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Vibration reduction design of the Hangzhou Bay cable-stayed bridges

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Abstract. Hangzhou Bay Bridge spans the Hangzhou Bay and is located at Zhejiang province in the southeast seacoast of China. The total length of the bridge is 36 km. The bridge is composed of bridge approaches made up of multi-span prestressed concrete box girders and two cable-stayed bridges over the north and south navigable spans respectively. The seismic response analysis of the bridge model shows that if the navigable spans are designed as the routine earthquake-resistance system, the displacements and internal forces in pylons, piers and deckes are too large to satisfy the anti-seismic requirement of the structure. Therefore, the seismic reduction design was carried out by using viscous dampers to dissipate the kinetic energy of the structure both longitudinally and transversely. Using the vibration reduction system and aiming at the reasonable optimal goal, the purpose to reduce the seismic responses in south and north navigable spans has been achieved.

Keywords: Hangzhou Bay Bridge; cable-stayed bridge; earthquake response; seismic reduction; viscous damper; optimal design.

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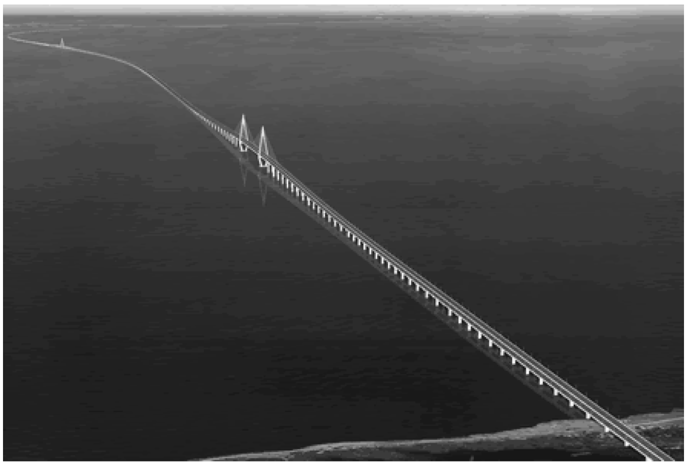

Fig. 1 Overview of the Hangzhou Bay Bridge

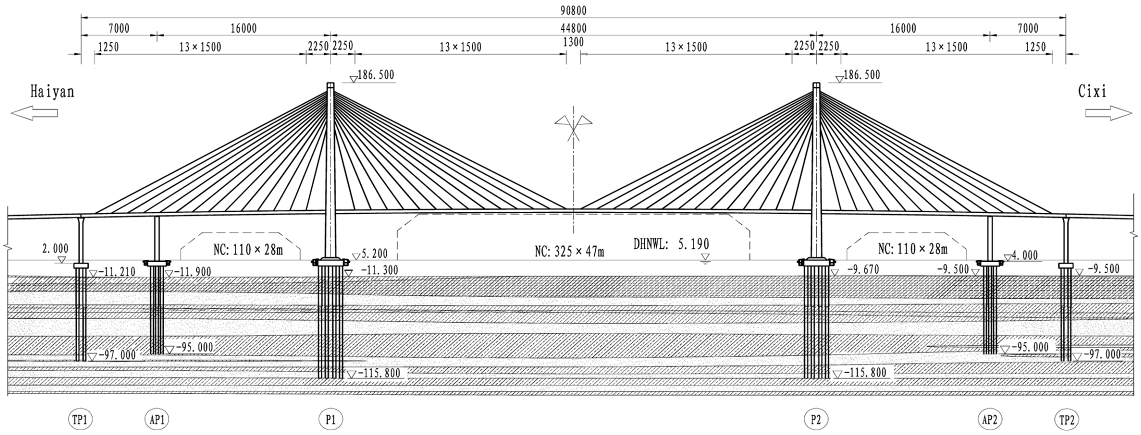

Fig. 2 Cable-stayed bridge over the north navigation channel (unit: cm)
NC–navigation clearance; DHNWL–highest water level; TP1, TP2–transition piers; AP1, AP2–auxiliary piers; P1, P2–pylons

1. Introduction

Hangzhou Bay Bridge as shown in Fig. 1 is located at Zhejiang province in the southeast seacoast of China. The bridge starts at Zhenjiadai locating at the north coast in Haiyan County, spans the Hangzhou Bay, and ends at Andongzhen locating at the south coast in Cixi City. The total length of the bridge is about 36 km. This sea-crossing link provides the shortest passage from Tongjiang to Sanya for “Five Longitude and Seven Latitude Highways” in the national highway network. Over the north navigation channel of the Hangzhou Bay is a twin-pylon cable-stayed bridge structure as shown in Fig. 2, and over the south navigation channel is a single-pylon cable-stayed structure as shown in Fig. 3. The twin-pylon structure has two cable planes with a total length of 908 m that is distributed over five spans of 70 m, 160 m, 448 m, 160 m and 70 m, respectively. The single-pylon structure also has two cable planes but with a total length of 578 m that is distributed over three

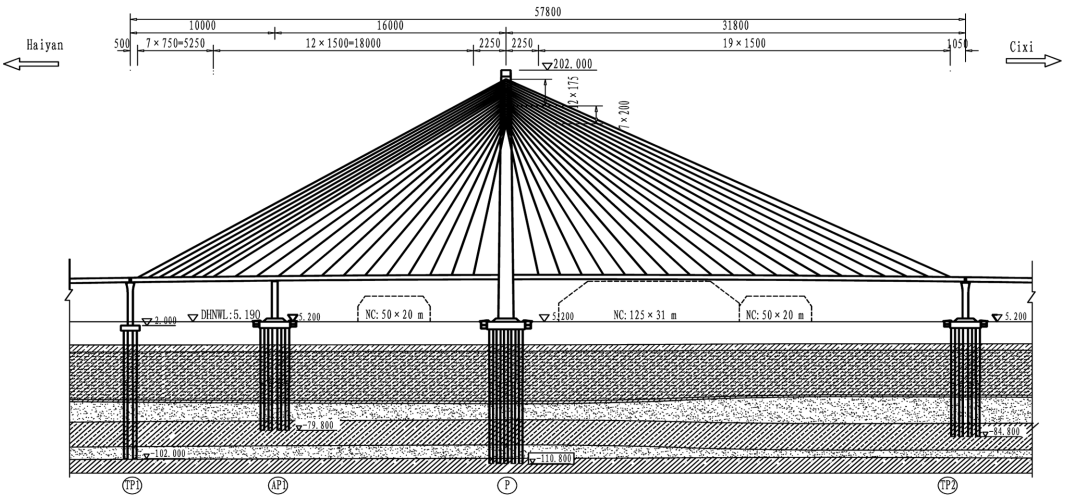

Fig. 3 Cable-stayed bridge on the south navigation channel (unit: cm)

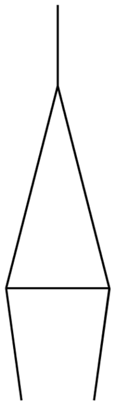

Fig. 4 Shape of the pylons in the north navigation channel

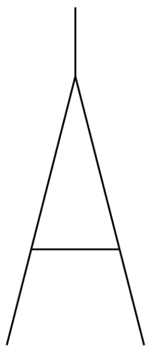

Fig. 5 Shape of the pylon in the south navigation channel

spans of 100 m, 160 m, and 318 m, respectively. Made of reinforced concrete, the two pylons in the north navigation channel extends 130 m above the bridge deck, and they are in diamond shape as shown in Fig. 4. As a longitudinally floating system, the main girders of the twin-pylon structure are transversely free on the auxiliary piers. The transverse wind-resistant settings are installed on the cable-stayed structure side of the transition piers only. The pylon in the south navigation channel is also made of reinforced concrete with a height of 160 m above the deck. It is in A-shape as shown in Fig. 5. The main girders of the single-pylon structure are longitudinally and transversely restrained at the pylon but free at the auxiliary pier in both directions. The transverse wind-resistant settings are also installed on the cable-stayed structure side of the transition piers. Both the south and the north cable-stayed structures are composed of steel box girders of nearly identical cross sections. Pile foundations are used to support all the piers and pylons in the north and south cable-stayed structures. The bridge approaches are made of multi-span prestressed concrete box girders with a varying span length from 30 m to 80 m. Note that, up to now, the Hangzhou Bay Bridge is the world's longest cross-sea bridge. The construction of the cable-stayed bridges began in 2003 and will be completed in 2008, open to traffic in 2009.

2. Earthquake resistant design criterion and seismic response analysis

Hangzhou Bay Bridge is a critical engineering structure in China. Its earthquake resistant design must meet some special requirements, in addition to the code-specified regulations. After a detailed feasibility study, it was decided to design the cable-stayed bridge structures for two levels of earthquake hazards: 10% probability of exceedance in 100 years, designated as Level P1, and 3% probability of exceedance in 100 years, or Level P2. The detailed design criterions for the two earthquake hazard levels are summarized in Table 1. Corresponding to the P1 and P2 levels of earthquake hazards, the peak ground accelerations are given in Table 2.

The dynamic characteristics and seismic time-history analysis of the south and north cable-stayed bridge structure were performed by using the SAP 2000, a nonlinear commercial finite element software. In the finite element analysis, spatial frame elements were adopted to simulate the deck, pylons and piers. The vertical stiffness, transverse stiffness and torsion stiffness, and the translational mass and rotational inertia of each element were mounted on the middle joint of the element. The deck and cables are connected at corresponding joints by the rigid arms.

The stayed cables were simulated by the large-displacement truss elements, and their Young's modulus $\boldsymbol{E}_{\mathrm{eq}}$ was modified, taking into account the effect of their sag and geometric rigidity caused by the dead load:

$$E_{eq} = \frac{E_0}{1 + \frac{\gamma^2 l^2}{12\sigma_0^3} E_0} \tag{1}$$

where E_0 is the Young's modulus of the cable material; γ is the cable weight of unit volume; l is the horizontal initial length of the cable; σ_0 is the stress in the cable.

Table 1 Design criterions for the Hangzhou Bay Bridge

Earthquake level	Design criteria
P1	The primary members of the structure are in elastic range so that the bridge can be open to traffic immediately after an earthquake event.
P2	The primary members of the structure are in inelastic range but they do not reach their ultimate limit states. The displacement and deformation are under control so that collapse will not occur.

Table 2 Peak ground accelerations for design earthquakes (gal)

Sites	P1		P2	
	Horizontal direction	Vertical direction	Horizontal direction	Vertical direction
North span	70 gal	60 gal	100 gal	95 gal
South span	50 gal	60 gal	80 gal	95 gal

The soil-structure interaction was also considered in the analysis. Provided by the client, ground acceleration time histories at the bridge site were given in three components: longitudinal, transverse and vertical seismic inputs. Each earthquake wave lasts for 40.96 sec and it is specified at an interval of 0.02 sec. According to the design specifications, two rules of combining the effects of component seismic inputs in two directions were considered. They are the longitudinal input plus one half of the vertical input, and the transverse input plus one half of the vertical input. In the numerical analysis, the first 300 modes of the bridge structure were included, which covers the main modes of vibration in pylons, deck, transition piers and auxiliary piers. The damping ratios of the concrete and steel are assumed to be 5% and 1%, respectively. Since the deck in the south and north cable-stayed bridge structures are made of steel and their pylons are made of concrete, a damping ratio of 3% was taken in the numerical calculation.

The analysis results indicated that under the longitudinal seismic excitation, the internal force at the bottom section of the upper pylon-pole in the north navigation channel is greatly larger than that at other locations and can be considered as the main control index in the design. Furthermore, the longitudinal displacements of the pylon and deck in the north navigation channel are also rather larger. Under the transverse seismic excitation, the shear forces on the bearings of the wind-resistant settings and the internal forces in the pylons and in the side of the transition piers with the wind-resistant setting are larger both in the south and north navigation channels. These forces were also considered as a control index in the design (Xu *et al.* 2004, Liu *et al.* 2004).

3. Method of seismic effect reduction design

3.1 Energy dissipation principle

The technique of energy dissipation is a relatively new research field in bridge engineering (Soong and Dargush 1997, Housner *et al.* 1997, Delis *et al.* 1996, Jung *et al*. 2003). The basic principle is to set some energy-dissipated devices on the bridge to increase the structural damping and absorb the kinetic energy of the structure. This will result in a reduction of the structural vibration amplitude coming from the seismic excitations, wind loads and other external loads. Using the vibration reduction design, the stability of a long-span bridge under wind loads and earthquake excitations can be effectively enhanced. Moreover, the wearing and tearing action of bearings and expansion joints can be reduced significantly; the vehicle's operation condition can be improved; and the service life of the bridge can be prolonged. Therefore, this technology is valuable for the seismic reduction of bridges in the areas of frequent earthquake events.

The seismic response control of cable-stayed bridges was studied by Dyke *et al*. (2003) and Agrawal *et al*. (2003). The benchmark structure they analyzed is the Cape Girardeau cable-stayed bridge in the U.S. The effectiveness of several semi-active control systems, including setting semi-active stiffness damper (SSASD), active variable stiffness (AVS) and semi-active friction damper (SAFD), were evaluated. Passive viscous dampers with and without a linear spring were used for a comparison between the passive and semi-active systems. In practical applications, passive devices such as viscous dampers are often the first choice of energy dissipation systems in some developed countries. For example, passive viscous dampers were applied to the Golden Gate Bridge in the U.S. (Huang 2000) for seismic response reduction. They were used to limit the longitudinal displacement and vibration induced by live loads and wind loads in the Great Belt Bridge in

Denmark. As velocity-dependent devices, viscous dampers can consume a significant amount of energy without increasing the stiffness of the bridge and allow the bridge deck freely to deform when the temperature changes. Moreover, passive viscous dampers have excellent stability and require no or little maintenance; their damping coefficients are adjustable over a wide application range. Based on the above discussions, viscous dampers were adopted in the design of the Hangzhou Bay Bridge.

3.2 Common restrained system of long-span cable-stayed bridge

The representative layout of spans in the design of modern cable-stayed bridges is the twin-pylon-three-span and the single-pylon-two-span. For either form of the span layout, the auxiliary piers can be placed at the side spans if necessary. For the cable-stayed bridge of twin-pylon-three-span, there is no restraint on longitudinal displacement of the deck at pylons in order to decrease the internal forces of the structure under longitudinal earthquake excitation. However, for the cable-stayed bridge of single-pylon-two-span, the longitudinal displacement of the deck at the pylon is restrained. In the transverse direction, whether it is a single-pylon bridge or a twin-pylon bridge, the displacements of the deck at pylons are always restrained. For the design of transition piers, there are two possible choices to make the bridge structure in compliance with the design requirements.

1) The deck is free at both transition piers: This scheme can greatly decrease the internal forces of the structure under the temperature change and wind loads. However, large displacements of the deck in the lateral direction will occur at the expansion joints. This will result in the large shearing deformation of expansion joints and shorten the service life of expansion joints. Because of the drawback given above, this scheme is rarely adopted nowadays.
2) The deck is free at one pier and laterally restrained at the other pier. This scheme can control the lateral displacement of the deck at the expansion joints and prevent the expansion joints from the shear damage. However, larger internal forces from the temperature change and/or the lateral loads will occur at the restrained transition pier and the difference of internal forces in two transition piers is larger. In the following section, the technique to solve the above-mentioned limitations will be discussed. A reasonable reduction vibration system will be designed.

Based on the demand-on-capacity ratio of the structure under transverse loads, all auxiliary piers were designed to be transversely restrained at two locations each. The transverse restraints to the auxiliary piers can decrease the internal forces in pylons and transition piers under the transverse loads. However, the internal forces of the auxiliary piers will increase. Therefore, the locations and number of the deck restraints at the auxiliary piers should be carefully determined through calculation and analysis.

3.3 Longitudinal seismic reduction system

If there is no restraint on the deck at pylons in the longitudinal direction, viscous dampers can be installed on the deck near pylons and connected to the pylons in the longitudinal direction. In such a case, the displacements and internal forces of the deck and pylons under the longitudinal seismic excitation can be obviously reduced and the size of the expansion joints can be minimized when the bridge is built in the active earthquake region. If the cable-stayed bridge is very large or the seismic intensity is very high, the size of the viscous dampers should correspondingly increase in order to provide enough damping force. The installation of the large viscous dampers will complicate the design at the installed locations between the deck and the pylons. In this case, the longitudinal viscous dampers may also be installed on both the auxiliary piers and the transition piers. According to the longitudinal support system used at the north navigable bridge as mentioned the above, two pairs of two longitudinal viscous dampers each have been installed on the deck near the pylons and are symmetrically connected to the pylons. This optimal design results in the great decrease of the longitudinal internal forces and displacements in the pylons. The computational results show that the internal forces of the deck from the seismic response are smaller than those from the wind loads and temperature change. In such a case, the internal forces and displacements caused by seismic excitations will be no longer the main controlling factor. The total damping force of longitudinal viscous dampers is only 1100 kN and the connection design of the dampers at this load level is straightforward. Therefore, there is no need to install the viscous dampers on the auxiliary piers and transition piers in the longitudinal direction. It should be mentioned that the longitudinal seismic reduction on the pylons in the south navigable span is inefficient because the deck is limited to longitudinal motion.

3.4 Transversal seismic reduction system

The seismic reduction system of the cable-stayed bridges in the transverse direction normally consists of seismic reduction devices installed, respectively, on the transition piers and a part of the auxiliary piers. According to the restraint condition of the structure, the seismic reduction devices on the transition piers can be installed either at one side of the piers or at both sides of the piers. For the second option of design compliance for transition piers, as discussed in Section 3.2, if the transverse restraint on the deck and the lateral restraint on the transition piers are not allowed to be damaged in any case, the transverse seismic reduction devices are only installed on the free side of the transition piers. However, if the transverse restraint is allowed to be snipped under a design earthquake, the devices should be installed on both sides of the transition piers. As the parameters of seismic reduction devices are related to the structural system, the design goal of the transverse restraint should be clearly stated when the analysis of seismic response is performed. The design of transition piers in south and north navigable spans of the Hangzhou Bay Bridge is one pier free and the other pier restrained laterally. The design goal is that the bearing of the transverse wind resistance is not allowed to be damaged in any case. Therefore, the transverse viscous dampers will be installed only on the free side of the transition piers.

When the transverse viscous dampers are installed on the auxiliary piers, the horizontal seismic forces of the deck in the transverse direction can be transferred to each pylons and piers. As a result, the forces in the transition piers and pylons could be reduced and evenly distributed among all of the components. Hence, the total capability of the structure to resist the seismic excitation will increase and the bearing capacity of each component can be increased adequately (Gong *et al.* 2000). If the parameters of the vibration reduction devices are properly selected, the internal forces in auxiliary piers may also be reduced. Therefore, for the transverse vibration reduction of a cable-stayed bridge, conventional transverse restraints must be designed only for some auxiliary piers and the viscous dampers are introduced for the other piers. In this case, the vibration reduction devices can function as conventional transverse restraints, but do not increase the temperature stress and stiffness of the structure. If the cable-stayed bridge has multiple auxiliary piers, the energy dissipated devices can be installed on some or all of the piers, which is determined by the requirements on earthquake resistance. The number and position of the devices to be used depend

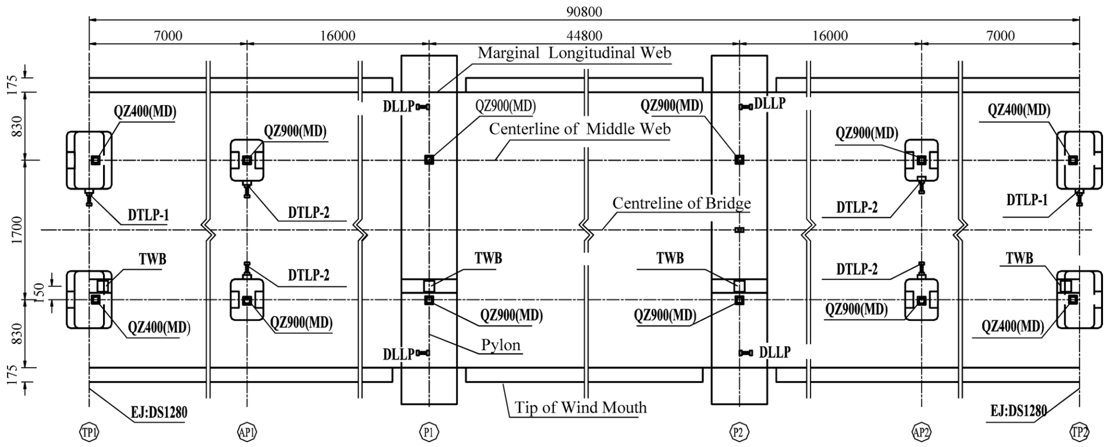

Fig. 6 Seismic response reduction system in the north navigable span (unit: cm)
MD–multi-direction; QZ–spherical steel bearing; EJ–expansion joints; TWB–transverse wind bearing; DLLP-1–damper of longitudinal limiting position; DTLP-2–damper of transverse limiting position; TP–transition piers; AP–auxiliary piers; P–pylons

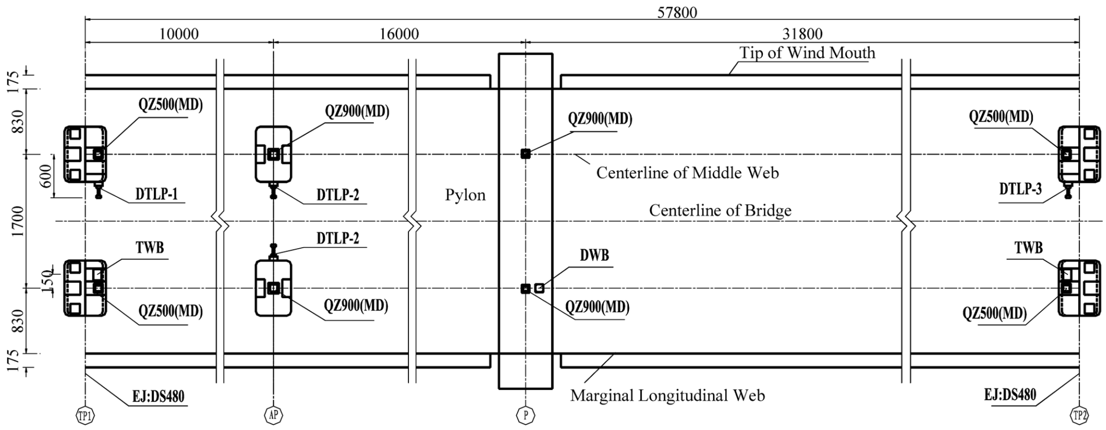

Fig. 7 Seismic response reduction system in the south navigable span (unit: cm)
DWB–double directions wind bearing

on the optimal performance determined from several comparative analyses on different design schemes. Our analysis results showed that the vibration reduction devices should be installed on all of the auxiliary piers in both north and south navigable spans.

In the above vibration reduction scheme, the viscous dampers installed in longitudinal and transverse directions, if properly configured, can independently work in a cable-stayed bridge. Therefore, the vibration reduction design can be carried out individually in the longitudinal and transverse directions for a specified structure. The coupled design in both directions can also be made if deemed necessary. The seismic reduction measures in the north navigable span have been taken in both longitudinal and transverse directions, as shown in Fig. 6. For the south navigable span, however, only in the transverse direction was the seismic reduction measure taken as illustrated in Fig. 7.

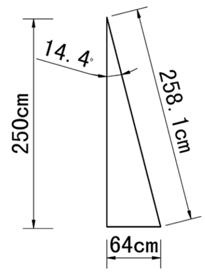

Fig. 8 The limit position of transverse viscous damper

3.5 Special requirements on vibration reduction devices

To ensure that the transverse damping devices operate appropriately when the bridge deck experiences a significant longitudinal displacement within the capacity of expansion joints, the transverse damping system (devices and their connection) must be able to accommodate the significant rotation induced by the longitudinal motion, as shown in Fig. 8. For example, if the length of a transverse damping device is 2.5 m and the allowable maximum deformation of an expansion joint is ±0.64 m, the rotational angle required is:

$$\theta = \arctan\left(\frac{0.64}{2.5}\right) = 14.4^{\circ} \tag{2}$$

In this case, the effects of the longitudinal deck displacement must be taken into account for the determination of the damping force and the maximum displacement of transverse damping devices. At the ultimate displaced position of the deck, the demand on the transverse damping devices reaches at a maximum. The damping force (F) and the maximum displacement (d) in this case must be larger than the damping force (F_0) and the displacement (d_0) when the deck were restrained for longitudinal motion. They are related by:

$$F = (258.1/250)F_0 \tag{3}$$

$$d = 1.032d_0 + 8.1\text{cm} \tag{4}$$

The maximum displacement of a longitudinal damping device must also be within the allowable deformation of expansion joints. Otherwise, the damping device will be partially restrained in motion and can not operate properly as the longitudinal displacement of the deck goes beyond the permissible bearing deformation.

4. Optimal analysis to reduce seismic responses

For each seismic response reduction system, the optimal design of a damping device in terms of minimum structural vibration can be conducted by adjusting the damping coefficient of the damping device and the velocity index. Although the north navigable span in plan is symmetric about the mid-span of the center span in the longitudinal direction, the varying elevations of the bridge foundation levels result in the asymmetric seismic responses. Even at two symmetric locations, the internal forces in pylons and transition piers are quite different. The structural components in the north and south navigable spans are also transversely symmetric about the centerline of the bridge. However, in order to limit the lateral displacements of the main girder at expansion joints and to keep it from the shearing failure, a transverse wind-resistant shear key was installed on one side of the transition piers with the other side of the piers free to move. In such a case, the differences of the internal forces on two transition piers at two symmetric points are significant. According to the above discussions, when the seismic response reduction analysis was carried out, our main goal was focused on the internal forces in the same type of components as small and evenly distributed as possible. This has been achieved by optimizing the coefficients of the viscous dampers to make full use of the capacity of all structural components and minimizing the required damping forces at the same time. To illustrate the effectiveness of the seismic response reduction scheme employed in this study, Figs. 9-25 and Tables 3-5 show the internal forces and joint displacements for the north and south navigable spans, respectively (Xu 2004, Liu 2004).

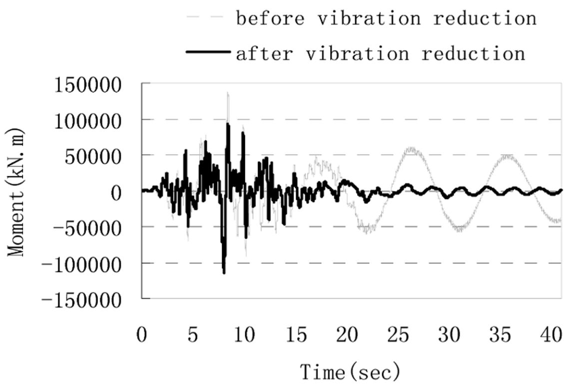

Fig. 9 Moment at upper section of the pylon in north navigable span under the longitudinal excitation P1

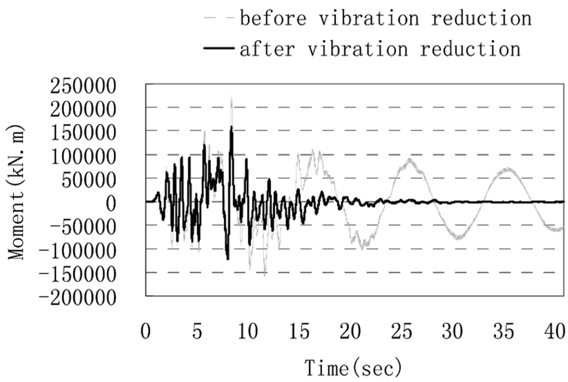

Fig. 10 Moment at the upper section of the pylon in north navigable span under the longitudinal excitation P2

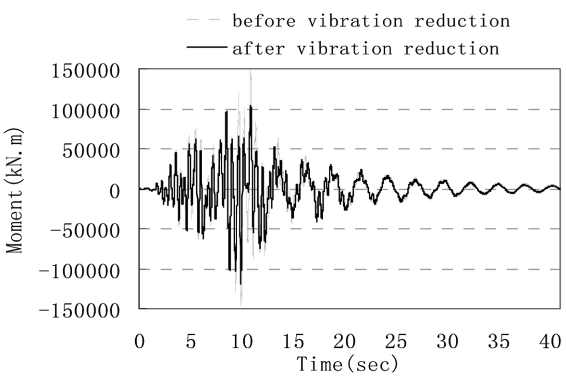

Fig. 11 Moment at upper section of the pylon in north navigable span under the transverse excitation P1

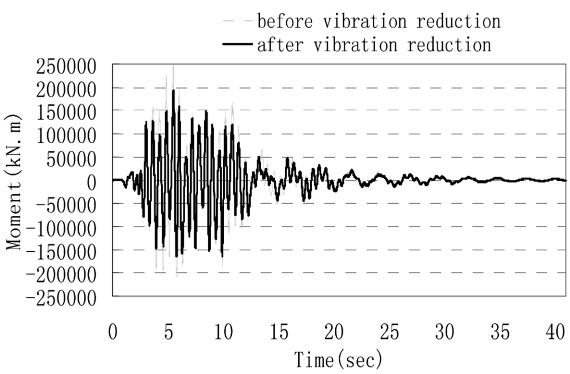

Fig. 12 Moment at upper section of the pylon in north navigable span under the transverse excitation P2

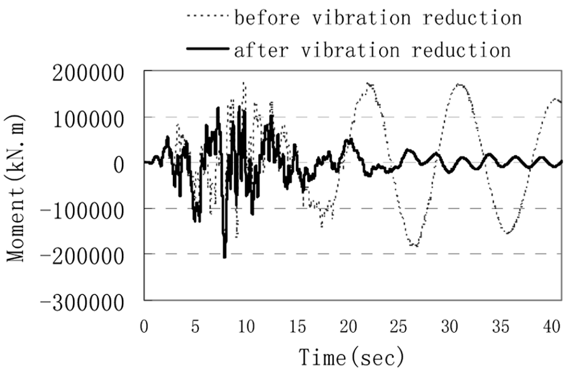

Fig. 13 Moment at bottom section of the pylon in north navigable span under the longitudinal excitation P1

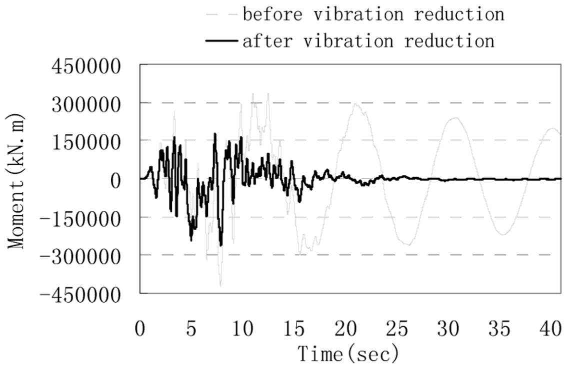

Fig. 14 Moment at bottom section of the pylon in north navigable span under the longitudinal excitation P2

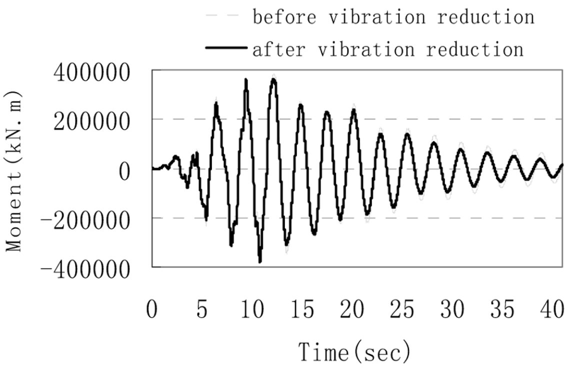

Fig. 15 Moment at bottom section of the pylon in north navigable span under the transverse excitation P1

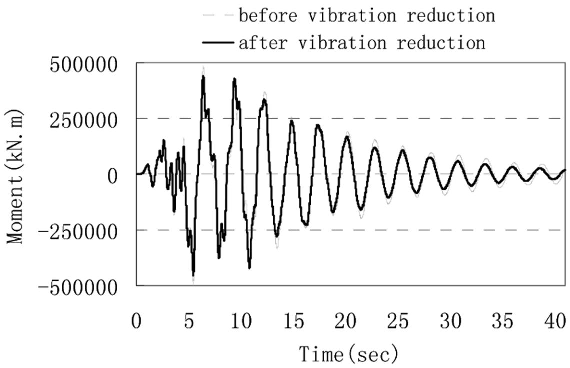

Fig. 16 Moment at bottom section of the pylon in north navigable span under the transverse excitation P2

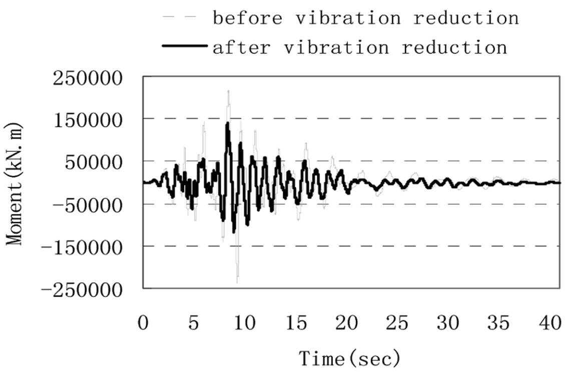

Fig. 17 Moment at bottom section of the transition pier in north navigable span under the transverse excitation P1

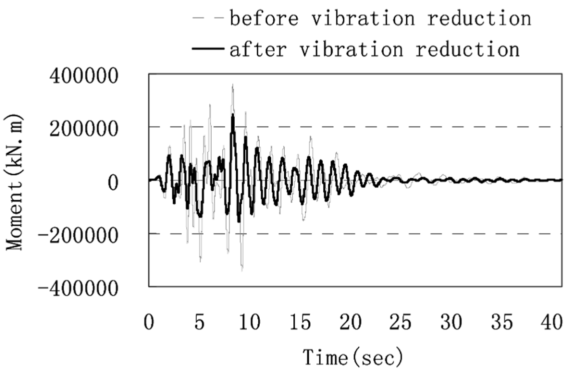

Fig. 18 Moment at bottom section of the transition pier in north navigable span under the transverse excitation P2

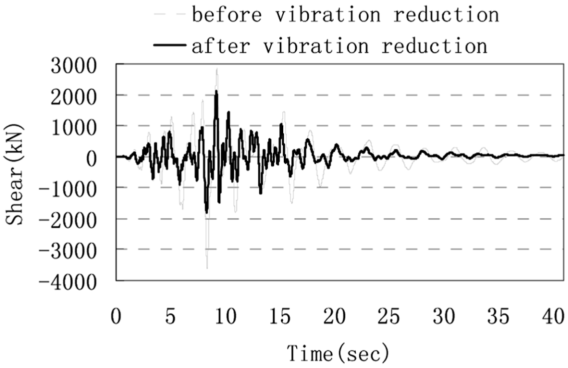

Fig. 19 Horizontal shear of the wind-resistant setting on the transition pier in north navigable span under the transverse excitation P1

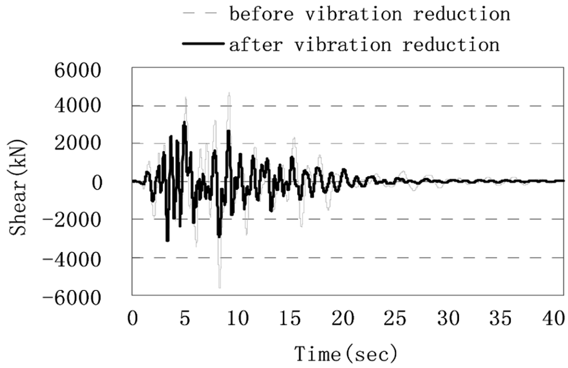

Fig. 20 Horizontal shear of the wind-resistant setting on transition pier in north navigable span under the transverse excitation P2

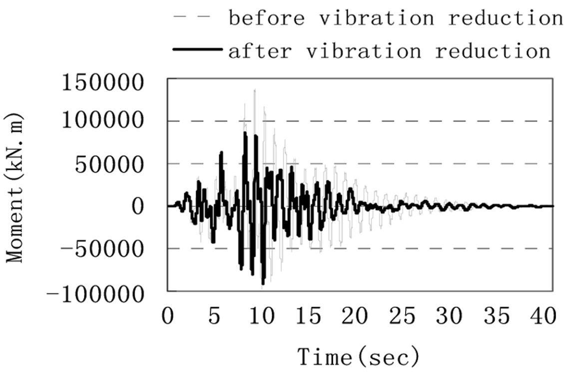

Fig. 21 Moment at bottom section of the auxiliary pier in north navigable span under the transverse excitation P1

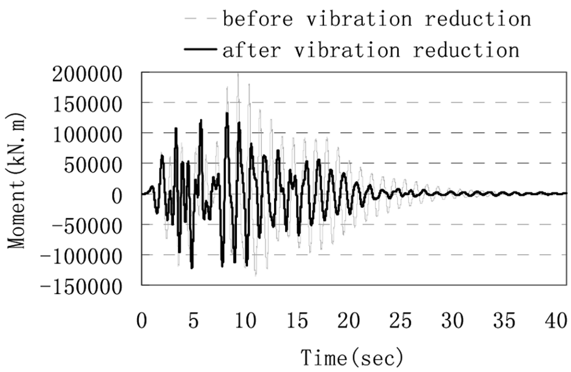

Fig. 22 Moment at bottom section of the auxiliary pier in north navigable span under the transverse excitation P2

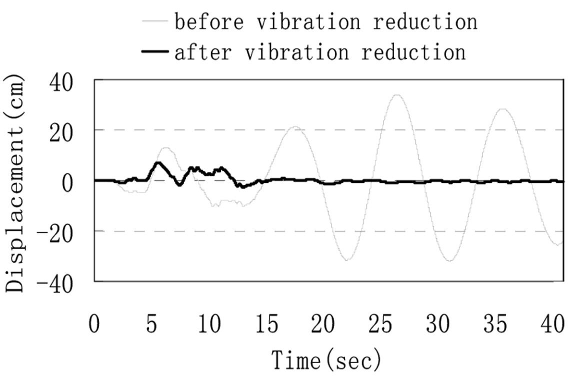

Fig. 23 Longitudinal displacement of the deck in north navigable span under the longitudinal excitation P1

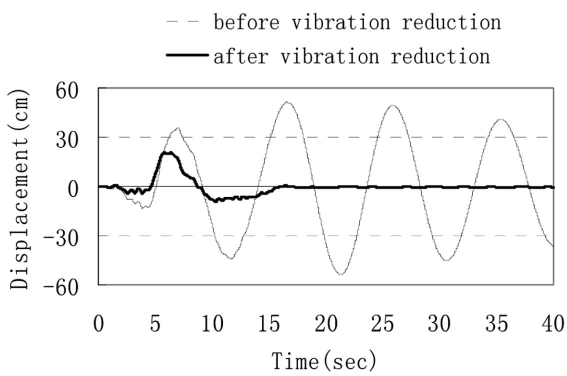

Fig. 24 Longitudinal displacement of the deck in north navigable span under the longitudinal excitation P2

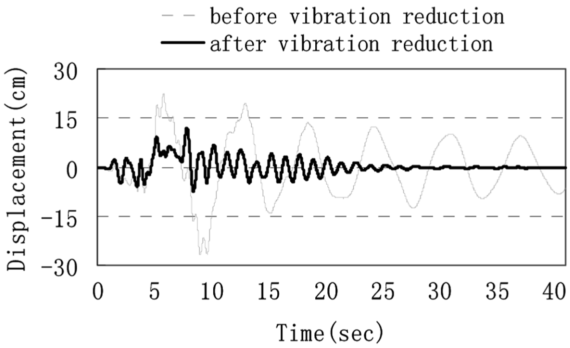

Fig. 25 Transverse deck displacement at the transition pier in north navigable span under the transverse excitation P2 when the wind-resistant setting becomes malfunctional

Table 3 Damping effect on the north navigable span under the probability level P2

Location of the section	Before vibration reduction	After vibration reduction	Vibration reduction ratio (%)
longitudinal moment (kN·m) at bottom section of pylon	436557	296170	32.2
transverse moment (kN·m) at bottom section of pylon	523153	455359	13.0
longitudinal moment (kN·m) at bottom section of the upper pole on pylon	225399	158879	29.5
transverse moment (kN·m) at bottom section of the upper pole on pylon	258887	215944	16.6
longitudinal moment (kN·m) at top section of the upper sloped-pole on pylon	126899	79976	37.0
transverse moment (kN·m) at top section of the upper sloped-pole on pylon	114868	71498	37.8
transverse moment (kN·m) at bottom section of pier TP1 with wind-resistant setting on one side	396097	246661	37.7
transverse moment (kN·m) at bottom section of auxiliary pier	197225	136019	31.0
horizontal shear (kN) of the wind-resistant setting at pier top	5644	3032	46.3
longitudinal displacement (cm) at the top of pylon	56.0	24.3	56.6
longitudinal displacement (mm) of the deck	53.6	20.5	61.8

Table 4 Damping effect on the bending moment (kN·m) of main components in the south navigable span under the transverse excitation of the probability level P2

Location of the section	Before vibration reduction	After vibration reduction	Vibration reduction ratio (%)
transition pier TP1 with wind-resistant setting at the bottom section	345145	128214	62.9
bottom section of auxiliary pier	56583	51876	8.3
transition pier TP2 with wind-resistant setting at the bottom section	188620	107838	42.8
bottom section of pylon	251674	226995	9.8
bottom section of the upper pole on pylon	165440	167742	−1.4
top section of the upper sloped-pole on the pylon	174165	159905	8.2
deck section at pylon	492884	346949	29.6
mid-span section of the deck	243698	192347	21.1

Table 5 Damping effect on the displacements(cm) of main components in the south navigable span under the transverse excitation P2 when the wind-resistant setting becomes malfunctional

Location of components	Before vibration reduction	After vibration reduction	Vibration reduction ratio (%)
the deck at TP1	39.6	7.4	81.2
the deck at TP2	34.7	8.2	76.3
the deck at its mid-span	13.0	8.9	31.4
the pylon tip	7.9	7.9	0.0

The results of the vibration reduction analysis indicate that the longitudinal displacements of the deck and the longitudinal internal forces of pylons were greatly reduced when the longitudinal viscous dampers are installed on the north navigable span. In the transition and the auxiliary piers, however, the responses can not be reduced because there are no connection between the longitudinal dampers and the piers at these locations. The deck and pylons in the north navigable span experience the symmetric vibration about the bridge centerline under the longitudinal seismic excitation.

The longitudinal dampers can effectively control the structural resonance and suppress the vibration shortly after the excitation is over. As such, the probability of having local damage of the structure under high reverse stresses can be greatly reduced. The shear force applied on the wind-resistant setting and the internal forces of transition piers with wind-resistant settings in the north and south navigable spans are significantly reduced by the transverse damping system. The internal forces in auxiliary piers and pylons can be mitigated as well. The likelihood that expansion joints fail in shear becomes smaller as a result of the reduced lateral displacements of the deck after dampers have been installed on the transition piers even in the event of malfunctional wind-resistant setting.

The behavior of viscous dampers is velocity dependent. The higher the velocity of a damper, the more effective the damper since more energy can be dissipated over the duration of excitations. In the project studied here, the transverse wind-resistant settings were installed only on one side of the pylons and transition piers. This ensures the safety of expansion joints under normal operational conditions. However, the velocity differences between the main girder and the transition piers on the other side and between the main girder and the auxiliary piers are relatively smaller, resulting in less seismic response reduction in the transverse direction. Due to a higher longitudinal velocity at the deck of the north navigable span, the seismic response reduction system is more effective in the longitudinal direction.

5. Conclusions

Based on the extensive analyses, the following conclusions can be drawn:

1) Damping devices can augment the structural damping and thus dissipate the kinetic energy of the structure being controlled. This study only investigated one scheme of implementation of damping devices. Further analysis can be conducted to make their design more effectively so that the earthquake resistance of large-span bridges can be enhanced substantially.

2) For cable-stayed bridges, the internal forces and displacements of the deck and pylons under longitudinal seismic excitations can be significantly reduced by setting some energy-dissipated devices. In the transverse direction, the seismic responses of auxiliary and transition piers as well as pylons can be significantly reduced by utilizing damping devices. The seismic demand on main components of the structure can then be distributed more uniformly.
3) Vibration reduction devices can rapidly damp out the free vibration response of the structure shortly after the seismic excitation is over. Therefore, the likelihood of having the structure damaged locally under high reverse stresses becomes lower.
4) According to the structural property of cable-stayed bridges, vibration reduction systems can generally be designed separately in the longitudinal and transverse directions. If deemed necessary, they can be designed simultaneously in both directions. The coupled design, however, must take into account the compatibility of seismic displacements.
5) The parameters of a vibration reduction system must be optimized for better performance in response reduction. The main goal of vibration reduction design is to minimize the internal forces of main structural components, to make them distributed along the length of a bridge as uniformly as possible, and to reduce the displacements at key joints. At the same time, the damping forces required should be as small as possible for economic designs.

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