A SHM-ORIENTED TEST-BED FOR LONG-SPAN SUSPENSION BRIDGES: SETTLEMENT STUDY

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The settlement problem of bridges, buildings, dams and other civil structures has been reported from time to time over the world. However, structural damages resulting from structural settlement are a perennial problem over the long service life of civil structures. Little research work has been conducted on the settlement problem of long-span suspension bridges. This paper presents both experimental and numerical studies to examine the effects of support settlement on structural response of a long-span suspension bridge and explore the possibility of using measured responses to detect the support settlement of the bridge. A laboratory-based test-bed, which comprises mainly a delicate physical model and a corresponding updated finite element model (FEM) of a long-span suspension bridge with various settlement scenarios implemented, is first introduced. The experimental arrangement, including instrumentation and experimental procedure, is then described. Four settlement cases were experimentally studied, which include two cases of anchorage settlement and two cases of tower settlement, to find their effects on bridge responses. The experimental results are compared with those computed from the updated FEM. The agreement of the results demonstrates that the FEM is a good representation of the physical model and can serve as a structural health monitoring (SHM) benchmark of long-span suspension bridges. Based on the FEM, the relationship between the structural response and the support settlement is finally utilized to develop a method for detecting the support settlement of the bridge.

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ABSTRACT: This paper presents both experimental and numerical studies to examine the effects of support settlement on structural response of a long-span suspension bridge and explore the possibility of using measured responses to detect the support settlement of the bridge. A laboratory-based test-bed, including a delicate physical model and a corresponding updated finite element model (FEM) of a long-span suspension bridge with various settlement scenarios implemented is first introduced. The instrumentation and experimental procedure is then described. Two cases of anchorage settlement and two cases of tower settlement were experimentally studied to find their effects on bridge responses. The experimental results are compared with those computed from the updated FEM. The agreement of the results demonstrates that the FEM is a good representation of the physical model and can serve as a structural health monitoring (SHM) benchmark of long-span suspension bridges. Based on the FEM, the relationship between the structural response and the support settlement is finally utilized to develop a method for detecting the support settlement of the bridge.

1 INTRODUCTION

The installation of comprehensive structural health monitoring (SHM) systems to important civil structures has become a trend to ensure the functionality and safety of these structures during their long service life and to prevent them from sudden failure and fatal disaster under extreme events (Mufti 2001, Xu et al. 2009). One main objective of the SHM is to detect structural damage due to various external or internal actions, including support settlement. The support settlement is caused mainly by soil deformation due to the action of static and dynamic loads of foundations as well as the change of water content and temperature. Depending on the magnitude of the settlement, structures could become seriously overstressed, cracked, and acclivitous, sustaining serious structural damage.

Settlement studies of bridges, buildings, dams, and railways have been reported by many researchers (Eden and Poorooshasb, 1968; David, 2003; Gikas and Sakellariou 2008; Rhyner et al. 2009). For example, settlement observations at the Kars Bridge in Ontario had been conducted since the start of construction (Eden and Poorooshasb 1968). Through the continuous field observations, they gave some suggestions for the

assessment of the bridge. Aggour and Aggour (1986) studied the settlement of simply supported bridges by treating the bridge as a space structure and concluded that the stresses introduced due to differential settlements of bridge supports must be considered in the design. Structural damages of the highway bridge Z24 caused by structural settlement were identified by the FEM updating method (Teughels and De Roeck, 2004).

Nevertheless, little reference has been found on how support settlements affect structural responses of long-span suspension bridges and how bridge structural responses can be used to detect support settlements. Since the support settlements always last a long period, it is difficult and time-consuming to observe structural response due to support settlement. A laboratory-based test-bed recently established by The Hong Kong Polytechnic University (Xu et al., 2011) provides a conceivable way to realize this settlement study. This test-bed was designed with reference to the Tsing Ma suspension bridge in Hong Kong with the geometric scale of 1:150. Meanwhile, the updated finite element model (FEM) of the physical bridge model in the test-bed was also built to compliment the physical model. Both of the physical bridge model and FEM can serve as a benchmark for the SHM of long-span suspension bridges. This paper focuses on the settlement study based on the SHM-oriented test-bed through a series of physical tests and finite element analyses, and a method is proposed for the detection of the structural settlement using some measured bridge responses.

2 DESCRIPTION OF THE SHM-ORIENTED TEST-BED

The bridge model stands on a steel beam, which is used as a shaking table, as displayed in Figure 1. The bases of the towers were welded on a steel plate which was then fastened to the shake table using four bolts. Slices of steel could be placed between the bottom steel plate of the tower and the shaking table at its first place, and then the removal of steel slice could simulate the settlement of the bridge tower. The two main cables are accommodated by the saddles sitting on the top of the two towers and extended to the Ma Wan (MW) and Tsing Yi (TY) anchorages. The anchorage at each side of the bridge was equipped with two specially-designed clamps to secure the main cables and two load cells to measure the cable tension forces. Both the anchorages are capable of being adjusted upward, downward, forward and backward for the simulation of the settlements or movements of the anchorages. Meanwhile, the FEM of the physical bridge model was established using the commercial software ANSYS. The FEM has been updated based on the dynamic properties from the model tests of the physical bridge model. More details about the bridge model can be found in Xu et al. (2011).

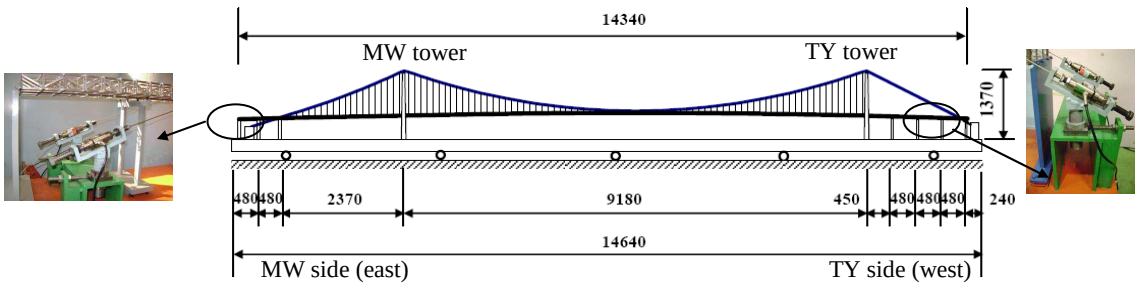


Figure 1. Configuration of bridge model and shaking table (unit:mm).

3 EXPERIMENTAL ARRANGEMENT AND PROCEDURE

3.1 Placement of sensors

Figure 2 shows the arrangement of sensors on the physical bridge model. The FBG sensor network was utilized to measure the deck strain and cable strain. The FBG sensors were mounted on the top surface of the outer upper longitudinal beam of the bridge deck and on the middle of cables between two suspenders, which are indicated as "FD" and "FC" respectively. The strain gauges were stuck on the bottom surfaces of the tower legs and "Ss" denotes the strain gauges stuck on the south side while "Sn" means that on the north side. The vertical displacements of bridge deck were measured by laser displacement transducers located on the foundation steel beam. Load cells were installed in the anchorages in both north and south sides to monitor the tensile forces of main cables.

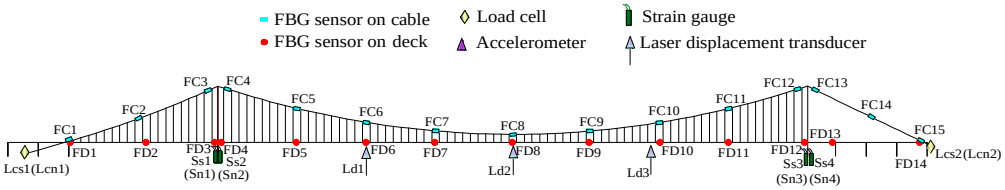


Figure 2. Types and locations of sensors.

3.2 Experiment scenarios

The initial configuration of the bridge model was reported by Xu et al. (2011), and it was taken as a reference for the tests as the first stage. Four settlement scenarios were then carried out, as listed in Table 1. The settlement tests of case 1 and case 2 were performed on the MW anchorage and TY anchorage, respectively, taken as the second stage. The displacements of anchorage settlement were generated by adjusting the top and bottom screw caps and monitored by dial gauges. In the third stage of the experiment, the settlement tests of the MW tower and TY tower were conducted. The settlement of the tower was realized by using the thinner cushion instead of the thicker cushion. It should be noted that the bridge was resumed to its initial configuration after the completion of each settlement test.

Table 1. Four settlement scenarios on test bed.

Case No.	Scenarios	Direction
1	Settlement of MW anchorage (1 mm, 2 mm, 3 mm, 4 mm, 5 mm)	Vertical
2	Settlement of TY anchorage (1 mm, 2 mm, 3 mm, 4 mm, 5 mm)	Vertical
3	Settlement of MW tower (1 mm, 2 mm, 3 mm)	Vertical
4	Settlement of TY tower (1 mm, 2 mm, 3 mm)	Vertical

4 SETTLEMENT TEST RESULTS AND DISCUSSIONS

4.1 Anchorage settlement tests

Main cables are key load-carrying components in a suspension bridge. The configuration change of main cables would lead to load re-distribution and affect the equilibrium of the whole bridge system. Table 2 lists the variation of cable force at the MW south side with the anchorage settlement from 1 mm to 5 mm. As shown in the Table 2, the MW anchorage settlement would increase the cable force near the MW anchorage greatly while it has only a little effect on the cable force near the TY anchorage. The change of cable force near the MW anchorage reaches about 5.0% to 20.8% for the settlement from 1 mm to 5 mm, respectively, compared with the initial cable tension force of 140 N. Similarly, the variation of cable force near the TY anchorage is obvious when the TY anchorage settlement occurs but it does not affect the cable force near the MW anchorage considerably. The variation of cable strain with the TY anchorage settlement of 1 mm and 5 mm is illustrated in Figure 3(a). Figure 3(b) shows the variation of strain of the tower legs for the TY anchorage settlement of 1 mm and 5 mm. It also can be found that the tower strains near the anchorages where the settlement occurred undergo fairly great changes during the anchorage settlement.

Table 2. Variation of cable force due to MW anchorage settlement.

Sensor	1 mm		2 mm		3 mm		4 mm		5 mm	
Location	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)
Lcs1	6.93	5.00	11.83	10.00	17.90	15.01	22.39	19.98	29.14	24.89
Lcs2	1.24	1.14	1.99	2.29	2.98	3.44	3.86	4.53	4.91	5.55

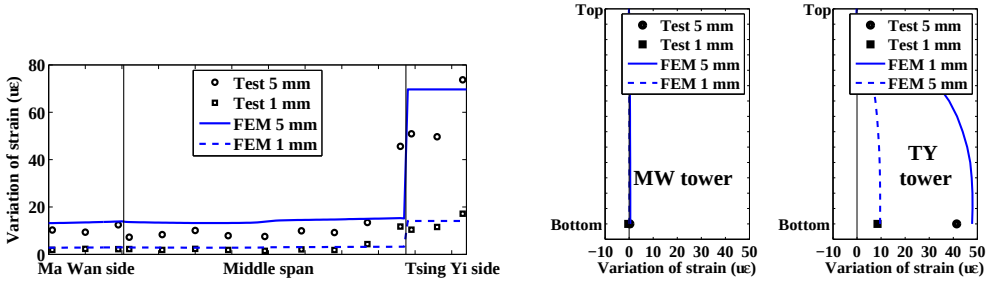
Note: F_{test} denotes the force measured from test and F_{FEM} denotes the force predicted by FEM

The variation of deck displacement in mid-span with MW anchorage settlement is listed in Table 3. The test results match well with those computed from FEM. And it was found that the two anchorage settlement scenarios both influence the vertical displacements of the bridge deck. Figure 4 displays the variation of strain in the bridge deck for the MW and TY anchorage settlements of 1 mm and 5 mm, respectively. The changes of strains in the bridge deck near the two towers and in the middle of the bridge deck are quite sensitive to the anchorage settlement no matter which anchorage has settlement. The measured and computed results also demonstrated that the variations of cable forces, cable strains, tower strains, deck displacements, and deck strains were linear with the anchorage settlement.

Table 3. Variation of deck displacement with MW anchorage settlement.

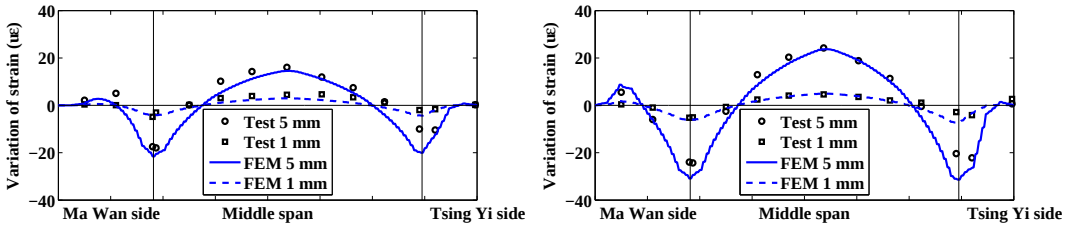
Sensor	1 mm		3 mm		5 mm	
Location	d_{test} (mm)	d_{FEM} (mm)	d_{test} (mm)	d_{FEM} (mm)	d_{test} (mm)	d_{FEM} (mm)
Ld2	0.82	0.66	1.93	1.98	3.22	3.30

Note: d_{test} denotes the displacement measured from tests; d_{FEM} denotes the displacement predicted by FEM



(a) (b)

Figure 3. Variation of cable and tower strain due to TY anchorage settlement: (a) cable; (b) tower.



(a) (b)

Figure 4. Variation of deck strain due to anchorage settlement: (a) MW anchorage; (b) TY anchorage.

From the anchorage settlement tests, it was found that the deck strains measured from sensors FD6 to FD10 and FD13 and all the three deck displacements measured from the displacement transducers are sensitive to both the MW and TY anchorage settlement. The cable strain from sensor FC3, the tower strain at the bottom of the MW tower legs, and the cable force in the MW side span are sensitive to the MW anchorage settlement, while the cable strain from FC15, the tower strain at the bottom of the TY tower legs, and the cable force in the TY side span are sensitive to the TY anchorage settlement. Therefore, a total of 15 measurement points in the above locations are selected to detect the anchorage settlement in the subsequent section.

4.2 Tower settlement tests

Table 4 lists both measured and computed results on the variation of cable force near the two anchorages with the settlement from 1 mm to 3 mm. The settlement of the MW tower reduces the cable force near the MW anchorage significantly but it has only a little effect on the cable force near the TY anchorage. The changes of cable force reach about -2.6% and -10.5% near the MW anchorage for the settlement of 1 mm and 3 mm respectively. Similarly, the variation of cable force near the TY anchorage is obvious when the TY tower subsides but the variation of cable force near the MW anchorage is small. The variations of strain in the cable and tower with the MW tower settlement of 1 mm and 3 mm are illustrated in Figure 5. The cable strains in the MW side span and the tower leg strains change greatly when the MW tower has the settlement. Similarly, the cable strains in the TY side span and the TY tower strains suffer great changes when the TY tower has the settlement. Similar to the anchorage settlements, it was found that the variations of cable forces, cable strains, deck strains, tower strains and deck displacements are almost linear with tower settlements.

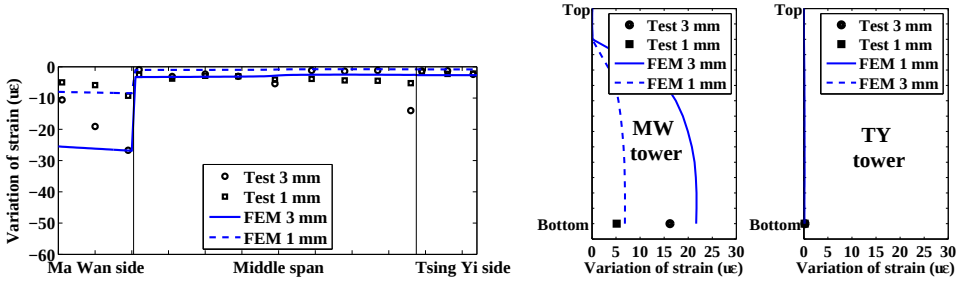
Table 4. Variation of cable force with TY tower settlement.

Location	1 mm		2 mm		3 mm	
	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)	F_{test} (N)	F_{FEM} (N)
Lcs1	-1.93	-1.85	-3.53	-4.05	-5.52	-6.66
Lcs2	-6.89	-8.65	-17.19	-17.81	-27.78	-28.58

As shown in Table 5, the displacements of the bridge deck are affected by the MW tower settlements. And it was also found in the TY tower settlement that the configuration of the bridge deck was changed. However, the influence of the TY tower settlement on deck displacement is relatively larger than that of the MW tower settlement. Figure 6 depict the variation of strain in the bridge deck with the MW tower settlement and the TY tower settlement of 1 mm and 3 mm, respectively. The changes of strain in the bridge deck near the MW anchorage, the MW tower, and TY tower are sensitive to the MW tower settlement. Similarly, the strains in the bridge deck near the TY anchorage, the TY tower, the MW tower, and in the middle change greatly when the settlement happens in the TY tower. Based on the above observations and discussions, one may also find that the 15 measurement points selected for detection of anchorage settlements are also sensitive to the tower settlements.

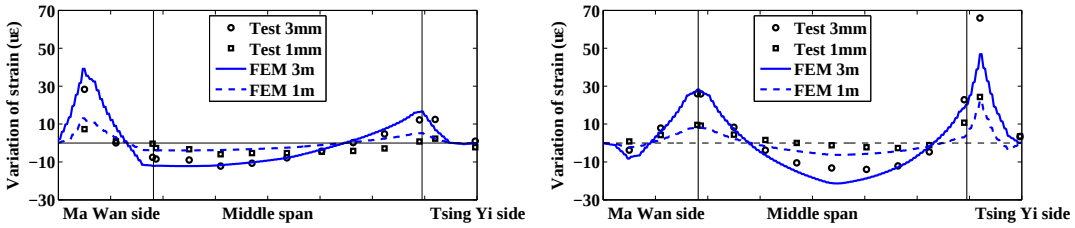
Table 5. Variation of deck displacement with MW tower settlement.

Location	1 mm		2 mm		3 mm	
	d_{test} (mm)	d_{FEM} (mm)	d_{test} (mm)	d_{FEM} (mm)	d_{test} (mm)	d_{FEM} (mm)
Ld2	-1.32	-1.22	-2.59	-2.32	-4.23	-3.85



(a) (b)

Figure 5. Variation of cable and tower strain due to MW tower settlement: (a) cable; (b) tower.



(a) (b)

Figure 6. Variation of deck strain due to tower settlement: (a) MW tower; (b) TY tower.

5 SETTLEMENT DETECTION METHOD

The agreement of the computed results with the measured results demonstrates that the FEM established is a good representation of the physical bridge model. Assume that the structural response changes are due to bridge support settlements only and the settlements are small so that the superposition principle can be applied.

$$\mathbf{y} = \mathbf{A}\mathbf{x} \tag{1}$$

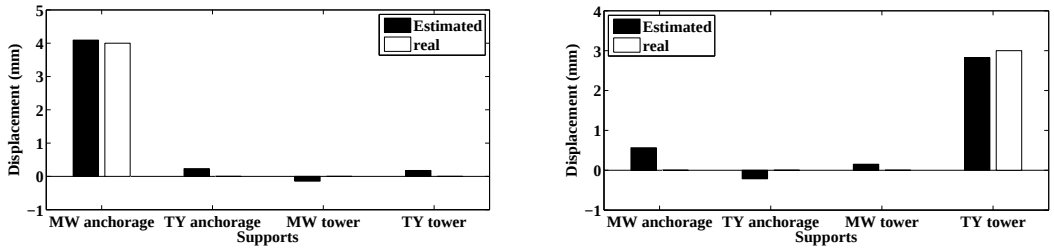
where **y** represents the *n*-dimensional structural response vector; **x** is the *m*-dimensional support settlement vector, where $n \geq m$ in this study; and **A** is the $n \geq m$ coefficient matrix reflecting the relationship between the structural response and the magnitude of settlement. The pseudo-inverse of the coefficient matrix is used to obtain the support settlement vector

$$\mathbf{x} = \mathbf{C}\mathbf{y} \tag{2}$$

where matrix **C** is the so-called left inverse of matrix **A** equal to $[\mathbf{A}^{\mathrm{T}}\mathbf{A}]^{-1}\mathbf{A}^{\mathrm{T}}$ provided that the columns of **A** are linearly independent. Furthermore, matrix **A** shall be well-conditioned.

For the present study, the settlement magnitudes of the MW anchorage, the TY anchorage, the MW tower, and the TY tower are denoted as x_{MWA}, x_{TYA}, x_{MWT} and x_{TYT}, respectively, to form the support settlement vector. The structural response vector consists of the 15 measured bridge responses mentioned above. The elements in matrix **A** can be attained in terms of the FEM of the bridge. Because the types of structural responses selected are different, the units of the structural responses shall be selected properly or the normalization of the structural response shall be conducted. For the present study, the units of the force, strain and displacement are N, με, and mm and no normalization is needed.

Two test results are used to verify the settlement detection method proposed above: 4 mm settlement of the MW anchorage, 3 mm settlement of the TY tower. Figure 7 presents both the estimated and real magnitudes of the two support settlements due to the MW anchorage and TY tower settlements. The estimated magnitudes of the settlements agree well with the real values. There are only 2.25% and -5.89% errors compared with the real values of the MW and TY tower settlements. These errors are partially attributed to measurement and modeling errors.



(a) (b)

Figure 7. Comparison of estimated settlement with real settlement: (a) MW anchorage settlement; (b) TY tower settlement.

6 APPLICATION TO PROTOTYPE BRIDGE

The Tsing Ma Bridge in Hong Kong is the prototype of the test-bed used in this study. A 3D SHM-oriented finite element model of the full-scale Tsing Ma Bridge has been established by using a commercial software package and displayed in Figure 8 (Liu et al., 2009).

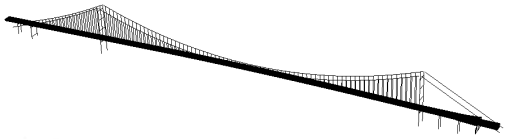


Figure 8. SHM-oriented FEM of Tsing Ma Bridge.

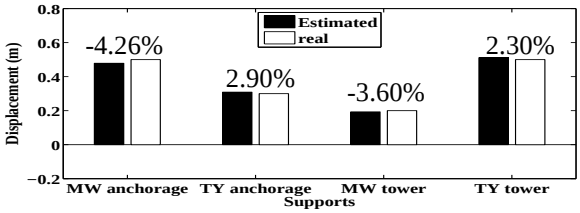


Figure 9. Estimation of multi-support settlements.

Although the locations of sensors may be determined through extensive sensitivity studies using the SHM-oriented FEM of the prototype, the results from the test-bed can be directly used instead. In this connection, the strains in the middle of the deck and near the two towers, the strains at the bottom of tower legs, the strains and tension forces in both ends of the main cables, and the displacements of the deck in the main span are selected for detection the support settlements as an application of the proposed method.

A multi-support settlement case, in which the MW anchorage, TY anchorage, MW tower and TY tower have the settlement magnitude of 0.5 m, 0.3 m, 0.2 m, and 0.5 m, respectively, is considered to assess the feasibility and accuracy of the proposed method for the prototype bridge. The computed structural responses from the FEM of the prototype are artificially contaminated with 2% random measurement noise. The estimated settlement results and the percentage errors are illustrated in Figure 9. The differences of the estimated settlement magnitudes from the preset values are only -4.26%, 2.90%, -3.60%, and 2.30% for the MW anchorage, TY anchorage, MW tower and TY tower, respectively.

7 CONCLUSIONS

This paper presents both the experimental and numerical investigations to examine the effects of support settlements on bridge structural responses and to explore the possibility of using the measured structural responses for detection of the support settlements of the bridge. The experimental results provided the locations and types of structural responses which are most sensitive to support settlements. It was also found that the variations of the structural responses are in linear relationship with the support settlements. A practical method for detecting the support settlements of the bridge has been proposed and verified by the test results from the test-bed and the numerical results from the prototype FEM. The results show that the proposed method is applicable and reliable.

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