towers for Case1 to Case3.

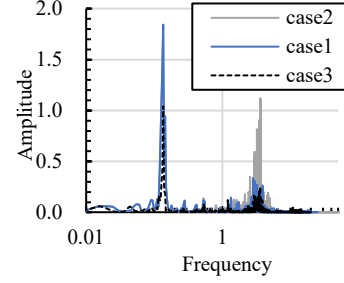


Fig. 8. Fourier amplitude spectra for Fig. 7.

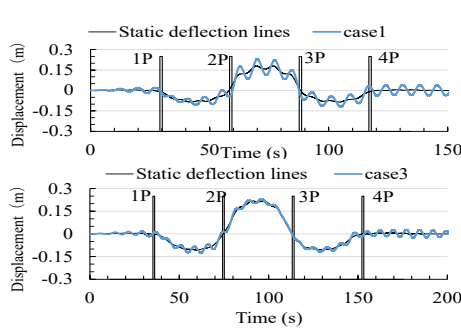


Fig. 9. Displacement responses at the span center of the main girder between 2P and 3P towers for Case1 and Case3.

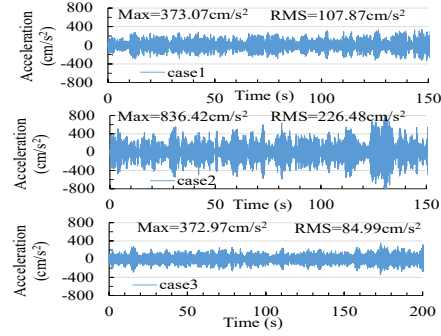


Fig. 10. Accelerations of vehicle's bounce motion for Case1 to Case3.

displacement in both cases is the same at 150 mm. However, because Case1 has a higher vehicle speed than Case 3, the dynamic displacement in Case1 is greater than in Case3.

Figure 10 shows the vertical acceleration responses of the leading vehicle in the A lane (see Figure 1). In the evaluation of the vibration serviceability, the ride comfort of vehicles on the bridge is evaluated using the vertical component of the vehicle's sprung mass. The largest RMS value of the bounce acceleration of the vehicle is observed in Case 2 at 226.48cm/s², and the smallest one at 84.99cm/s² is observed in Case 3. This demonstrates that road surface roughness and vehicle speed are important factors for vehicle ride quality assessment.

3.4 Vibration characteristics of bridges and vehicles in traffic congestion

The vertical displacement at the span center of the main girder between 2P and 3P towers for Case 4 is shown in Figure 11. Case 4 scenario assumes traffic congestion on A, B, and C lanes and normal traffic on E, F, and G lanes. In comparison to Case1 shown in Figure 9, a larger displacement of the main girder than Case1 is observed as expected because Case4 considers the traffic on all six lanes. An example of bounce acceleration

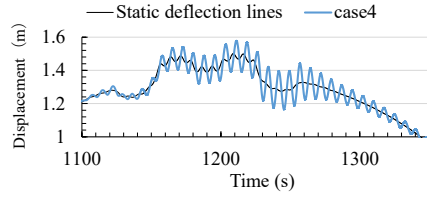


Fig. 11. Displacement responses at the span center of the main girder between 2P and 3P towers for Case4.

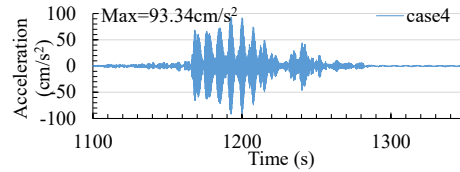


Fig. 12. Acceleration of vehicle's bounce motion for Case4.

of a vehicle traveling on A lane during a traffic jam is shown in Figure 12, in which the vehicle speed of the vehicle during a traffic jam is set as 5 km/h. Vibrations of vehicles in congested lanes could be greatly affected by bridge vibration. As shown in Figure 12, the maximum amplitude of the vehicle acceleration is 93.34cm/s^2 which is smaller than that of the vehicle vibration for Case1.

3.5 Ride comfort of vehicles on the bridge

Methodology. The ride comfort of vehicles on the bridge is investigated based on ISO 2613-1[7] which is an international standard for the evaluation of whole-body vibration exposure. ISO2613-1 describes the evaluation of the "effects of vibration on health", "impact of vibration on comfort", and "effects of vibration on the occurrence of motion sickness". This study only examines the effects of vibration on ride comfort and does not examine the health or motion sickness effects due to the short duration of the vehicle's journey over the target span. Assuming a vehicle traveling at 80 km/h, the time the vehicle stays on the target span is approximately 30 seconds.

The intensity of the vibration is transmitted to the human body through the passenger's seat, and is therefore determined using the bounce vibration of the vehicle obtained from the traffic-induced vibration analysis[4]. Approximate indications of likely reactions to various magnitudes of weighted vibration in some public transports are as follows[7].

- Less than 0.315m/s^2 : *not uncomfortable*
- 0.8 to 1.6m/s^2 : *uncomfortable*
- Greater than 2m/s^2 : *extremely uncomfortable*

Ride comforts. The weighted RMS acceleration of the vehicle when running at different speeds, from Case1 to Case4, is shown in Figure 13, together with boundaries for ride comfort. The three curves in Figure 13 show the boundaries of ride comfort, from top to bottom: "*extremely uncomfortable*", "*uncomfortable*" and "*not uncomfortable*". The dominant RMS accelerations between 1 and 3 Hz are relevant to the bounce motion at the front axle of the vehicle. Those around 3.5 to 4.0 Hz are related to the bounce motion at the rear axle of the vehicle, while the dominant RMS accelerations around 10 to 30 Hz are caused by the vehicle's unsprung vibrations. Considering that the natural frequency of the first bending mode of the bridge is 0.136 Hz, which does not match the vehicle bounce frequency of 3.5 Hz, the vibration of the cable-stayed bridge is unlikely to affect vehicle ride comfort.

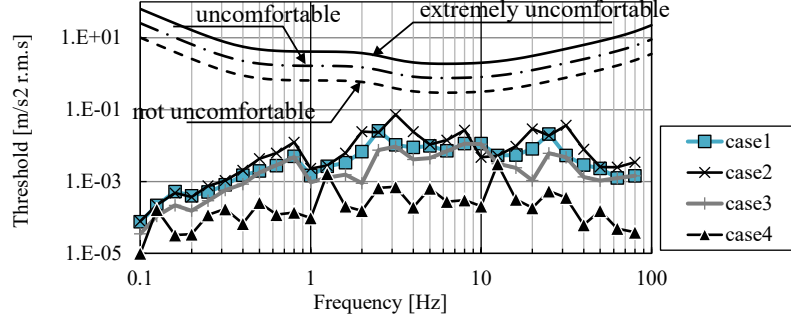


Fig. 13. Evaluation of the ride comfort of vehicles.

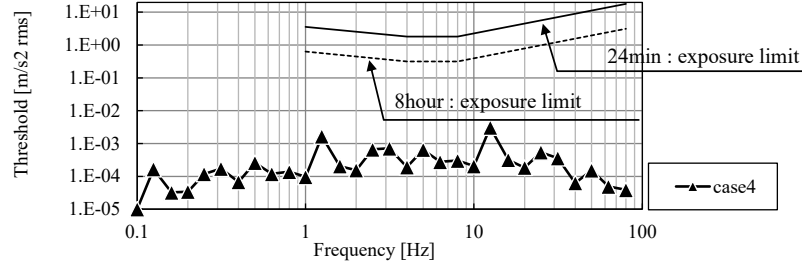


Fig. 14. Evaluation of the ride comfort of vehicles during traffic congestion considering fatigue-decreased proficiency boundary for Case4.

Vibration comfort of vehicles during traffic congestion. During traffic congestion on long-span bridges vehicles are exposed to a long duration of vibration. Therefore, the vibration comfort of vehicles considering fatigue-decreased proficiency boundary[8] is worth investigating. Figure 14 shows the weighted RMS acceleration of the vehicle for Case 4 with a fatigue-decreased proficiency boundary, where “24min” and “8 hours” denotes the fatigue-decreased proficiency boundaries for the exposure time of 24 minutes and 8 hours respectively. As can be seen in Figure 14, the weighted RMS acceleration of the vehicle on congested lanes is far below the boundary for the exposure time of 8 hours. As it is unlikely that the vehicle will be on the bridge for several hours during traffic jams, there is no problem with vibration comfort even during traffic congestion. The vehicle speed during traffic jams was assumed to be 5 km/h, but it seems that driving at a constant speed has a greater influence on ride comfort than when the vehicle is stopped.

4. Impact factor

4.1 Definition

In highway bridge design, dynamic amplification of bridge responses due to traveling vehicles are considered using the impact factor or dynamic amplification factor. In the Japanese highway bridge design code, the impact factor is defined as $i=20/(50+L)$ (L : span length in meters). The impact factor is applicable to bridges with a span length of

less than 200 m, however, the cable-stayed bridge in this study has a span length of 650m long which is out of the applicable range of the impact factor. Therefore, the impact factor of the bridge is investigated by means of traffic-induced vibration analysis of the bridge. The impact factor is calculated as the ratio of the maximum static response and maximum dynamic response[4].

The definition of the impact factor is shown in Figure 15. The impact factor is the maximum dynamic amplitude " $y_{l,dmax}$ " divided by the maximum static amplitude " y_{stmax} ", plus one. " $y_{l,dmax}$ " is the maximum value of the dynamic amplitude " $y_{l,d}$ ", and is obtainable by subtracting the static deflection " y_{st} " from the dynamic deflection response " y_{dy} " obtained by traffic-induced vibration analysis. " y_{stmax} " is the maximum displacement when the design live load is loaded on the bridge.

4.2 Impact factor of main girder

The estimated impact factors of the main girder are summarized in Figure 16. The impact factor is found to be in the range of 1.01 to 1.05. Although the amplitude of road unevenness is larger than in other scenarios, the impact factor for Case 2 shows the same value as Case 1. One reason is that the bridge vibration is dominated by the first bending mode at 0.136 Hz, and the contribution from the vehicle vibration with a frequency of around 3.5 Hz was extremely weak. When multiple vehicles with resonant headway are traveling on all lanes like in Case 5, the largest impact factor is observed among six scenarios. On the other hand, lower impact factors were observed for slower vehicle speeds and fewer vehicles, as in Case 2 and Case 6.

4.3 Impact factor of main tower and cable

The impact factors of the main tower at the middle section and tower base are calculated from the bending moment and summarized in Figure 17. The impact factors of cables, which are obtained from analytical tensions, are summarized in Figure 18. The impact factors for the main towers and cables were less than 1.05.

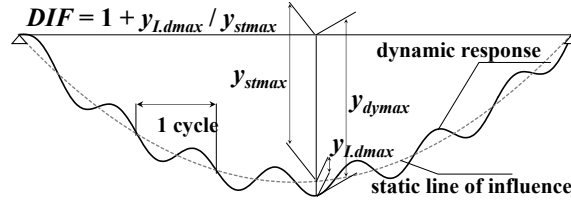


Fig. 15. Definition of impact factor.

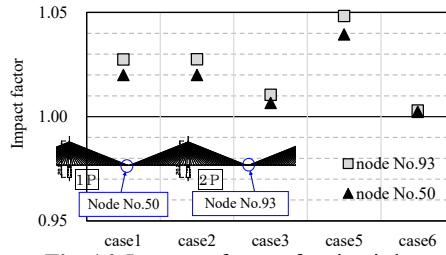


Fig. 16. Impact factor of main girder.

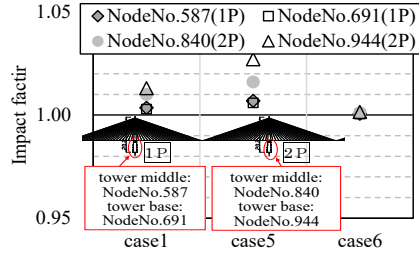


Fig. 17. Impact factor of main tower.

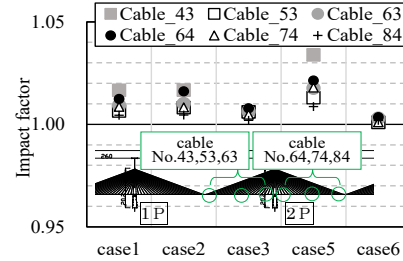


Fig. 18. Impact factor of main cable.

5. Conclusions

This study analytically investigates the vibration serviceability and impact factors of a seven-continuous cable-stayed bridge with a maximum span length of 650m where the live load deflection is expected to be about 4m. Three-dimensional traffic-induced vibration analysis for the bridge is used in the study. Observations through the study are summarized as follows.

The ride comfort of vehicles on the bridge is investigated based on ISO 2613-1 for the evaluation of whole-body vibration exposure. Observations demonstrated that the vehicle response does not exceed the ride comfort limit regardless of vehicle speed, roadway surface roughness condition, and the number of vehicles on the bridge. Although a large deflection is expected under design live load, it was confirmed that the impact of large deflections on driving safety is negligible.

The impact factors were found to be less than 1.05 for the main girder, less than 1.03 for the base of the main tower, and less than 1.04 for the cable. The largest impact factor is observed when several vehicles are running with resonant headway at higher speeds.

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