Multi Hazard Assessment of LongSpan Bridges in South Iceland

Final Report
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Abstract

This report describes multi-hazard considerations in vibrations of long-span bridges with focus on seismic and wind action. Such bridges, being flexible structures, are inherently vulnerable to wind loads. However, some response parameters of such bridges, for example tower and deck acceleration, can be significant during large earthquakes which produce ground motions with low frequency content. To illustrate this problem, a case study of the Runyang Suspension Bridge (RSB) is used. A finite element model of the bridge is created and verified against published literature. A set of ground motions from large worldwide earthquakes and spatially varying wind velocity time-series, simulated as a realization of a random field, are used for evaluating the dynamic response of the bridge with and without control devices. The control devices applied in this study are passive Tuned Mass Dampers (TMDs). Careful investigation of the uncontrolled response of the bridge shows that while wind load is mainly important for the displacement of the bridge deck, whereas seismic loads can induce significant acceleration of the tower and the deck. Since the response of the tower and the deck are coupled at some higher modes of vibration, seismic action, although most critical for the tower, is also relevant for deck acceleration. These observations indicate the need for a multi-performance-based control strategy. It is found that TMDs optimal for reducing seismic-induced deck acceleration can lead to amplification of wind-induced deck displacement. At the same time, TMDs optimal for reducing wind-induced displacement response are, in some cases, harmful for seismic-induced deck acceleration. These results clearly show multi-hazard interaction in control performance. To account for this problem, a control strategy for both seismic and wind response of the deck and tower is investigated. This consists of TMDs placed at the top of each of the towers and 4 TMDs placed on the deck. By tuning the TMDs to different vibration modes of the bridge, the system is shown to be effective for both seismic and wind action. The main results of this study are published in a scientific article which is provided in the appendix.

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1. Introduction

This report is a summary of research performed in a project funded by the Icelandic Road and Coastal Administration. The project was awarded a grant of 1 MISK in the year 2023. The objective of the project was to investigate multi-hazard considerations in long-span bridges with emphasis on the South Iceland environment. Research activity completed in the project is based on a long-span bridge in China, known as the Runyang Suspension Bridge (RSB). Due to a lack of published research in this area, the work envisioned took more time than expected. On the other hand, a more detailed literature survey and demonstration of multi-hazard effect, seismic and wind interaction, emerged as an important task before a specific case of South Iceland is investigated. This task has been successfully carried out, and the results have been published in a peer-reviewed international journal. The paper based on this work is included in Appendix 1 of this report.

2. Status of the project and future work

Despite some deviations from the original plan, the research work has been successful in producing valuable information and insights related to the research problem. It has clearly identified different considerations that need to be looked at when studying wind and seismic action on a long-span bridge. Identification of crucial response parameters, structural elements at risk, how different actions affect different response parameters, and design of suitable vibration reduction strategies has been thoroughly studied in this work with a case-study example of the Runyang Suspension Bridge. These results are valuable in performing further studies with environmental conditions specific to the South Iceland lowland. In this regard, a detailed study on the planned bridge over the Ölfus River in Selfoss is being carried out by the doctoral candidate supported by the grant. The results of this study will be presented in the doctoral dissertation of the candidate.

3. Finances

The grant was used to pay part of the salary of the doctoral candidate Abdul Matin Jami.

4. Conclusions

The project has produced valuable research that has been published in an international peer-reviewed journal. The publication is included in Appendix 1 of this report.

ACKNOWLEDGEMENTS

We acknowledge financial support from the Vegagerðin research grant which was used to finance part of the work reported here.

DISCLAIMER

The authors of the present report are responsible for its contents. The report and its findings should not be regarded as to reflect the Icelandic Road Authority's guidelines or policy, nor that of the respective author's institutions.

Appendix 1:

Jami et al. (2023) Multi-mode vibration control strategies of long-span bridges subjected to ulti-hazard: a case study of the Runyang Suspension Bridge. Journal of Vibration Engineering and Technologies, https://link.springer.com/article/10.1007/s42417-023-01157-3

Multi-mode vibration control strategies of long-span bridges subjected to multi-hazard: a case study of the Runyang Suspension Bridge

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Abstract

This study is an attempt to illustrate and discuss multi-hazard interactions in vibration control of long-span bridges subjected to wind and seismic loads. Such bridges, being flexible structures, are inherently vulnerable to wind loads. However, some response parameters of such bridges, for example tower and deck acceleration, can be significant during large earthquakes which produce ground motions with low frequency content. To illustrate this problem, a case study of the Runyang Suspension Bridge (RSB) is used. A finite element model of the bridge is created and verified against published literature. A set of ground motions from large worldwide earthquakes and spatially varying wind velocity time-series, simulated as a realization of a random field, are used for evaluating the dynamic response of the bridge with and without control devices. The control devices applied in this study are passive Tuned Mass Dampers (TMDs). Careful investigation of the uncontrolled response of the bridge shows that while wind load is mainly important for the displacement of the bridge deck, whereas seismic loads can induce significant acceleration of the tower and the deck. Since the response of the tower and the deck are coupled at some higher modes of vibration, seismic action, although most critical for the tower, is also relevant for deck acceleration. These observations indicate the need for a multi-performance-based control strategy. It is found that TMDs optimal for reducing seismic-induced deck acceleration can lead to amplification of wind-induced deck displacement. At the same time, TMDs optimal for reducing wind-induced displacement response are, in some cases, harmful for seismic-induced deck acceleration. These results clearly show multi-hazard interaction in control performance. To account for this problem, a control strategy for both seismic and wind response of the deck and tower is investigated. This consists of TMDs placed at the top of each of the towers and 4 TMDs placed on the deck. By tuning the TMDs to different vibration modes of the bridge, the system is shown to be effective for both seismic and wind action.

Keywords: Multi-Hazard, Long-span Bridge, Tall Building, TMD, Wind and Earthquake Engineering.

1. Introduction

Advances in science and technology make it increasingly feasible to build megastructures. Engineering ingenuity has made it possible to build many iconic structures around the world. Bridges across large crossing are not only of functional value but also important landmarks with aesthetic appeal showcasing engineering and architectural marvel. A few examples of such iconic bridge are the Golden Gate Bridge (San Francisco, USA), Çanakkale Bridge (Gelibolu-Turkey), Akashi Kaikyo Bridge (Kobe-Japan), Yangsigang Yangtze River Bridge (Wuhan-China), Great Belt Bridge (Korsør-Denmark), Humber Bridge (Hessle-United Kingdom), the Rio-Antirio Bridge (Patras, Greece), etc. The history of bridge construction goes back centuries. The Arkadiko Bridge is claimed to be the oldest existing bridge in the world, built between 1300 and 1190 BC, (Simpson, 1998). Functionality and complexity of bridges have grown with the development of transportation systems to the extent that today long-span bridges are considered one of its essential components.

The geometry of long-span bridges makes them vulnerable to dynamic forces induced by natural phenomena such as earthquakes and wind. The probability of simultaneous occurrence of large wind and seismic action on long-span bridges is low (see, for example, Padgett and Kameshwar, 2016). However, during their useful life, such bridges may be exposed to these forces at different times with intensities that might pose threat to their safety and functionality. Such bridges therefore need to be robust against different hazards relevant at their sites. Multi-hazard consideration also becomes important because different natural forces might affect these bridges in different ways (see, Jami et al., 2022, and references therein). Recently, Marić et al. (2022) discussed multi-hazard considerations in cost, safety, reliability, and serviceability of long-span bridges.

Vibration suppression and energy dissipation methods have proven useful to enhance functionality and safety of large structures, including bridges (Jangid, 2022; Elias et al., 2021; Cetin et al., 2019; Pisal and Jangid, 2016; Aly 2014; Saha and Jangid, 2009; and Soneji and Jangid, 2007). Such methods include base isolation, supplemental damping, and various types of passive/active/hybrid tuned dampers. Passive tuned mass dampers (TMDs) are one of the most common vibration control devices investigated in the literature. Several studies have confirmed their effectiveness is reducing vibrations of bridges. The Meiko Bridge, the Akashi Kaikyo Bridge, and the Trans-Tokyo Bay Bridge in Japan, the Kessock Bridge in the UK, and the Jindo Bridge in Korea are examples of long span bridges where TMD is used as control devices (Fujino 2002 and Fujino and Siringoringo, 2013).

Several studies have investigated vibration control of long-span bridges. Xing et al. (2014) studied the efficiency of TMDs in wind-induced vibration control of a long-span bridge. They report that a TMD-type counterweight placed at the mid-span of Sutong cable-stayed bridge is effective in reducing wind-induced vibrations of the deck. Chen et al., (2003) present a procedure to optimize TMD parameters for multi-mode buffeting response control of long-span bridges. Wang et al. (2010) presented a study on the optimal placement of dampers to control seismic vibrations of the Runyang Suspension Bridge. Experimental and theoretical studies on the performance of TMD in controlling wind-induced vibration of bridges is presented in Gu et al. (1998) and Cai and Chen (2004). Analytical studies using TMD-inerter to mitigate vortex-induced vibration of long-span bridges are reported in Xu et al. (2019), Xu et al. (2020) and Xu et al. (2022). Gu et al. (2002) and Tanida (2002) studied the application of active and semi-active TMDs for wind-induced vibration control of long-span bridges. Soneji and Jangid (2006), present effectiveness of elastomeric and sliding types of isolation systems for the seismic response control of cable-stayed bridges. Shum et al. (2008) present wind-induced vibration control of long-span bridges using multiply pressurized tuned liquid column dampers. Zhu et al. (2016) studied the effectiveness of Fluid Viscous Damper (FVD) for mitigating seismic-induced vibrations of a cable-stayed bridge. Ha et al. (2010) present a methodology for optimization of complex damper parameter for seismic response reduction of long-span bridges. Use of TMDs for controlling vibration of long-span bridges due to vertical component of ground motion is studied by Lavasani et al. (2020) and Pourzeynali and Estaki (2009).

Use of passive TMDs for vibration control of long-span bridges has so far been mostly concentrated on wind load. Wind-induced vibrations are, in most cases, the most critical consideration for long-span bridges. However, seismic excitations can also pose a threat to different components of such bridges, for example, the towers of suspension bridges. Wind and earthquake forces excite different vibration modes of long-span bridges. Wind loads are in general critical for displacement response of the deck, while seismic loads may excite vibration modes of the towers. For an overall performance enhancement of long-span bridges subjected to seismic and wind forces, vibration control strategies can be varied depending on the type of excitation, the structural component that is the most affected, and the response parameter to be controlled, for example, displacement or acceleration. As TMDs need to be tuned to certain vibration frequencies of the structure, for a complex structure excited by forces with drastically different frequency content, a multi-mode multi-performance strategy is necessary. Investigation of such scenarios for long-span bridges is lacking in the literature. This work is an attempt to shed light on important considerations in multi-performance, multi-hazard vibration control of long span bridges using a case study of the RSB. The effect of ground motion and wind-induced force on the super long-span bridge's two main parts (tower and deck) are evaluated, and implications of multi-hazard interactions in multi-performance control are investigated using various control strategies.

In the following, a summary of vibration mechanisms of long-span bridges is provided. It is followed by a case study of the RSB. The mathematical models of the bridge and the control devices are presented. Then the wind and ground motion excitation used for evaluating the performance of control devices are presented. Different vibration control strategies with TMD placement and tuning are investigated for multi-performance control. Vibration control performance of these strategies is evaluated and discussed in detail, and the main findings are summarized.

2. Vibration mechanisms of long-span bridges

Vibrations can have adverse effects on long-span bridges, from bridge elements' fatigue to collapse under extreme cases. In addition, excessive vibration reduces the serviceability of bridges. Due to the unique dynamic characteristics of long-span bridges, namely low damping and high flexibility, they are susceptible to vibrations induced by many dynamic processes. A summary of different vibration mechanisms of long-span bridges is presented in the following.

2.1. Seismic-induced vibration

Ground acceleration causes vibration of different components of a bridge. The energy carried by earthquake ground motions usually has a higher frequency content than the fundamental natural frequencies of the deck vibrations. In general, displacement transmissibility is higher than acceleration transmissibility. Other elements of the bridge, for example the towers, are stiffer and might be excited more than the deck by seismic ground motions.

Many big cities and their transportation routes, which include long-span bridges, have been built in earthquake-prone areas. However, very few have so far been damaged by earthquakes. For example, the Ji Da cable-stayed bridge in Taiwan was damaged by the Chi-Chi earthquake in 1999; Higashi-Kobe cable-stayed bridge was damaged during the Kobe earthquake in 1995. In general, the design of sub-structures and pylons mainly focuses on seismic load as the potential dynamic load, while wind load governs the design of the cables and girder (Fujino and Siringoringo, 2013).

2.2.Wind-induced vibration

Wind-induced force is a random dynamic process, and its effects on structures varies in space and time. In the design of long-span bridges, among various types of dynamic forces, wind-induced vibration is considered the most critical dynamic excitation. Oscillating components of a long-span bridge, such as towers, deck, and cables, shape the wind flow around the bluff body of the bridge result in a complex fluid-structure interaction. This interaction is studied under the title of resulting aerodynamic force and self-excited vibration. Wind-induced vibration processes are schematically summarized in Figure 1.

Cables are the most flexible component of long-span bridges (cable-stay bridges and suspension bridges). Low damping and high flexibility make cables susceptible to vibrations. Cable fatigue and damage to the anchorage are the most common problems caused by cable oscillations.

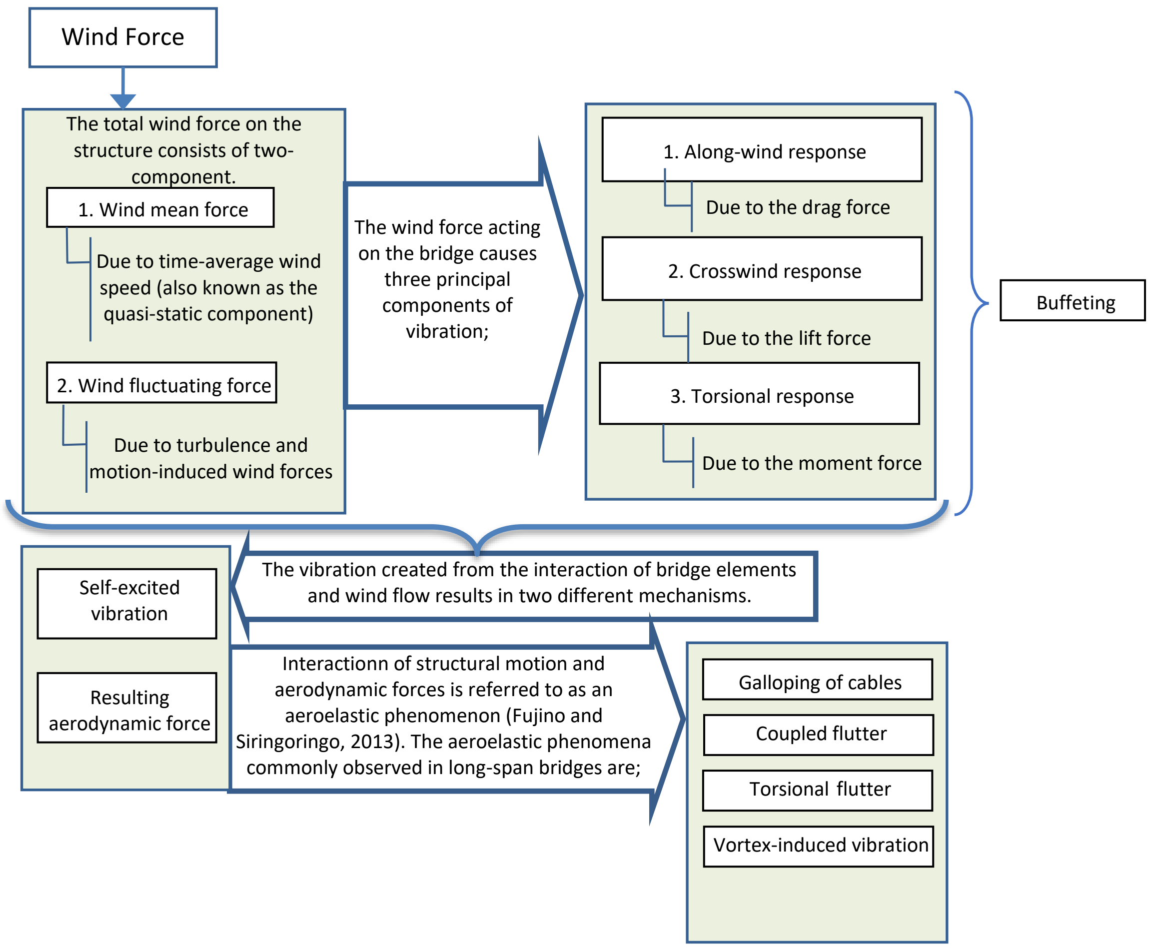

Figure 1 Wind-induced vibration mechanisms (based on Fujino and Siringoringo, 2013).

2.3. Traffic-induced vibration

Traffic-induced vibration is a common phenomenon that has been widely studied for long-span bridges. For example, Paultre et al. (1992) presented a comprehensive review of traffic-induced vibration mechanisms. In design codes, moving traffic loads are considered through a dynamic amplification factor. This factor is based on bridge characteristics such as the geometry of the bridge, its natural frequencies, joints and support conditions, and vehicle characteristics such its dynamic specification, speed, weight, and pavement conditions.

One of the concerns for long-span bridges serving train traffic is the vehicle-structure interaction. The large moving load of the train passing at high speed along a long-span bridge produces forces and vibrations that can cause several problems such as train serviceability, passenger comfort and safety, fatigue of the bridge elements, and even resonance under certain circumstances.

2.4. Human-induced vibration

Events such as rallies or marathons passing a bridge can form human-induced vibrations. In fact, there have been many reports regarding human-induced vibration in bridges, especially footbridges. In some cases, human-induced vibrations have caused collapse, killed people, and resulted in economic loss. The Angers Bridge collapse in 1850 is an example of a bridge collapse due to human-induced vibration. (Arioli and Gazzola, 2013). Many people walking in step, generate periodic forces in horizontal and vertical directions, resulting in bridge vibrations. Therefore, resonance is a potential hazard for the bridge with natural frequencies close to human-induced vibration frequencies, which are considered in some design codes, as 2 Hz in the vertical direction and 1 Hz in the horizontal direction (Fujino and Siringoringo, 2013).

3. Case study of the Runyang Suspension Bridge (RSB)

This section provides a case study of the RSB to illustrate how wind and earthquake forces affect the bridge and its different components differently, and what implications it has in selecting a proper vibration control strategy. The mathematical model of the bridge, the wind and seismic excitations used in the study, and the control systems being considered are described in the following sections.

3.1. Mathematical models

The RSB crosses the Yangtze River in Jiangsu Province, China. The RSB is a super long-span suspension bridge with a main span of 1490 m (see Figure 2). The main span of this bridge is suspended between two towers. Each tower is constructed from two steel-concrete columns with a height of 210 meters and three prestressed concrete cross beams (see Fig. 2-(a) & (b)). The main cables are 0.9 m in diameter, and each consists of 184 strands. Each strand contains 127 wires, 5.3 mm in diameter, made of steel with 1670 MPa yield strength. Each vertical suspender contains 109 steel wires of 5 mm diameter and 1670 MPa yield strength. The steel girder is 38.7 m wide and 3 m high, with a 2% slope on each side (see Fig. 2-c). The main components of the deck are plates, longitudinal beams (U-ribs), and transverse beams (transverse diaphragms). The thickness of each part is: 10 mm for the lower inclined web, 12 mm for the upper inclined web, 6 mm for longitudinal beams, 8 mm for transverse beams, and 14 mm for the deck. The distance between transverse beams is 3.22 m, (see Ji et al., 2006). A Finite Element Model (FEM) of the bridge was created in the computer program SAP2000. The model is based on specifications of the bridge available in the literature (Ji et al., 2006; Li et al., 2010). Both the bridge and control device are assumed to remain in the elastic range. Three-dimensional frame elements are used to model the towers and the girder. Cable elements are used to model the main cables and the vertical suspenders. Connection between the girder and the suspender cables is assumed to be rigid. Masses of frame elements are lumped at their end nodes. Mass of non-structural elements such as wearing course, parapet walls, etc., are added to the model. Main cable anchors, abutments, and piers are assumed to be fixed in the foundation, i.e., soil-structure interaction is not considered.

The FEM of the RSB with a schematic representation of TMDs placed at different locations is shown in Figure 3. The modal frequencies and mass participation ratios of the first 10 modes of vibration of the RSB are presented in Table 1. The first horizontal symmetric mode corresponds to the transverse movement of the deck. The fifth mode is dominated by the movement of the tower in the X direction. The shapes of the first horizonal symmetric mode and the fifth mode are shown in Figure 4. We focus on these two modes because they contribute the most to the transverse response of the deck and the tower respectively.

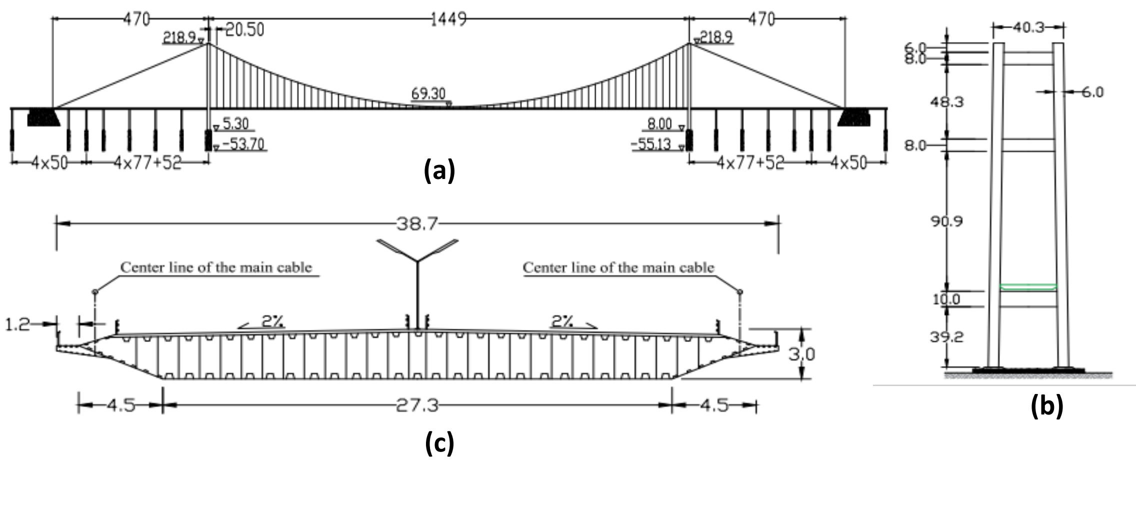

Figure 2. The RSB (a) elevation. (b) Geometry of the tower. (c) Cross-section girder. (Dimensions in m)

The modal properties of the RSB bridge have been published by Wang et al. (2010), which presented a finite element model updating of the RSB using ambient vibration measurement data. The first horizontal symmetric mode of their FEM has a frequency of 0.054Hz. Based on ambient vibration test, they identified the frequency of the mode to be 0.059Hz. The frequency of the corresponding mode of our model is 0.05Hz, which is close to the published results. Li et al. (2010) report similar frequencies of the first tower mode, i.e., 0.05Hz based on FEM and 0.051 based on ambient measurements. The frequencies of the first vertical mode of the bridge based on FEM model of Li et al. (2010) is 0.09Hz and that of our model is also 0.09Hz.

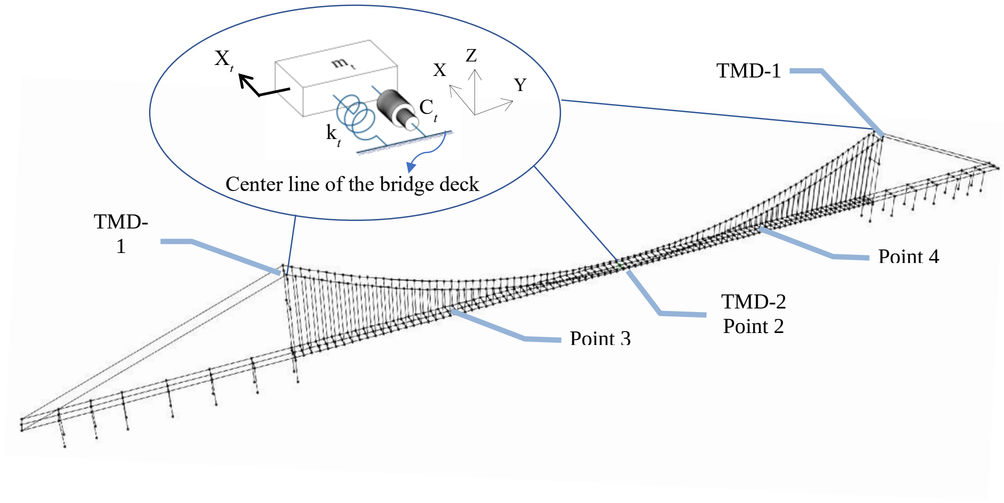

Figure 3. A FE model of the RSB showing the locations of the TMDs investigated in subsequent sections. The TMDs move only in the transverse direction of the bridge.

Table 1. Modal frequencies, and modal participation mass ratio of RSB for modes 1 and 5 (U and R denote translation and rotation, respectively)

Mode #	Direction	Frequency (Hz)	Modal mass participation ratio						Location of max displacement
			Uy	Ux	Uz	Ry	Rx	Rz	
1	Transverse	0.0497	0	0.089	8.00E-11	0.00389	9.82E-20	0	Point 2
2	Vertical	0.0884	0.00017	3.77E-17	1.40E-20	4.21E-17	0.01231	2.99E-08	Points 3 and 4
3	Transverse	0.1365	8.67E-08	1.99E-18	2.92E-16	7.81E-19	1.59E-09	0.0207	Points 3 and 4
4	Vertical	0.1459	1.01E-19	8.23E-08	0.00414	1.97E-08	3.45E-20	7.78E-16	Points 2, 3 and 4
5	Transverse	0.2307	3.01E-14	0.41291	1.17E-07	0.8276	4.31E-13	6.31E-13	Point 1
6	Transverse	0.2313	3.69E-07	7.82E-13	3.94E-12	1.49E-12	9.11E-10	0.33106	Point 1
7	Vertical	0.2499	6.56E-17	2.91E-07	0.07077	7.84E-07	3.37E-17	1.06E-11	Points 3 and 4
8	Transverse	0.2669	2.29E-14	0.00209	2.56E-07	0.02602	6.94E-14	5.10E-15	Points 2, 3 and 4
9	Vertical	0.2681	0.00256	5.67E-14	5.60E-17	1.39E-12	0.00482	1.17E-08	Four points on the deck
10	Vertical	0.3914	1.85E-15	7.94E-07	0.01075	5.09E-07	1.59E-17	1.23E-13	Five points on the deck
11	Transverse	0.4367	2.20E-05	3.59E-14	4.48E-16	1.88E-14	8.21E-08	0.00487	Points 3 and 4
12	Torsion	0.4405	2.98E-13	5.80E-05	1.62E-08	0.00304	1.82E-13	2.50E-17	Point 2
13	Longitudinal	0.4785	0.43752	1.05E-14	4.44E-15	1.64E-12	0.00101	1.82E-05	Point 1

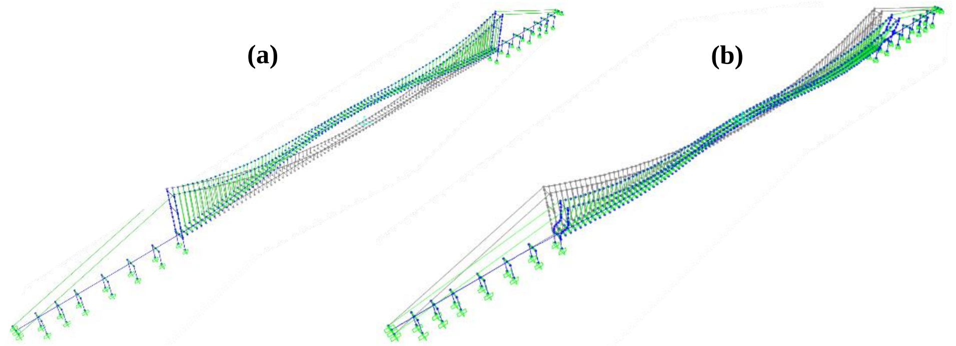

Figure 4. Mode shapes of the RSB, (a) Mode-1, the first horizontal symmetric mode, (b) Mode-5, the first tower transverse mode,

3.2. Wind and seismic loads

Linear time history analysis with earthquake ground motion and wind forces are used to evaluate the performance of several vibration control schemes for the case of the RSB bridge. Details of ground motion and wind forces used for simulating the response of the bridge with and without control devices are given in the following. It is highlighted that this work is not a site-specific study but a general investigation of multi-performance vibration control of long-span bridges. Therefore, the seismic ground motion and wind force properties are not representative of the actual hazard at the RSB bridge site. Both the seismic and wind action are only considered in the transverse direction of the bridge.

3.2.1. Ground motions

In selecting ground motions, large earthquakes are given priority, as ground shaking from large earthquakes is richer in low frequency content. By inspecting Fourier Amplitude Spectra of hundreds of ground motions from the PEER database (Ancheta et al., 2013), 10 ground motions which had significant energy near the important vibration modes of the bridge were selected. Ground shaking is considered in the transverse direction of the bridge only. Some details of the selected ground motions are presented in Table 2. The ground acceleration time histories of the selected motions are shown in Figure 5. The corresponding Fourier Amplitude Spectra (FAS) shown in Figure 6 indicates that even for such large earthquakes, the spectral content at the first mode of the deck is insignificant. However, the ground motions contain significant energy at the tower vibration modes. Long-span structures such as the one studied here experience differential motion at their supports during earthquakes. The differential motion is a result of wave passage effect and random variability quantified by lagged coherency (see, for example, Zerva, 2009). Lagged coherency is a measure of similarity between motion at different measurement points. As the distance between the measurement points increases, the motions become more dissimilar and lagged coherency decreases. It also decreases with increasing frequency. Low frequency motions are more coherent than high frequency motion. Based on the Smart 1 array data from Taiwan, Rupakhety and Sigbjörnsson (2012) report that up to ~2Hz, lagged coherency at 200 m separation distance is very close to 1. Zerva (2009) reports similar results for a separation distance of 1000 m. Based on a case-study of the El-Esnam Earthquake ground motion simulation, AfifChaouch et al. (2016) report that at frequencies lower than 0.5 Hz, the lagged coherency is very close to 1 even for large separation distances. Based on these considerations, it is reasonable to ignore the effect of lagged coherency in the seismic response of the RSB because the frequencies of its modes that contribute to seismic response are well below 1Hz. To account for wave passage effect, multi-support excitation was considered. Ground motion at one side of the main span was shifted in time by 0.6s with respect to that at the other side. This is based on assumed apparent wave propagation velocity of 2.0 km/s and main span length of 1.5 km. The difference in peak response of the tower with uniform support excitation and multi-support excitation was found to be insignificant (less than 5%). Based on these results, it was decided to use uniform support excitation for evaluation of the control performance.

Table 2, Ground motion time series used in this study (PGA =Peak Ground Acceleration, R_{jb} = Joyner-Boore distance)

No.	Earthquake Name	Year	Station Name	Magnitude	Component	PGA (m/sec2)	R_{jb} (km)
1	Tabas Iran	1978	Tabas	7.35	N16^{0}W	8.45	1.79
2	Imperial Valley-06	1979	El Centro Array #4	6.53	N40^{0}W	3.63	4.9
3	Imperial Valley-06	1979	El Centro Array #7	6.53	N40^{0}W	4.6	0.56
4	Chi-Chi Taiwan	1999	CHY101	7.62	E-W	3.9	9.94
5	Chi-Chi Taiwan	1999	TCU029	7.62	E-W	1.95	28.04
6	Chi-Chi Taiwan	1999	TCU031	7.62	E-W	1.22	30.17
7	Chi-Chi Taiwan	1999	TCU036	7.62	E-W	1.22	19.83
8	Chi-Chi Taiwan	1999	TCU039	7.62	E-W	1.36	19.89
9	Chi-Chi Taiwan	1999	TCU087	7.62	E-W	1.12	6.98
10	Chi-Chi Taiwan	1999	TCU128	7.62	E-W	1.63	13.13

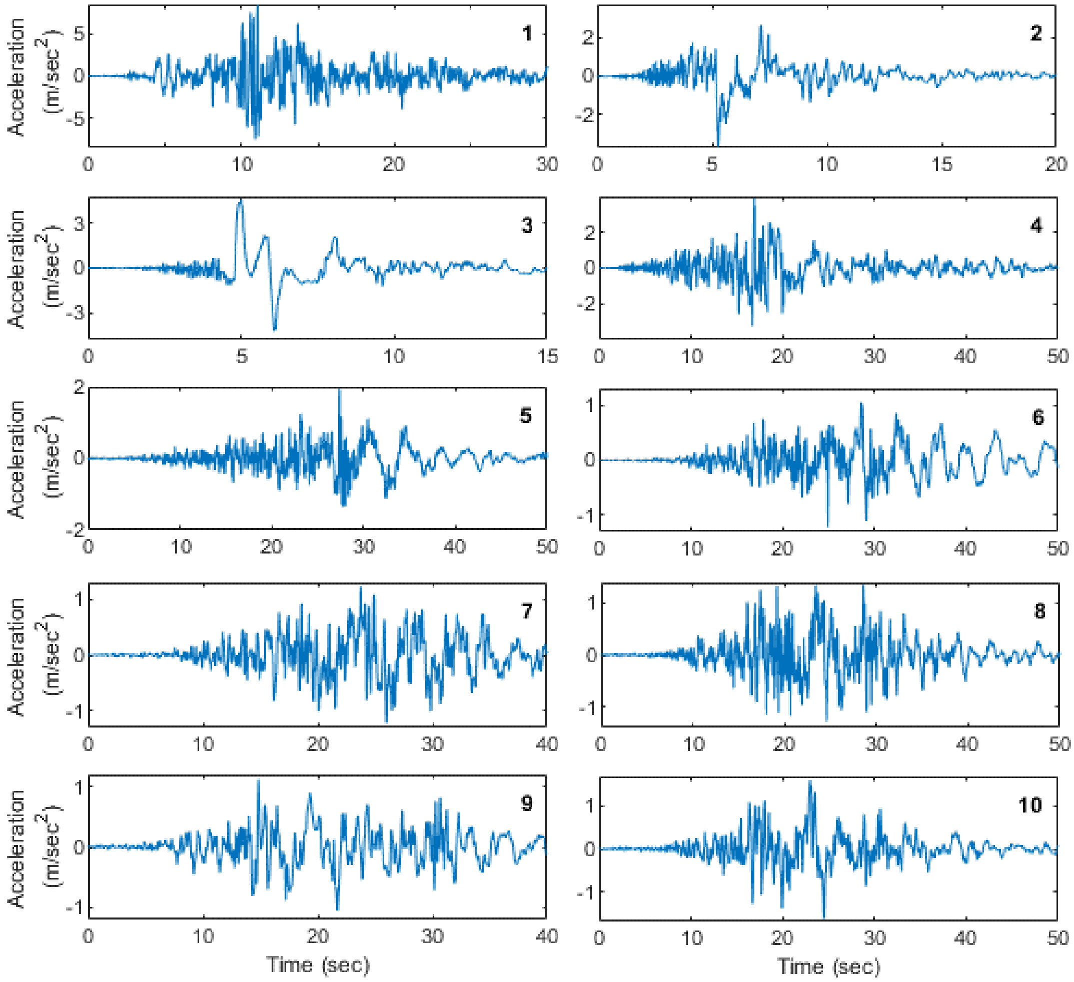

Figure 5. Time histories of the ground accelerations listed in Table 2.

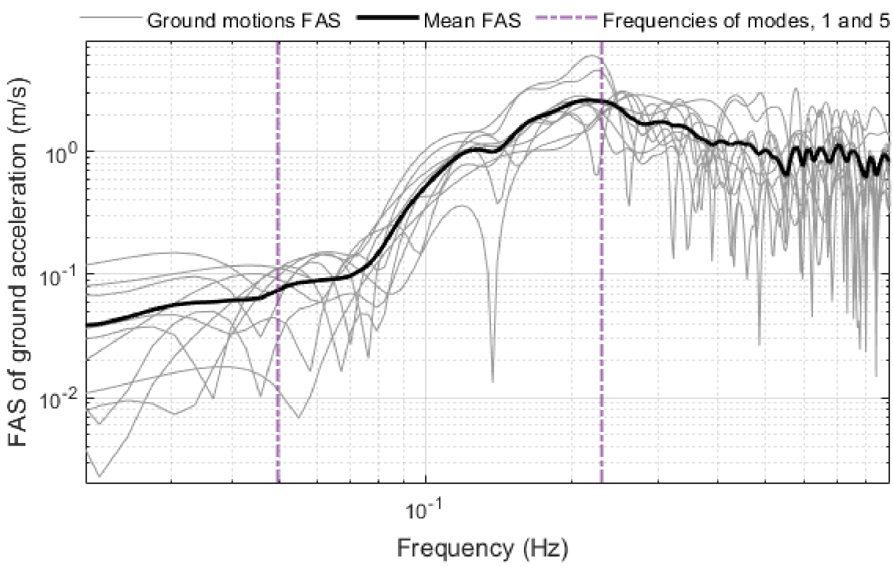

Figure 6. Fourier amplitude spectra (FAS) of the selected horizontal ground motion time series shown in Figure 5. The grey lines correspond to individual ground motions and the black line is the average FAS. The vertical lines correspond to the frequencies of the two modes listed in Table 1.

3.2.2. Wind-force

Wind force time series is applied on the deck and tower nodes. Turbulent wind field in 2 space dimensions is modelled as a random process. Realizations of the random process are simulated using the spectral representation method of Shinozuka and Deodatis (1991). Spatially correlated wind field is simulated in a 2-D grid. The simulation is carried out using a toolbox provided by Cheynet (2020). The wind parameters used in the simulation are: friction velocity, u* = 2.2 m/sec (Wang et al., 2016), Von Karman constant = 0.4, roughness length z_0= 0.03m, sampling interval Δ_t = 0.25s, and the co-coherence function decay coefficients, C_{ij}, correspond to velocity component *i* (*u*, *v*, *w* in *x*, *y* and *z* directions, respectively) and space dimension *j*. These coefficients are selected based on N400 (Wang et al., 2018), i.e., C_{uy}= C_{uz}= 10; C_{vy}= C_{vz}= C_{wy}= 6.5; and C_{wz}= 3.

The fluctuating wind forces are applied at several nodes in the tower and the deck. The force at a node is related to the wind velocity at the node as

$$F(t) = \rho C_D A U u(t) \tag{1}$$

where ρ is the density of air (taken as 1.2 kg/m^3), A is the tributary area, U is the average wind velocity, $u(t)$ is the fluctuating wind velocity, and the drag coefficient C_D is taken as 1.2. The wind forces are calculated at 80.1 m separation along the length of the deck and 18 m along the height of the tower. It was found that a finer resolution in simulating the wind field did not have significant effect on the simulated response. Figures 7 and 8 show a realization of the wind velocity and its power spectrum at the centre of the deck. Figure 9 shows the co-coherence of wind velocity in horizontal and vertical separations.

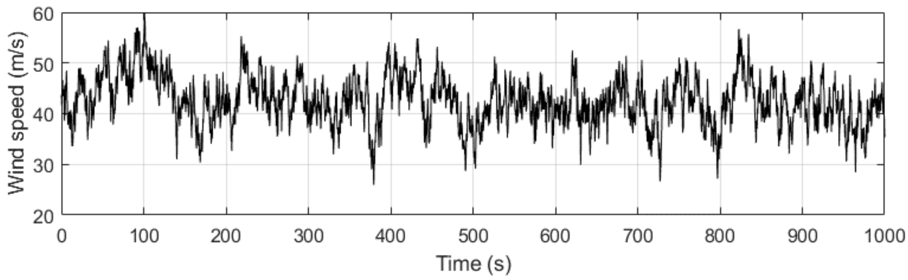

Figure 7. A sample time history of along-wind velocity at the centre of the deck.

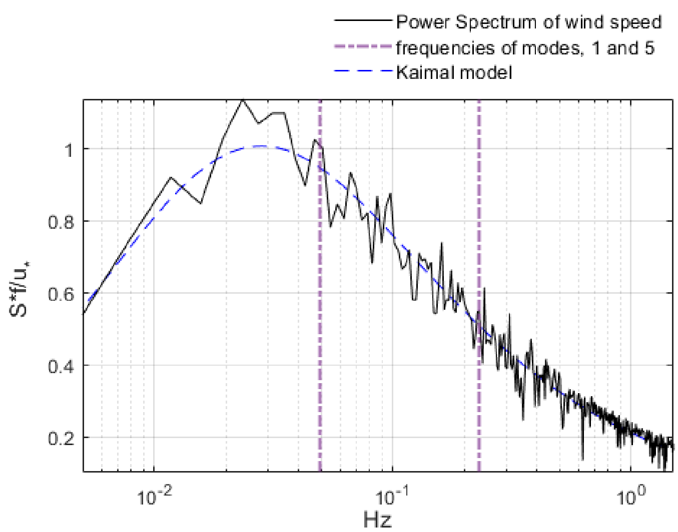

Figure 8. The power spectrum of wind speed at the centre pf the deck (see Kaimal et al., 1972).

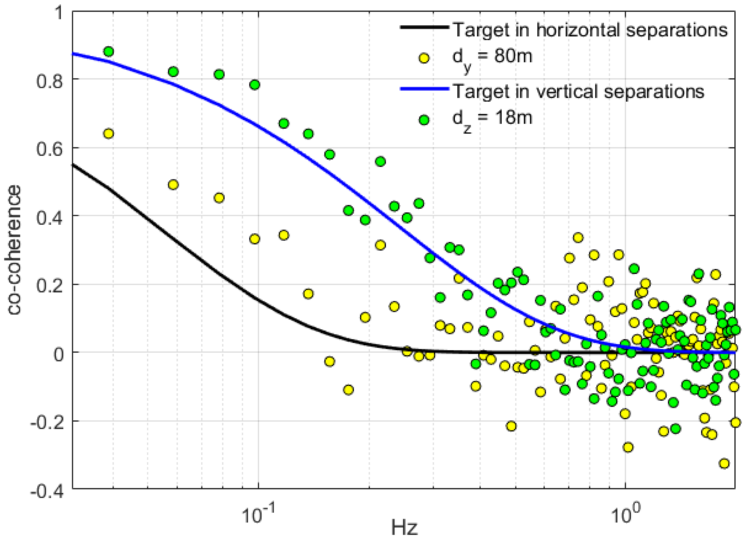

Figure 9, Co-Coherence of wind velocity in horizontal and vertical separations.

3.3. Uncontrolled response

The response of the bridge to ground motion and wind force was simulated using time history analysis. Wind forces and seismic loads are applied separately. Table 3 lists the peak displacement and acceleration response of the top of the south tower (Point 1) and the deck at mid-span (Point 2) for different ground motions and wind load. The effect of wind force on the displacement and acceleration response of the tower is negligible. Wind force is primarily relevant in terms of displacement response of the mid-span of the deck (Point 2). Seismic load is more relevant for the tower, both in terms of displacement and acceleration response. For deck acceleration, seismic load is much more relevant than wind load. Displacement response of the deck induced by ground motion is often much lower than that to wind load, but significant in some cases like ground motion 1 and 4. The peak displacement response of the deck induced by wind force is found to be 3.29 m. For the same bridge, Zhang (2007) report, from aero-static analysis, peak mid-span displacement of ~2.5 m at a wind speed of 45 m/s. For the same mean wind speed our results are comparable although slightly higher than those of Zhang (2007), which can be attributed to differences in wind loading parameters such as drag coefficient, and the turbulence components in dynamic analysis. The peak wind speed at the deck mid-span in our analysis reaches 60 m/s, for which the aero-static mid-span displacement reported in Zhang (2007) for spatially varying wind load is ~3.3 m, which is very close to our result. We performed response analysis with spatially uniform wind field and found that the peak response is significantly higher than for the case of a spatially variable wind field. Similar observations are reported in Zhang (2007) for the same bridge.

Figure 10 presents the FAS of some of the key response parameters, i.e., displacement and acceleration at Points 1 and 2. For seismic load, displacement and acceleration at Point 1 and acceleration at Point 2 are presented, whereas for wind load only the displacement at Point 2 is presented. The energy in the wind force is concentrated at much lower frequencies than that in the seismic force. Wind force excites the first transverse mode but has limited energy at the vibration modes of the tower (see figure 8). For this reason, the wind force is primarily important for displacement (buffeting) response of the deck. Seismic motion, on the other hand, has significant energy at frequency corresponding to mode 5 (see Fig. 6). This is reflected in the FAS of the tower-top response shown in Figure 10. Mode 5, which is the first mode of the tower, contributes significantly to the acceleration response of the tower. The tower top displacement is also primarily due to mode 5. The fifth mode, although primarily related to the tower, also includes significant motion of the deck mid-span. Since the seismic load has significant energy at the frequency of this mode, it contributes to acceleration response of the deck mid-span. The contribution of the first mode in the overall acceleration response of the deck to seismic excitation is much smaller than that of the fifth mode.

Table 3, Peak displacement and acceleration responses of tower (Point 1) and deck mid-span (Point 2).

Response	Location	Earthquake ground motion time history no.										Wind force
		1	2	3	4	5	6	7	8	9	10	
Acc. (m/sec2)	Point-1 (Tower)	5.44	3.01	3.36	4.50	3.17	2.60	2.49	2.17	2.23	2.67	0.02
	Point-2 (Deck)	1.97	1.64	1.59	1.51	1.32	0.89	0.98	1.04	1.05	1.29	0.28
Displ. (m)	Point-1 (Tower)	2.27	1.43	1.38	2.03	1.50	1.13	1.13	0.90	0.91	1.27	0.02
	Point-2 (Deck)	1.95	1.15	0.88	1.31	0.84	0.56	0.78	0.88	0.70	0.81	3.29

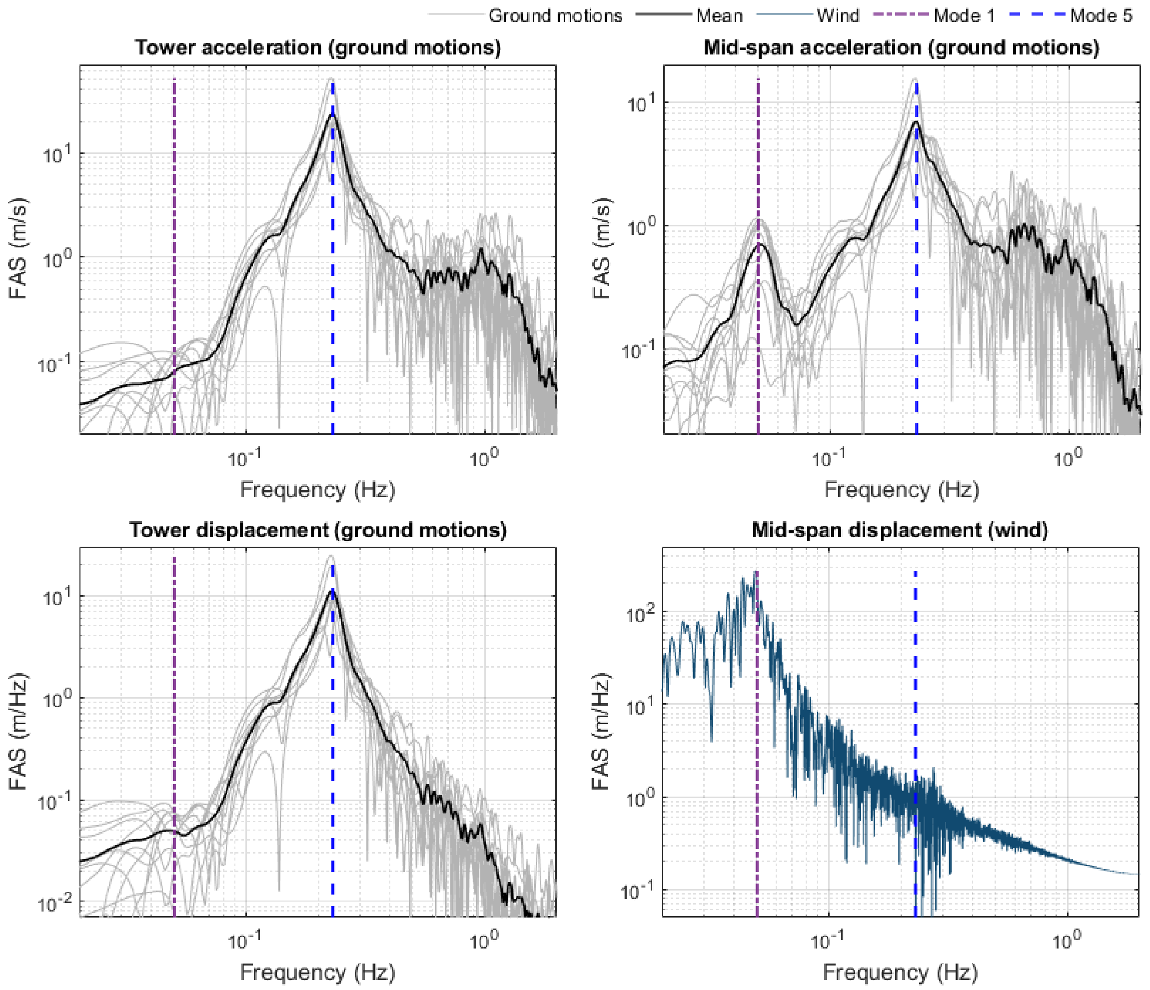

Figure 10. FSA of tower and mid-span acceleration and displacement response in transverse direction due to wind-induced force and ground motions.

3.4. Control strategies

For multi-performance vibration control of the bridge response, passive tuned mass damper (TMD) systems are considered. The TMDs consists of a mass attached to the structure with an elastic spring and a viscous damper. It is schematically shown in Figure 3. TMDs reduce the vibration of the structural modes they are tuned to by generating resisting inertia forces. The viscous dampers in the TMDs help to reduce the displacement of the TMD mass. Once the mass of the TMD is assigned, its stiffness and damping coefficient can be tuned for optimal vibration control. For a single TMD, the tuning is usually done to the most dominant mode of vibration on the structure. This mode, in many cases, is the fundamental mode. Multiple TMDs tuned to different modes are desirable when multiple modes of vibration contribute to the response.

In a multi-hazard scenario, the tuning of TMDs is not straightforward. Tuning in such cases depends on factors such as (i) the frequency content of excitation as it dictates which vibration mode is dominant in the response (ii) the structural component whose response is being controlled, and (iii) the structural response to be controlled, i.e., acceleration, displacement, etc. Factor (i) is related to the nature of the hazard, whereas factors (ii) and (iii) are linked to the location and characteristics of the TMD.

Different types of optimization methods can be used to tune TMDs for different performance objectives. One of the most used tuning methods for single TMDs is based on the Den-Hartog's formulation (Den Hartog, 1956). This formulation has been adopted in this study for wind-induced vibration control. For base excited structures, such as those under seismic action, the formulation of Sadek et al. (1997) is more appropriate and is therefore adopted in this study. The TMDs in this study are intended for reducing vibrations in the transverse directions of the bridge. They are therefore assumed to be fixed in the longitudinal and vertical directions. They are modelled as lumped masses attached to the structure with a 2-node link element. Apart from different formulations for optimizing TMD parameters for seismic and wind actions, the properties of the TMDs vary based on which vibration mode they are tuned to.

Based on the analysis of the uncontrolled response of the bridge to ground motions and wind forces, performance objectives are identified as reduction in (i) wind-induced displacement of the deck (ii) seismic induced acceleration of the towers (iii) seismic induced acceleration of the deck. Different combinations of TMD tuning and placements are investigated to achieve these objectives. The combinations and optimal TMD parameters are described below.

Case 1.

The performance objective is reduction in seismic response of the towers. Two TMDs, one at the top of each tower, is used in this case. The TMDs are named as TMD-1, with 1 indicating its location, i.e., the top of the tower. These TMDs are tuned to the fifth mode of the bridge, which is the first mode of the towers. Each TMD has a mass equal to 1% of the total mass of the bridge. The frequency and damping ratio of TMD-1 are 0.23Hz and 6%, respectively.

Case 2.

The performance objective is reduction in wind-induced displacement of the deck mid-span. The TMD is tuned to the first vibration mode of the bridge. The mass of the TMD is 1% of the total mass of the bridge. This TMD is named as TMD-2, with 2 indicating its location, i.e., the mid-span of the deck. The frequency of the TMD is 0.049 Hz, and its damping ratio is 6%.

Case 3.

The performance objective is reduction in seismic response of the deck. Since the seismic response is dominated by the fifth mode of vibration, the TMD is tuned to this mode. The TMD is placed at the mid-span of the deck and is named as TMD-2. Its mass is 1% of the total mass of bridge. The frequency and damping ratio are 0.23Hz and 6% respectively, i.e., the same as in Case 1.

Case 4

Here, the control strategy is multi-hazard and multi-performance. Two sub-cases are investigated. Case 4a is combination of Case 1 and 2, i.e., to simultaneously control seismic response of the tower and wind-induced displacement of the deck. Case 4b is a combination of Case 1 and Case 3, with the objective of simultaneously controlling seismic response of the tower and the deck. To achieve this, TMDs are placed at the top of the towers and the mid-span of the deck.

3.5. Control effectiveness

The effectiveness of the control devices is measured as percentage reduction in response. Table 5 presents a summary of control effectiveness of the different cases described above. For seismic response, maximum, minimum, and average reduction are presented. Case 1 results in reduction of tower acceleration by ~11-41%, with an average value of 24%. Reduction in tower displacement is also good, but slightly lower than that of tower acceleration. A single TMD placed at deck mid-span and tuned to the first mode of the bridge, i.e., Case 2 reduces the peak wind-induced acceleration and displacement by ~36% and 21%, respectively. Case 3 is effective in some cases, but results in amplification of deck acceleration during some of the ground motions. Case 4a is effective in seismic response control of the tower and wind response control of the deck, but not desirable for seismic response control of the deck. On the other hand, Case 4b is effective in controlling the response of the deck under most of the ground motions, but it greatly amplifies the wind-induced displacement. Comparison of Cases 4a and 4b clearly show that a control system which is designed for seismic control of the deck amplifies wind response and vice versa. This is an important observation highlighting the multi-hazard interaction in vibration control of long-span bridges. If we consider that wind-induced displacement of the deck is more important to control than seismic-induced acceleration, we can conclude that Case 4a is the best solution.

In terms of acceleration of the deck, seismic response is much higher than the wind response. If we assume that both seismic and wind loads are relevant for the structure, although not acting simultaneously with high intensity, reducing seismic-induced acceleration of the deck might be important. But this reduction comes at a cost of amplified wind-induced displacement, which is clearly undesirable.

To overcome this problem and provide response reduction in multi-hazard scenario, we investigated an additional strategy which is called Case 5. Case 5 makes use of the TMDs as in Case 4a with three additional TMDs distributed along the deck. These additional TMDs are tuned to the fifth mode of vibration of the bridge because the seismic response of the deck is governed by this mode. The mass of each TMD is 0.33% of the total bridge mass and damping ratio is 3.5%. The location of these TMDs is decided based on the deflected shape of the deck in this mode, which is shown in Figure 4b. The deck deflects with two nodes and three peaks in the transverse direction. One of the peaks is located near the mid span, and the other two on either side of it. The distributed TMDs are therefore placed at quarter, half, and three quarter the length of the deck. The results corresponding to this case are also shown in Table 5. This case provides good reduction of deck response due to wind load. Unlike Case 4a, seismic response of the deck is also effectively controlled with only a minor amplification of acceleration response for one of the ground motions. It provides similar levels of seismic response reduction of the deck as Case 4b, but without amplification of the wind response. The reduction of seismic response of the towers is similar as in Case 1.

Table 4. Response reduction (%) effectiveness of different cases of TMDs (negative numbers indicate amplification of response).

Case	Hazard	Performance		Ground motion			Wind
				Ave	Max	Min	
C-1	Earthquake	Towers	Accel.	24.00	40.99	11.26	
			Displ.	18.21	39.02	6.75	
C-2	Wind	Mid-span	Accel.				35.82
			Displ.				20.65
C-3	Earthquake	Mid-span	Accel.	20.52	39.14	-3.69	
			Displ.	15.25	23.34	6.77	
C-4a	Earthquake and Wind	Towers	Accel.	24.01	41.00	11.26	18.88
			Displ.	18.22	39.03	6.77	5.32
		Mid-span	Accel.	0.60	8.92	-5.79	35.75
			Displ.	2.82	14.28	-5.47	20.65
C-4b	Earthquake	Towers	Accel.	24.14	41.54	11.13	24.68
			Displ.	18.31	39.40	6.62	7.09
		Mid-span	Accel.	21.28	39.86	-4.08	1.98
			Displ.	15.52	23.50	6.66	-30.63
C-5	Wind and Earthquake	Mid-span	Accel.	18.80	41.12	-1.27	37.33
			Displ.	11.24	19.99	0.32	19.48

4. Conclusions

This study presents a case study of the Runyang Suspension Bridge (RSB) to illustrate multi-hazard and multi-performance considerations in vibration control of long-span bridges. Time history analysis of the bridge with and without tuned mass dampers (TMDs) are carried out, using a set of earthquake ground motions and spatially variable wind field simulated over a grid, to investigate the effects of wind and earthquake actions on the peak response of the deck and the towers of the bridge. The main conclusions that can be drawn from the results presented here are listed below.

a. Wind load primarily excites the fundamental mode of the bridge and is important for deck displacement.

b. Seismic load excites the first vibration mode of the tower and is important for its acceleration as well as displacement response. Since this mode is coupled with the transverse deflection of the deck, seismic load also induces significant acceleration of the deck.

c. Tuned Mass Dampers (TMDs) can be effective in reducing seismic- as well as wind-induced response of the RSB when designed and placed appropriately. It was found that a TMD designed to reduce wind-induced displacement of the deck results in undesirable effects on its seismic response. Similarly, TMDs designed to control seismic response of the deck was found to amplify its wind-induced displacement by about 30%. Importance of multi-hazard and multi-performance considerations in passive vibration control is therefore clearly demonstrated.

d. To control different response parameters such as displacement and acceleration of the different elements, i.e., the deck and the tower, multiple TMDs tuned in different ways are necessary. For example, of the solutions investigated here, a system with a TMD on each tower (tuned to the 5^{th} mode), and 3 TMDs on the deck tuned to the 5^{th} mode and one additional TMD on the deck mid-span tuned to the first mode provide the best solution in the multi-hazard and multi-performance consideration.

The results highlight the importance of multi-hazard considerations in vibration control of bridges, an area which has not been adequately addressed in the literature. This study does not provide a comprehensive framework for such work and has a limited scope. It does not formalize the uncertainties of the different loading types and their joint distributions, nor does it provide advanced optimization approaches. However, the study provides an illustration of some important issues in multi-hazard vibration control of structures affected by loads of different nature and frequency characteristics and is thus potentially a beginning of a more thorough and advanced future investigations in this area.

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