Lecture Notes in Civil Engineering

Waleed Abdullah ·
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Civil Structural Health Monitoring

Proceedings of CSHM-9 Workshop

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Preface

The 9th International Workshop of Civil Structural Health Monitoring, CSHM-9 2024, was planned to provide a unique educational and training experience for international and local scientists, engineers, researchers and entrepreneurs to explore and discuss recent advances and the state-of-the-art, state-of-the-practice and future trends in smart sensors, wireless sensor networks, signal acquisition and processing, real-time data transferring and management for intelligent infrastructural health monitoring and to seek out opportunities for international cooperation.

CSHM-9 was held in Kuwait between 12–14 February 2024, at the Sheikh Jaber Al-Ahmad Cultural Centre (JACC). It is the largest cultural center and opera house in the Middle East. CSHM-9 workshop was jointly organized by the Society for Civil Structural Health Monitoring (SCSHM, formally ISHMII) and the Kuwait Society of Engineers (KSE).

CSHM-9 received manuscripts from local as well as international authors. The scientific committee has critically reviewed and referred all submitted papers and accepted a total of 27 papers. The proceedings contain 18 papers by authors who registered for the workshop and presented their work in-person or online. The papers covered the main topic which included: New Seismic Design Code Applications, Historic Buildings Retrofitting Against Seismic, State of Seismic Damage Detection/Prediction Techniques, SHM in Earthquake Prone Regions and Case Studies Applications. The proceedings also include papers based on the keynote lectures delivered by international experts.

We would like to acknowledge and give special appreciation to members of the scientific committee for their valuable contribution and invited speakers and delegates for sharing their experiences. We would also like to extend our appreciation to the local organizing committee, the media and public relation committee for the devotion of their precious time, advice and hard work during the workshop.

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Contents

Monitoring and Structural Assessment of Instrumented Cable-Stayed Bridges Using Recorded Seismic Responses

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Abstract. This paper describes case studies and lessons learned on seismic monitoring and structural assessment of several cable-stayed bridges using seismic records from wired and wireless monitoring system. The subject bridges are Yokohama-Bay bridge, Tokachi bridge and Shin-Nakagawa bridge in Japan. At the Yokohama-Bay bridge (total length 860 m, main span 460 m), a comprehensive wired monitoring system has been established for more than 30 years. Records from various levels of earthquakes including the largest one in Tohoku 2011 were studied to understand characteristics of seismic responses and explain structural behaviors such as tower-girder sliding and pounding. At the Tokachi bridge (total length 501 m, main span 251), a single-plane cable-stayed bridge, behaviors of friction bearings were investigated using recorded seismic responses from wired monitoring system and system identification was implemented to explain bearing behaviors under various earthquake levels. At the Shin-Nakagawa bridge (total length 533 m, main span 283 m), the development of wireless sensor network (WSN) and the results of 45-month continuous seismic monitoring campaign are described. The WSN monitoring system successfully recorded high quality seismic responses from 63 seismic events which included near-field and far-field earthquakes. Important analyses on the seismic records and performance of LRB seismic isolation bearing, using the recorded seismic responses are presented.

Keywords: Seismic monitoring · Cable-stayed bridge · Seismic Response · Wireless Sensor Networks · Bearings

1 Introduction

Seismic monitoring of bridge structures has been implemented in many earthquake-prone countries to special bridges, long-span bridges or bridges with new technologies. Generally, the monitoring system consists of a series of vibration measurement sensors that record seismic responses of important bridge elements. The seismic monitoring systems have shown great potential for structural assessment after occurrence of earthquake. From structural point of view, seismic monitoring of bridges has several main objectives, namely, to quantify seismic load and its distribution on structure, evaluate seismic response, and evaluate local and global structural performance during large earthquake,

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detecting possible damages due to earthquake, and verify performance of structural retrofit system. Seismic monitoring is very important for large, important structures and structures with specific features and innovative technologies such as seismic isolation or response control systems [1].

From the research and development point of view, there are still numerous challenges in the seismic monitoring of bridge structures. Two of the most important challenges are the deployment strategies and interpretation of recorded seismic data. Deployment strategies involve development or selection of reliable sensors and architecture of monitoring system. This includes selection of sensor type, determination of number and location of sensors, determination of level of accuracy, durability of sensors and cost performance analysis of monitoring system. The deployment strategy needs to consider the objective of seismic monitoring mentioned previously and how the specific objective of monitoring on the bridge can be achieved by the selected sensor and deployment strategy. In the case of seismic monitoring, it should be remembered that essentially earthquakes do not occur very frequently and at unpredictable times, but the monitoring system must always be ready and reliable. Data processing and interpretation of recorded seismic responses are essential to provide important information for structural assessments which are needed by bridge owners or operators. For this purpose, effective and reliable structural identification methodologies are important for rational and systematic means of data interpretation.

In this paper we describe case studies and lessons learned from implementations of seismic monitoring on three cable-stayed bridges. The subject bridges are Yokohama-Bay bridge, Tokachi bridge and Shin-Nakagawa bridge in Japan. In the first two bridges, conventional permanent wired sensing system was implemented and the results of data analysis from numerous earthquakes over years of monitoring experience is described. In the third bridge, experience on deployment of a semipermanent wireless monitoring system is presented. The paper describes typical analysis using system identification and data interpretation that can be used for structural assessment. Realizing the importance of seismic performance of bridge supports or bearings, a special attention is given to evaluate performance of bearings or connections between tower and girder during an earthquake.

2 Seismic Monitoring of Yokohama Bay Bridge

The first case study is the Yokohama-Bay Bridge. The bridge is located at the entrance of Yokohama harbor and is a part of the Yokohama-Tokyo bay-shore expressway. It is a continuous three-span cable-stayed bridge with the main girder consisting of a double-deck steel truss-box. The central span is 460 m with side spans of 200 m each. The upper and lower deck have 6 and 2 lanes, respectively, with the upper deck being part of the Yokohama Expressway Bay shore route and the lower deck a part of the national route. The lower deck was added later and completed before the 2004 Chuetsu-Niigata earthquake. The bridge has two H-shaped towers of 172 m height and 29.25 m width with a welded monolithic section. Construction was completed in 1988 and the bridge was opened in September 1989. The bridge was built in a seismic area, with the location near the site of 1923 Great Kanto Earthquake. Considering the weak ground condition, and high center of gravity of the bridge; a special seismic isolation design system was adopted to connect bridge girder with towers and piers in a form of Link-Bearing Connections (LBC). The LBC system consists of 10-m-long end-links links at the end-piers (P1 and P4) and 2-m-long tower-links at the towers (P2 and P3) and accommodates girder longitudinal motion (Fig. 1). The system is designed to maintain a long longitudinal fundamental period and allows the girder to be suspended from towers and piers. By lengthening the natural period, the structure experiences smaller acceleration thus the effect of inertia force of superstructure on substructure during an earthquake.

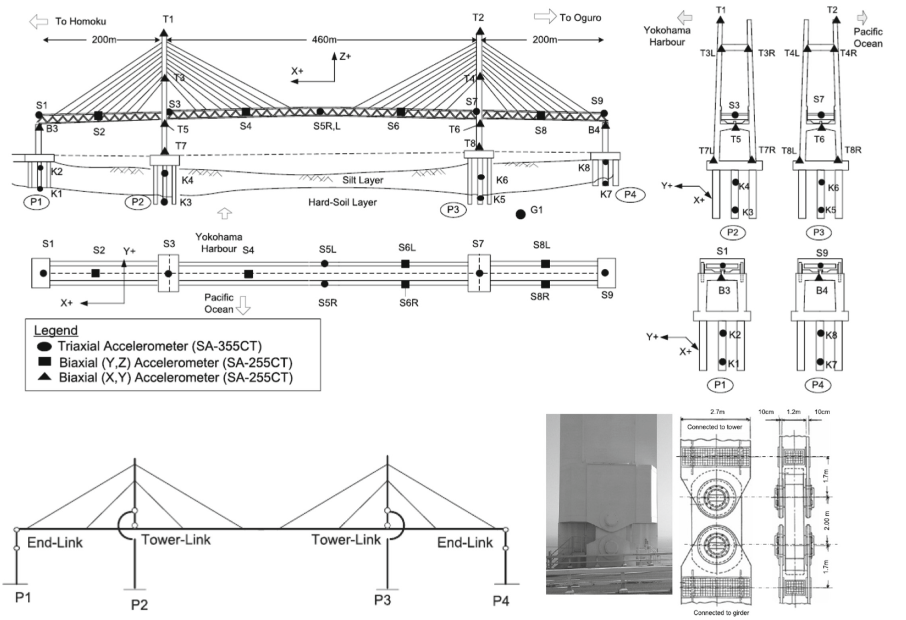

Fig. 1. Yokohama Bay Bridge sensor codes and locations, characteristics of pier–girder and tower-girder connections and detail figure of tower-link (link bearing connection - LBC) [2]

The bridge has a permanent structural seismic monitoring system consisting of conventional wired accelerometers. Two types of accelerometers are used, namely, servo type accelerometers SA-355CT (triaxial) and SA-255CT327 (biaxial) produced by manufacturer Tokyo Sokushin. These accelerometers have the maximum amplitude capability 20 m/s^2, sensitivity 2m/s^2/V and linearity 0.03% of the full-scale. All sensors have frequency bandwidth of 0.05–35 Hz and operate at sampling frequency 100 Hz. The monitoring system includes 85 channels at 36 locations including piers, towers, girder and foundation that measure accelerations in three directions (Fig. 1).

The monitoring system was deployed after the bridge was opened and more than one hundreds earthquakes have been recorded by the monitoring system from January 1990 until now. The system is operated and maintained by Metropolitant Expressway Public Co. Less than half of the earthquakes are of significant level (JMA intensity > 3) or Peak Ground Acceleration (PGA) equal or more than equivalent to 0.08 m/s^2.

The largest ground motion ever recorded at the bridge is due to 2011 Great East Japan earthquake. Figure 2 shows the main shock accelerations recorded on three levels of tower P3 foundation: hard-soil layer (246:42 m), mid foundation level (218:05 m), and base of the tower.

Seismic records of the bridge have been studied extensively and the study of seismic records from long-term monitoring system was divided into three categories: (1) earthquakes before 2004, which is before addition of new traffic lane on the lower deck. (2) earthquakes due to 2004 Chuetsu-Niigata and after, the 2004 Chuetsu-Niigata earthquake generated main-shock and series of aftershocks before seismic retrofit [2], and (3) earthquakes due to 2011 Great East-Japan earthquake and aftershocks [3]. The objective of seismic monitoring were to confirm the seismic loads by comparing the ground motion with retrofit ground motion, assess overall structural seismic performance and detect possible unwanted structural behaviour after strong earthquake. For assessment of global structural performance, data analysis by system identification was normally conducted. Considering the nature of long-span bridge and dense instrumentation, multi-input multi-output (MIMO) system identification that utilizes correlations between input-output data was normally implemented to estimate modal parameters of dynamical system through a realization process. To implement the system identification responses from triaxial accelerometers located at the bottom of the end-pier and towers were selected as inputs, while the responses from bridge decks, piers and towers were used as outputs.The system identification and spectra analysis most of time were adequate to provide quick observation of bridge response for small and moderate earthquakes where assumptions of linear or weakly non-linear structural responses are still reasonable. However, for larger earthquake like 2011 Tohoku earthquake, time-frequency analysis by wavelet transform or peicewise nonlinear system identification were conducted for larger earthquake considering possible nonlinearity. With dense sensor configuration, identifications of significant number of modes typically 14 modes between 0.1 and 2.5 Hz is possible. In addition, dense sensor configuration also allows for modal identification with higher spatial mode-shapes, which is important for structural assessment using performance indicator associated with mode-shapes.

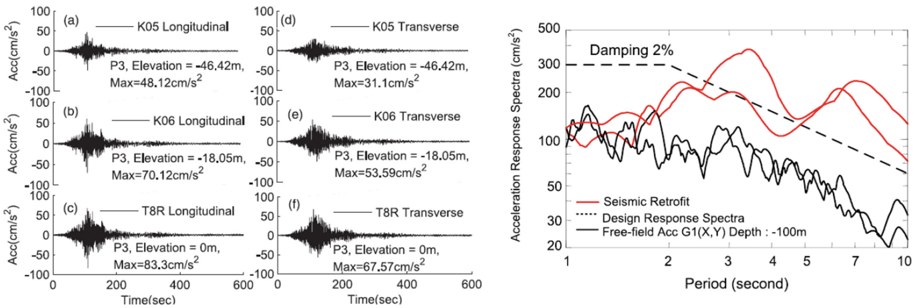

Fig. 2. Ground-motion recorded at Yokohama Bay Bridge in 2011 Tohoku earthquake [3]

From the analyses of seismic responses related to the 2011 Great East-Japan, it was observed that natural frequencies generally decrease with the increase of excitation amplitude. Damping ratio estimates show large variation within 0.5–6% [3]. In some cases, such as earthquakes before 2004, the results indicate that damping ratios of lower modes increase with the increase of earthquake amplitude, which might be due to the results of greater energy dissipation caused by friction in bearings that occurs during large earthquake. Based on the results of seismic monitoring between 1990 and 2005, the possibility that LBC may not function properly during a large earthquake was recognized [2]. In such a case, excessive moment at the bottom of end-pier may be resulted and the LBCs could fail causing uplift deformation at the girder.As a feedback of the monitoring, the seismic retrofit of the bridge conducted in 2005 included a fail-safe design by connecting the girder to footing using prestressed cables to prevent uplift of the girder-end in case of LBC failure [1] (Fig. 3).

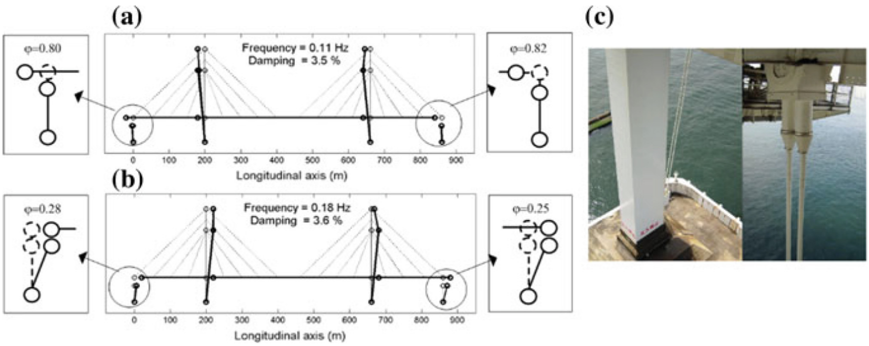

Fig. 3. Example of the first longitudinal mode with (a) slip-slip condition, and (b) stick-stick condition. (c) Photos of a fail-safe design system using pre-stressed cable [1, 2].

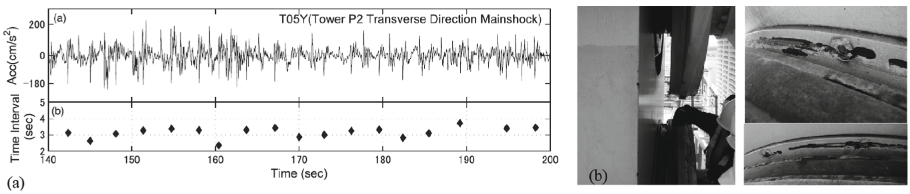

Fig. 4. (a) Close-up look at the tower transverse (in-plane) acceleration at the tower–girder connections shows periodic impulse response for main shock, and (b) time interval between consecutive impulses (c) Photos of scratch marks on the surface of the bottom link head of LBC caused by combination of vertical and transverse movement of the girder after 2011 Tohoku earthquake [3].

In the 2011 Tohoku earthquake, The bridge responses during the main shock were dominated by transverse movement of girder and tower, with the maximum transverse displacements on the top of the tower and in middle of the girder 55 cm and 62 cm, respectively. The earthquake did not cause girder unseating, since the maximum longitudinal displacement was only about 15 cm; that is, less than 1.5 m maximum allowable longitudinal displacement. While the LBCs were found to have functioned properly in longitudinal direction, the tower transverse accelerations were characterized by many periodic spikes resembling impulses, especially during the largest excitation of the main shock. Periodic impulses indicate occurrence of transverse pounding between tower and girder, and the impulses also appear on the girder vertical accelerations. By observing the time interval between two successive impulses, the structural mode associated with the impulse was investigated and it was found that the girder first transverse mode triggered the pounding between tower and girder at about 3.2 s (0.31–0.32 Hz) [3].

Despite its occurrence, transverse pounding did not cause structural damage at the present earthquake. Post-earthquake visual inspection confirmed the occurrence of transverse pounding in the pier–girder and tower–girder connections (Fig. 4). Marks created by girder on the surface of wind shoes was found as the result of collision between wind shoes and girder; which confirms the presence of periodic impulses on acceleration records. The pounding process is studied using simplified model of two-side contact problem and complete finite element analysis [4]. In the model, the values of spring constant that represent the contact stiffness between pier or tower and the girder are determined by adjusting the value of modal parameters identified from seismic response with that of bridge model.

The above explanation demonstrates that recorded accelerations from permanent seismic monitoring system and data interpretation using system identification are instrumental in explaining the structural behavior during the earthquake and help us in understanding the results of post-earthquake visual inspection.

3 Seismic Monitoring of Tokachi Cable-Stayed Bridge

The second case study is the lesson learned from monitoring of Tokachi cable-stayed bridge in Obihiro, Hokkaido, northern Japan. The bridge spans over the Tokachi river, connecting Obihiro city and Otofuke. It is a three-span continuous girder cable-stayed bridge with a total length of 501 m with 251 m center span, as shown in Fig. 5. The girder consists of four-chamber box prestressed concrete structure with a width of 32.8 m. At the time of its completion, the bridge was the widest prestressed concrete cable-stayed bridge in Japan. The towers (P1 and P2) are made of 68 m tall single-column reinforced concrete. Below the girder, they are rigidly connected to an oval wall-type reinforced concrete pier with a large cross section and high rigidity. The main girder is supported by a single cable plane. Bridge is that it is fully restrained in the direction perpendicular to the bridge axis. Meanwhile the bridge girder is movable in the longitudinal direction (bridge axis). Both ends of the girder are supported by spherical frictional plate (SFP) bearings (see the inset photo in Fig. 5) that are movable in the longitudinal direction (bridge axis) but fixed in the direction perpendicular to the bridge axis by an inverted T-shaped abutment. It should be noted that the bridge is located in a cold area with long winter that mostly covered with snow. In such harsh environmental condition monitoring the performance of bridge's sliding bearing is importance to ensure its functionality during an earthquake.

The bridge is instrumented with a permanent seismic monitoring system of wired sensors at 15 locations measuring accelerations in three principal directions (i.e., longitudinal, lateral, and vertical). All accelerometers are of servo type with 0.01 cm/s^2 of

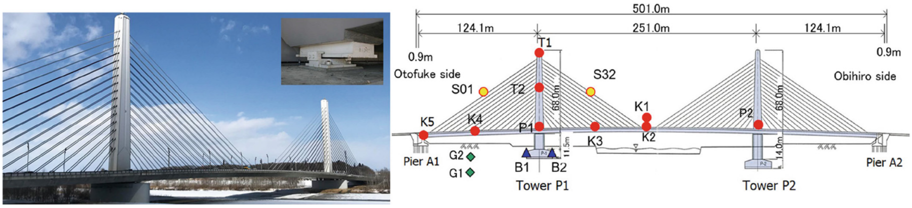

Fig. 5. Tokachi cable-stayed bridge (42.93 N, 143.203E), dimension, and sensors layout. (Inset: moveable spherical frictional plate (SFP) bearings used at both ends of the girder) [5]

measurement resolution, ± 2000 cm/s^2 of measurement range and 100 Hz of sampling frequency. The monitoring system was deployed after the bridge was opened in 1996 and numerous earthquakes have been recorded with the 2003 Tokachi earthquake as the largest one. Unfortunately, the monitoring system was not in service to record the 2011 Tohoku earthquake (Fig. 6).

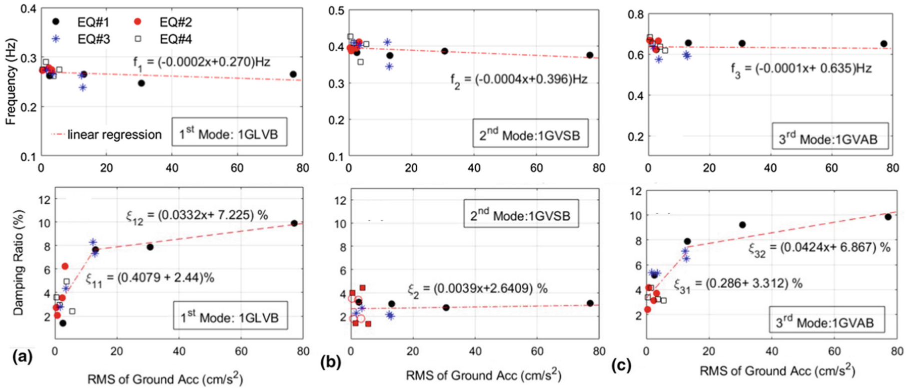

Fig. 6. Variation of identified natural frequencies and damping ratios with respect to the rms of ground motion for all earthquake frames and the linear trendlines of their relationships. For modes: (a) 1st Girder Long. with vert. bending (1GLVB), (b) 1st Girder Vert. Sym. Bending (1GVSB), **c** 1st Girder Vert. Asym. Bending (1GVAB) [5]

In the study by the authors [5] extensive investigation on the records from the near-field 2003 Tokachi earthquake, which is the largest earthquake ever recorded by the monitoring system, is presented. The bridge dynamic characteristics including frequency and damping were estimated by time-invariant and time-variant recursive subspace system identification methods. Modal parameters obtained from system identification using seismic records vary with respect to the excitation levels. There are two interesting findings from the system identifications; one is the relatively low damping of the tower dominant modes, and the other is the significantly high damping of the two girder's longitudinal modes. The identification results reveal that damping ratios of the low order modes estimated during the largest earthquake reached more than 10% of the critical damping, which is significantly higher than what usually assumed in the seismic design code. The two modes have significant longitudinal modal displacement at the girder ends where the moveable SFP bearings are located. Significant relative longitudinal displacement between girder and tower was also observed from the records suggesting the longitudinal movement of the girder. The unique characteristics of damping behavior of the first and third mode is thought to be caused by the friction force at the SFP bearings. An observation of the mode-shapes generated by finite element models and identified from seismic records show that the two modes have significant longitudinal modal displacement (δ) at both ends of the girder compared to the other modes as shown in Fig. 7. The friction force was resulted by the movement of the SFP bearings, and since this movement was mainly contributed by modal displacement of the first and third mode, it significantly increases modal damping of the modes. Movement of the girder was confirmed by relative longitudinal displacement between girder and tower due to movable SFP bearing shown in Fig. 7(b). The relative displacement up to 15 cm was observed at t = 80 s in the 2003 Tokachi earthquake which is the time of the largest shaking.

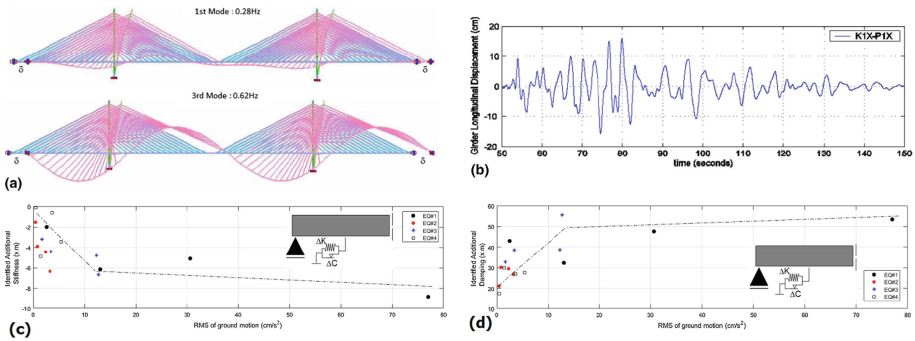

Fig. 7. **(a)** FE generated mode-shapes of the first and third mode with significant longitudinal modal displacement (δ) at both girder ends that are supported by spherical frictional plate bearings. **(b)** Relative girder-pier longitudinal displacement during (c) identified supplementary stiffness and **(d)** identified supplementary damping caused by friction forces of the movable SFP bearings at the girder ends by inverse analysis [5].

The dependencies of natural frequency and damping ratio on the seismic excitation levels are related to the change in the structural dynamic parameters, namely, the stiffness and damping. Since the structure conditions are practically the same during the entire seismic excitation, the changes in stiffness and damping can be related to the variation in the moveable SPF bearing conditions at different excitation levels. These changes or variations are represented as a supplementary stiffness and a supplementary damping at both girder ends whose quantities vary following the movable bearing condition at different excitation levels. In order to quantify the supplementary stiffness and supplementary damping, an inverse analysis is conducted using modal information obtained from system identification [5]. The results of inverse analyses indicate that the increase of supplementary damping becomes more significant as the excitation intensifies as shown in Fig. 7. The figures show that gradients of regression lines are higher at the beginning that is when the rms acceleration is below 15 cm/s^2. This indicates that the rate of changes of both supplementary stiffness and supplementary damping values are greater at the low level of excitation. At higher excitation levels, however, the gradients gradually decrease suggesting less variations of supplementary stiffness and damping.

Results of the analyses confirm the influence of moveable SFP bearings performance with its Coulomb-friction type stick–slip behavior on the dynamic characteristics of the bridge. The bearings remain stuck at smaller excitation causing higher stiffness of the structure and slip during a larger excitation resulting in stiffness reduction. The sliding process induces friction force between contacting surfaces of the SFP bearings which results in the increase of damping as the excitation level increases.

The findings on damping and its variation are important from structural engineering viewpoint. Damping ratios of cable stayed bridges are commonly obtained from forced vibration tests and their values are generally small in the range of 2–5%; much smaller than the values observed in this study. This range of damping ratio usually recommended in the design code. However, it should be realized that the small damping ratios determined from such tests are attributed to the small vibration amplitudes. This study demonstrates that for a cable-stayed bridge with movable bearings, significantly larger damping is observed when the bridge is under large earthquake excitations. In such condition, the large damping could contribute significantly to the reduction of bridge seismic responses.

4 Seismic Monitoring of Shin-Nakagawa Cable-Stayed Bridge by Wireless Sensors Network (WSN)

The third case study is the monitoring of Shin-Nakagawa cable-stayed bridge using long-term wireless sensor networks. The bridge is a single-plane single-pylon cable-stayed located in the suburb of Mito City, Ibaraki Prefecture, east Japan (Fig. 8). Total span length of the bridge is 533 m and width 22 m consisting of a continuous steel box girder. The longest span is 283.9 m crossing over the Nakagawa River, and the three-continuous side spans of 113 m, 75 m, and 58.9 m, respectively, are supported by three piers. The stay cables system comprises fifteen cables arranged in a harp configuration radiating from the main pylon 50.4 m above the deck. The substructure consists of four reinforced concrete piers named P31, P32, P33, P34 and P35. Only P35 is located on the left bank of the Nakagawa River, while all the other piers are located on the right bank of the Nakagawa River. The main pylon (P34) is in the middle of the bridge with the total height of 127.5 m. All piers are supported by pile foundation while the main pylon is supported by caisson foundation.

The bridge girder is supported by several types of bearings and connections above the piers and at the main pylon: namely, laminated rubber bearings (LRB), wind shoes (WS), pendel bearing (PB), viscous damper (VD), and uplift prevention bearing (UP) as illustrated in Fig. 8. The support bearings accommodate free longitudinal movement of the girder. To absorb excessive girder longitudinal vibration, viscous dampers were placed between girder and the main pylon. Possible uplift vertical motion at the end girder is prevented by the uplift prevention bearings at the pier P35. More details on the bridge structural configuration can be found in [6].

Fig. 8. Shin-Nakagawa cable-stayed bridge (36°21′25.81″N, 140°33′28.04″E) and layout of wireless sensor network [6].

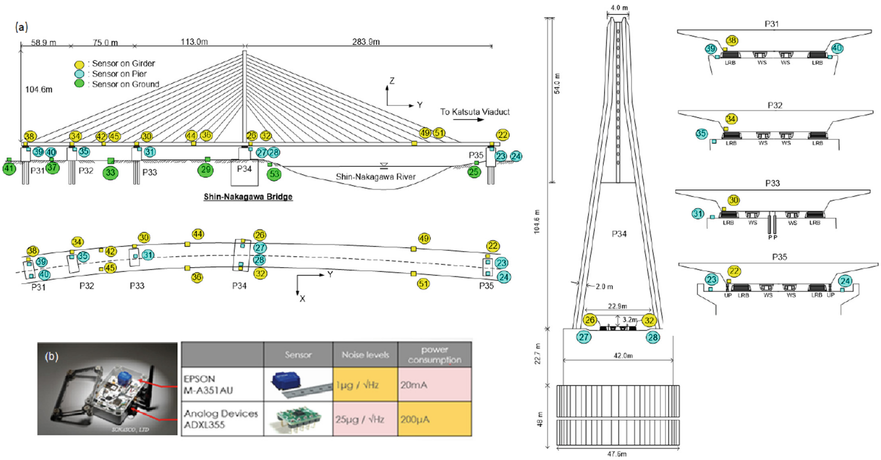

Fig. 9. Shin-Nakagawa cable-stayed bridge (36°21′25.81″N, 140°33′28.04″E) and layout of wireless sensor network [6].

The bridge was opened in 1997 and since then has become a part of the East Mito route highway. In 2011 Tohoku earthquake, several isolation bearings on the pylon (P34) and piers 31 were ruptured. The damaged isolators were replaced after the earthquake. The VDs and UPBs were installed after the 2011 Great East Japan (Tohoku) earthquake as part of a seismic retrofit system. Since the incident, performance of isolation bearings have been a concern and a long-term seismic monitoring was implemented by wireless sensors network. A newly developed WSN system is implemented. The system used a protocol Choco which was developed using Concurrent Transmission Flooding (CTF) principle, with five main features and strategies for effective structural monitoring system [7]. The WSN utilizes a simultaneous high-speed flooding technique that was found efficient in transmitting the packets at the same time without causing synchronization problem to the carrier's frequency or phase [6]. In this system, a master node system, called the sink node, is assigned to coordinate communications of all the other nodes. The sink node periodically transmits time-synchronization packets to synchronize signal with the other nodes. This technique enables creation of a stable and highly efficient sensor network.Ttwo types of sensors namely the analogue devices ADXL362, and

EPSON M-A351AU were selected as a low-power sensor, and a high-precision sensor, respectively.

A total of 26 sensor nodes consisting of triaxial wireless accelerometers were deployed on the bridge (Fig. 9). The girder sensors were placed on the side of the girder right above the isolator bearings. For the girder nodes, the sensor boxes were fixed to the lower flange of the girder box with steel clamps to prevent any relative movements. The arrangement was intended, among others, to evaluate the performance of isolators during an earthquake by comparing the response characteristics of the piers and girders.

The monitoring system was installed in summer 2017 and has continuously recording seismic responses for 45 months until December 2020. Within this period, the system has successfully recorded structural responses from 63 seismic events consisting of small-to-moderate earthquakes including the far-field and near-field earthquakes. In the study by authors [6, 8] comparisons of the ground motions recorded by WSN and existing nearby strong motions seismographs were presented in details to demonstrate capability of the WSN system sensing and recording the ground motion and seismic responses of the bridge. Figure 10 shows the examples recorded ground motions and acceleration response spectra for the largest near-field earthquake on July 17th, 2018. The figure also compares the records from WSN and the nearby KNET seismograph records (i.e. KNET-IBR007) which belongs to NIED (Japan national seismograph networks).

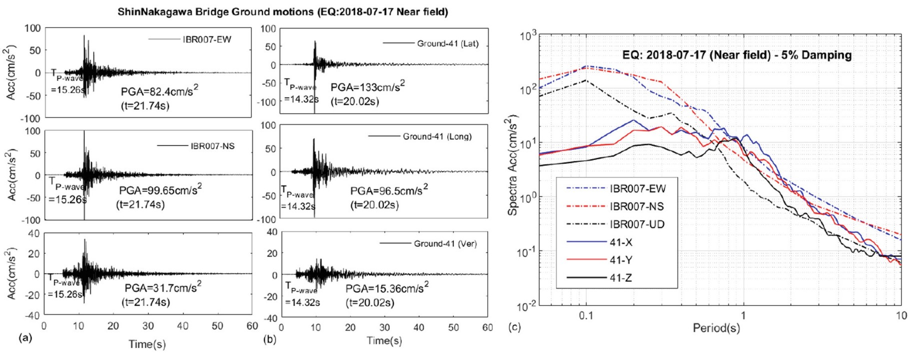

Fig. 10. Ground accelerations recorded on the free-field during the largest near-field earthquake (July 17th, 2018) by: (a) KNET sensors, (b) Wireless Sensor Networks, (c) Acceleration response spectra [6].

Dynamic characteristics of the bridge under various levels of seismic excitations were evaluated using time-domain system identifications. Similar to the previous cases, the state-of-the-art, multi-input, multi-output system identifications were utilized using input-output correlation between responses on ground and structural responses on the pier, tower and girder. In the previous study by authors [6], effect of ground motion levels on the dynamic characteristics namely natural frequencies and damping ratios were investigated. It should be mentioned, however, that so far the recorded earthquakes are limited to moderate level of earthquake with largest recorded peak ground acceleration is 133 cm/s^2, which is considered quite moderate to examine overall structural behaviour during earthquake especially related to nonlinearity caused by large earthquake.

As a reference for further investigation and to help understanding the results of recorded seismic responses analysis a three-dimensional finite element model of the bridge was developed using structural analysis software ABAQUS (Fig. 11). The simulated seismic responses were compared with the recorded ones at the sensor nodes positions. Figure 11(c) exhibits examples of comparisons of pier cap accelerations in longitudinal (along bridge axis) direction obtained from finite element model and the recorded accelerations during the largest near-field earthquake (July 17th, 2018). In general, reasonable response comparisons were obtained. In general the finite element model was able to represent the overall dynamic characteristics of the bridge as described in the modal frequencies and the main characteristics of seismic responses.

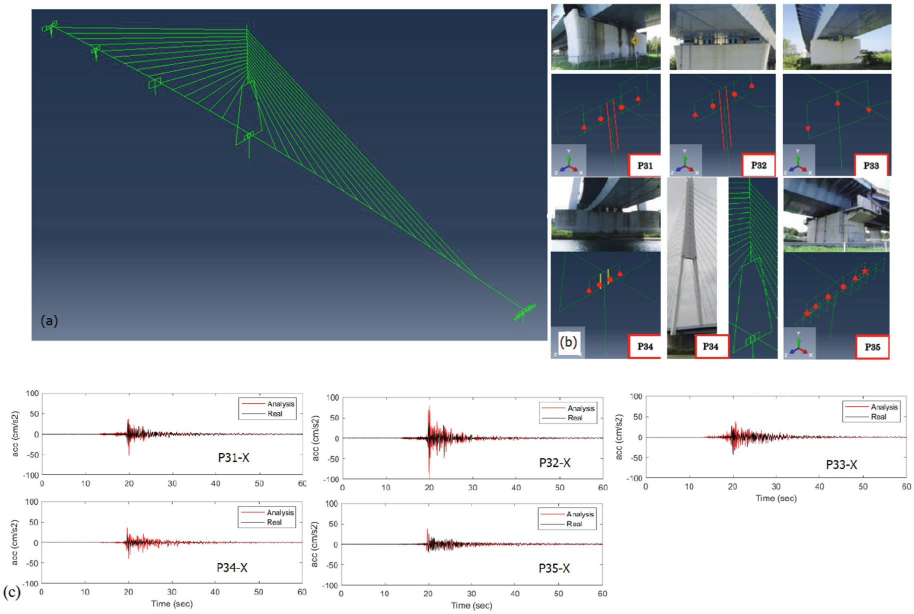

Fig. 11. Finite element model of the bridge (a) general model of the structure (b) modelling of pier/tower girder connections. (c) Comparison of pier accelerations between finite element model and the recorded accelerations at the location of pier-girder connections during the largest near-field earthquake (July 17th, 2018).

As mentioned previously, one of the objectives of monitoring in this bridge is to evaluate performance of seismic isolation bearings from recorded seismic responses. While modal parameters obtained from system identification are normally used to evaluate global structural condition, it is difficult to understand the local condition of isolation bearing directly from the changes in modal parameters because the changes are also influenced by other factors. Therefore, it is more desirable to assess isolation bearing performance directly from a localized analysis of the vibration signal near the bearings. For this purpose, time-frequency analysis of the girder and pier seismic response is conducted using wavelet transform. The wavelets analysis is employed to investigate the high-frequency filtering effect as evidence of functioning isolation system [9]. Seismic responses of the piers or substructure elements normally contain high-frequency components from ground motions in addition to the structural frequency components. In a conventional non-isolated bridge, the high-frequency component is transferred to the girder because pier and girder are rigidly connected. In a functioning seismically isolated bridge, however, the high-frequency component will be filtered out from the girder acceleration by the isolation system connecting the piers and girder. Previous study and simulation have shown that the changes in wavelet transform map can be associated with structural changes related to the filtering effect in a seismically isolated bridge [9]. The ridge of wavelet transform can be interpreted as the location where the major energy of vibration response is localized among the distributed energy density over the time-frequency plane. Thus, it is considered as the governing or dominating vibration characteristics at a specific time instant.

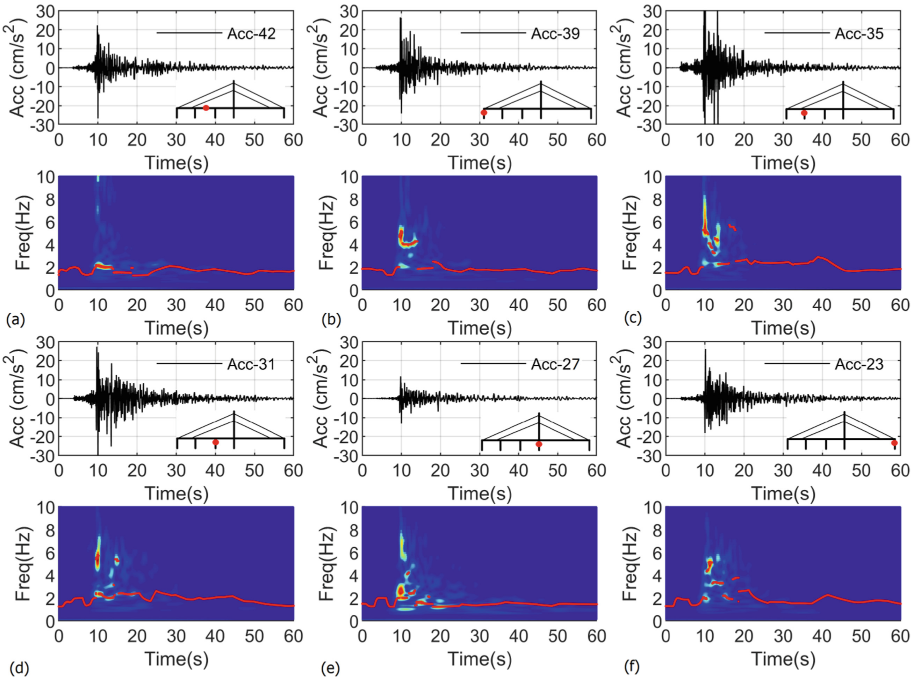

Fig. 12. Recorded longitudinal seismic responses of Shin-Nakagawa bridge due to the largest near-field earthquake (July 17, 2018): accelerations, time-frequency map of acceleration and instantaneous frequency (noted by the lines) obtained from the ridges of time-frequency map of accelerations: (a) Ch-42 (girder); (b) Ch-39 (P31); (c) Ch-35 (P32); (d) Ch-31 (P33); (e) Ch-27 pylon (P34); and (f) Ch-23 (P35).

Figure 12 displays examples of recorded accelerations, the time-frequency map, and the corresponding instantaneous frequency due to the largest near-field earthquake (July 17th, 2018). The largest peak in the time-frequency map occurs during the peak excitation period t = 10–20 s. At this time, the girder response is dominated by a single low frequency response at 2.1 Hz, which corresponds to the girder's second longitudinal mode. Based on the instantaneous frequency, it was confirmed that the single frequency of girder longitudinal mode remains the dominating frequency until the end of excitation (Fig. 12.a). Meanwhile, all piers and pylon out-of-plane accelerations have the same characteristics (Fig. 12.b–f). Nonstationary characteristics of the acceleration responses are evident from the time-frequency map and instantaneous frequency. In the beginning of the response ($t < 10$ s), before the arrival of the first wave, the piers and pylon characteristics of instantaneous frequency are very similar to that of the girder. After the arrival of the secondary wave, however, the responses were characterized by higher instantaneous frequencies, between 4.76 Hz and 5.3 Hz. The frequency range of 4.76–5.3 Hz corresponds to the pier's out-of-plane frequency at 5.25 Hz. After the period of peak excitation ($t > 20$ s), the instantaneous frequencies of the piers and pylon accelerations drop to 2.1 Hz and remain so until the end of excitation. The difference in the dominating frequency of the piers and girder accelerations during the peak excitation ($t = 10$–20 s) suggests that the girder and piers become uncoupled. In this condition, the girder is isolated from the piers and the pylon thus the high-frequency contents of the piers' vibration are not transferred to the girder. This is known as the high-frequency filtering effect. The results are expected from a functioning isolation bearing and similar response characteristics have been observed in the previous studies of movable bearing and functioning link bearing of the seismically isolated bridges [1]. Utilizing the results of time-frequency analysis, a systematic classification by clustering algorithm can be performed to determine whether the bearings have functioned normally. The analyses can reveal possibilities of bearing malfunction or locked bearings based on the observed seismic responses [9].

5 Comparative Cost of Monitoring System

The above case studies have demonstrated the benefit of permanent seismic monitoring system and how seismic monitoring with systematic data interpretation using system identification can be used to explain structural behaviour including global and local components during the earthquake. This information help us to assess structural condition and assist the post-earthquake visual inspection normally conducted after an earthquake. As mentioned previously, in addition to data interpretation, it is also important to critically evaluate the expected performance and cost when designing a monitoring system. In general, the total cost of monitoring depends on the strategy, duration, and purpose of monitoring. For a long-term permanent seismic monitoring of a long-span bridge with a duration of 20–30 years, high accuracy servo type wired accelerometer is commonly used in Japan [10]. This type of monitoring generally costs about JYP 3–4 million (USD 27,000–36,000) per sensor node including installation cost. There will be additional cost of cabling depending on the bridge length and maintenance of about JYP 0.2 million (USD 1,800) per sensor node. The same system or a system with lower sensor specification can be used in a semipermanent monitoring of shorter period (3–5 years). WSN is a good alternative for a continuous semipermanent monitoring of the shorter period 3–5 years. The initial cost of WSN is about JYP 0.25 million (USD 2,300) per sensor nodes and JYP 0.5 million (USD 4,600) for sink node without the need of cost for cabling. The additional cost will be for batteries that need to be replaced every year. Therefore, the use of WSN of the current type in average only requires about 20% of initial cost normally spent for monitoring system using servo-type wired sensors. This substantial cost reduction with relatively similar reliability of monitoring system offers a promising alternative of semipermanent monitoring system for large bridges.

6 Conclusions

This paper describes case studies of seismic monitoring of long-span bridges including the experience of typical data analysis and interpretation for structural assessment especially related to bearings connecting towers or piers and girder. The case studies are Yokohama-bay bridge, Tokachi bridge and Shin-Nakagawa bridge. In the first two bridges the conventional monitoring system by wired sensor networks are used, while in the Shin-Nakagawa bridge a newly developed wireless sensor system were deployed for continuous long-term monitoring. The bridges have different bearing systems, whose performance are important in distributing and controlling seismic forces during an earthquake. Using various system identification methods (i.e. linear, nonlinear or time-frequency), dynamic characteristics of the bridges were identified using recorded seismic responses and their characteristics with respect to earthquake levels and conditions were investigated. Results of system identification were used as reference for structural assessment. Finite element model of the bridges were developed to help understanding the structural responses and interpretation of seismic behaviour such as behaviour of link-bearing connection, bearing friction and pounding. The capability of WSN for a longterm seismic monitoring of long-span bridges has been demonstrated in the Shin-Nakagawa bridge. With efficient sensing, recording, deployment, and data transmission system, the WSN provides a promising alternative of an efficient and economical monitoring system compared with conventional permanent wired system.

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