ABSTRACT

Global positioning system (GPS) technology is an emerging tool for measuring both static and dynamic displacement responses of long span cable-supported bridges to strong winds. A systematic investigation is being carried out at The Hong Kong Polytechnic University on the application of GPS for wind and structural monitoring of large civil structures. A motion simulation table is first developed to assess the displacement measurement accuracy of GPS. Wind data recorded by the anemometers and displacement responses measured by the GPS of the Tsing Ma suspension bridge in Hong Kong are then analyzed. The statistical relationships between wind speed and wind-induced bridge displacement response are finally explored for wind-resistant performance assessments of the bridge. The overall conclusion is that GPS technology is a useful tool for wind and structural monitoring of long span cable-supported bridges.

KEYWORDS: WIND, LONG SPAN BRIDGE, STRUCTURAL MONITORING, GPS

Introduction

Many wind-sensitive large civil structures appear in recent years. Wind and structural monitoring becomes an imperative and challenging task to assure the structures function properly during their long service lives and to prevent them from sudden failure or fatal disaster during strong winds. Structural displacement is a key parameter to assess functionality and safety of large civil structures. Due to its global coverage and continuous operation under all metrological conditions, Global Positioning System (GPS) technology has become a useful tool for measuring both static and dynamic displacement responses of large civil structures to gust wind. Among others, Lovse et al. (1995), Tamura et al. (2002) and Breuer et al. (2002) applied the GPS technology to measure displacement responses of slender structures. Ashkenazi and Roberts (1997), Fujino et al. (2000), Nakamura (2000) and Wong et al. (2000) applied the GPS technology to measure displacement responses of long-span cable-supported bridges. Feasibility studies related to the application of GPS for displacement response measurements of tall buildings include those by Kijewski and Kareem (2001), Celebi and Sanli (2002), Chen et al. (2002) and Ogaja et al. (2001), along with other examples in the literature. The accuracy of GPS for dynamic displacement measurement actually depends on many factors such as data sampling rate, satellite coverage, atmospheric effect, multi-path effects, and GPS data processing methods. It is still questionable whether GPS could provide the measurement accuracy of dynamic displacement to subcentimeter-millimeter level. Moreover, wind and structural health monitoring systems (WASHMS) including GPS have been installed in a number of large civil structures, but it is not clear how to use GPS data to assess the performance of structures under strong winds.
In this regard, a systematic investigation is being carried out at The Hong Kong Polytechnic University on the application of GPS for wind and structural monitoring of wind-sensitive large civil structures. A motion simulation table is first developed to assess the displacement measurement accuracy of GPS. Wind data recorded by the anemometers and displacement responses measured by the GPS of the Tsing Ma suspension bridge in Hong Kong are then analyzed. The statistical relationships between wind speed and wind-induced bridge displacement response are finally explored for wind-resistant performance assessment of the bridge.

**Motion Simulation Table**

In reality, it is important to measure wind-induced dynamic displacement response for tall buildings, mainly in the horizontal plane, and for long span bridge decks, mainly in the vertical plane. The fundamental frequency of a tall building and a long span cable-supported bridge in one direction may not be the same as that in the other direction. In addition to the two perpendicular motions, the torsional motion of a tall building and a bridge deck may also occur. Furthermore, new types of GPS receivers come along from time to time. The assessment of measurement performance of various types of GPS for wind-induced displacement response of large civil structures using the motion simulation table may become a routine job. Therefore, an advanced motion simulation table is desirable for calibration works of GPS (Figure 1). The motion simulation table as a test bed may also facilitate the development of new data processing methods.

![Figure 1: Schematic Diagram of Motion Simulation Table Tests](image)

**Table Configuration and Control System**

The major components of the motion simulation table include a movable platform, two ball screws, two servomotors, an electronic control system, a power terminal box, a
supporting frame, and a computer system with a 16-channel data acquisition system. The table can be turned over 90° and 2-D motion in the vertical plane can then be generated. To ensure that the motion simulation table can reproduce the targeted input motion accurately, a sophisticated control system is incorporated in the table. The 16-channel data acquisition system interfaces with a main computer program running on the COMPAQ desktop.

**Table Characteristics**

The motion simulation table is able to generate sinusoidal wave, circular wave, white noise random wave and any other waves defined by input wave time histories in two perpendicular directions with an upper frequency to 2 Hz. This frequency range covers the dominant natural frequencies of most tall buildings and long span cable-supported bridges. The maximum stroke (displacement) of the movable platform is ±50 mm in both directions for frequencies less than or equal to 0.8 Hz, whereas the maximum stroke is ±20 mm for frequencies more than 1.6 Hz. For frequencies between 0.8 Hz and 1.2 Hz, the maximum displacement is ±35 mm. For frequencies between 1.2 Hz and 1.6 Hz, the maximum stroke is ±25 mm. Further details can be found in Chan et al. (2006).

**Assessment of Measurement Accuracy of GPS**

Before applying the GPS technology to civil structures, calibrations are essential to assess the GPS performance in dynamic displacement measurements. The calibration tests using the 2-D motion simulation table were thus carried out on the site of Pak Shek Kok in Hong Kong in 2004. The calibration tests included the static tests to find the background noise of GPS measurement, and the dynamic tests to assess the dynamic displacement measurement accuracy of the GPS with input sinusoidal motion and wind-induced dynamic displacement responses of a long span bridge measured during typhoons in the field.

**Test Site**

In order to identify the best possible performance of the GPS used, one important selection criterion of a test site is its visibility in all directions from a satellite mask angle. This mask angle is in general taken as 15° (Hofmann-Wellenhof et al., 2001; Wolf and Ghilani, 2002). The test site of Pak Shek Kok was therefore selected. The neighboring buildings and mountains were relatively low and at a great distance to the test site, and the maximum elevation angle was estimated at less than 12° with respect to the antenna base.

**Hardware and Software Configurations**

The field calibration work was conducted for three days in June 2004. The GPS receivers of Leica GX1230 and AT504 choke ring antennas were the major components of the measurement instrumentation (Figure 1). One of the choke ring antennas was installed on the movable platform of the motion simulation table as a rover station while the other was fixed on a tripod approximately 12 m away from the simulation table as a reference station. For time synchronization between the GPS and the motion simulation table, another GPS receiver (Ashtech GG24) was connected to the computer to synchronize the computer clock to the atomic clock. Raw GPS observations were logged into the 32 Mb CF cards at 20 Hz, which is the highest sampling rate of the GPS used at both the reference and rover stations. The raw data were then transferred to a laptop computer on site regularly for post-processing. The
motions of the platform were also recorded and transferred to the laptop computer for comparison with the GPS results. The raw GPS data were first converted to RINEX format with Leica’s Geo-Office software. The in-house software developed by the Department of Land Surveying and Geo-Informatics at The Hong Kong Polytechnic University was then used to process the GPS raw data in kinematic mode to calculate the position of the rover antenna at each epoch. The cut-off angle of the GPS data was 15°. The in-house software can also output other relevant information such as the relative dilution of precision (RDOP) value and least squares residuals for quality control purpose of the calibration work. The dynamic trajectory in the GPS coordinate system (WGS84) was finally transformed into the motion simulation table coordinate system.

**Test Cases**

The calibration work was carried out in three phases. In the first phase, the background noise in GPS measurements was measured for 9 hours with the rover station being stationary. The GPS data were then analyzed to find basic characteristics of the background noise on the site, from which a proper bandwidth filtering scheme was designed for processing the GPS data recorded in dynamic mode. In the second phase of calibration work, sensitivities of the GPS to the amplitude and frequency of one-dimensional sinusoidal motion were investigated. In the third phase of calibration work, the capability of the GPS used in tracking complex motions was examined.

![Figure 2: Time Histories of Background Noise in GPS Measurements](image)

**Background Noise in GPS Measurements**

When the GPS antennas at both the rover and reference stations are stationary, any displacements reflected in the GPS measurements can be considered as background noise (Kijewski, 2003). Since the baseline in the measurement is very short, most of the GPS errors are cancelled when using double-differencing data processing technique, and the errors remaining in the background noise are mainly due to multipath effect. Figure 2 shows 30 minutes-long GPS-measured X, Y (horizontal) and Z (vertical) coordinates (background noise) when the antenna on the platform was stationary. The background noise fluctuates between -5.9 and 5.1 mm in the X-direction, between -4.4 and 6.3 mm in the Y-direction, and between -20.5 and 4.7 mm in the Z-direction. The standard deviations of the background noise are 1.358 mm, 1.061 mm and 2.894 mm in the X, Y and Z-directions, respectively. The spectral analysis of the background noise records manifests that the background noise contains plentiful frequency components but the dominant noise energy distributes over a relatively
low frequency range. The probability analysis demonstrates that the background noise follows the Gaussian distribution closely.

A tenth order elliptic bandpass filter is applied to dynamic displacement measurement data from the GPS to reduce the background noise. For sinusoidal motion, a bandwidth filtering scheme, depending on the frequency $f_{dom}$ of the table motion, is implemented in this study.

$$\frac{f_{dom}}{5} \leq \text{frequency range interested} \leq 5 \times f_{dom}$$

(1)

This bandwidth filtering scheme covers a relatively large frequency range and is easily implemented. The band-pass filter is applied to the background noise data obtained from the static tests. The results show that after the filtering, the mean values of the background noise are almost eliminated and all other statistics are reduced significantly. For example, after the filtering the maximum standard deviation of the background noise is 0.51 m only in the X-direction.

**1-D Sinusoidal Motion in Horizontal Direction**

The dynamic displacement measurement error in the GPS measurement is defined by

$$\text{Error(\%)} = \frac{x_g - x_t}{x_t} \times 100\%$$

(2)

where $x$ represents one of the statistics such as the minimum peak displacement, the maximum peak displacement or the displacement standard deviation (Std). The subscript of “g” refers to the GPS while “t” refers to the motion simulation table. As the peak in each cycle of sinusoidal motion can demonstrate the peak tracking ability of the GPS, the average minimum peak displacement (Amin) and the average maximum peak displacement (Amax) are used in this study. Amin is defined as an average value of the trough of all cycles in the time history. Amax is defined as an average value of the crest of all cycles in the time history.

![Figure 3: Comparisons of Horizontal Sinusoidal Motions before and after Filtering (Frequency = 0.025 Hz)](image-url)
Figure 3 depicts sinusoidal motions of 0.025 Hz with amplitude of 2 mm, 5 mm, 10 mm, 20 mm and 40 mm in the Y-direction generated by the motion simulation table and those recorded by the GPS before and after the filtering. It can be seen that without the filtering, the GPS can satisfactorily track the sinusoidal motions of 20 mm amplitude and above. With the filtering, the GPS can satisfactorily trace the sinusoidal motions of 5 mm amplitude and above. The measurement errors calculated according to Equation (2) manifest except for the sinusoidal motion of 2 mm amplitude, the measurement errors are all very small for the sinusoidal motions of other amplitudes. There are less than 1.5% error in the average minimum peak displacement (Amin), less than 2% error in the average maximum peak displacement (Amax), and less than 0.6% error in the displacement standard deviation (Std). The absolute error in the displacement standard deviation decreases with increasing motion amplitude. One may conclude that the performance of the GPS is satisfactory when the amplitude of the sinusoidal motion of 0.025 Hz is not less than 5 mm.

Sinusoidal motions of 20 mm amplitude with frequency of 0.1 Hz, 0.5 Hz, 1 Hz and 1.8 Hz in the Y-direction, generated by the motion simulation table, are compared with those recorded by the GPS with the filtering. The comparison shows that the GPS can satisfactorily track the sinusoidal motions generated by the table except for the motion of 1.8 Hz. The tracking accuracy of the GPS is reduced with increasing frequency. The measurement errors in the average maximum peak displacement, the average minimum peak displacement, and the displacement standard deviation are 3.52%, 4.90%, and 3.84% respectively, for the sinusoidal motion of 1 Hz. Since this error level is acceptable for most large civil structures, one may conclude that the performance of the GPS is satisfactory when the frequency of the sinusoidal motion of 20 mm amplitude is not greater than 1 Hz.

1-D Sinusoidal Motion in Vertical Direction

For the application of GPS technology to a long span cable-supported bridge, the measurement of vertical displacement of bridge deck is one of the main concerns. Therefore, the measurement accuracy of the GPS in the vertical direction is assessed in terms of sinusoidal motion. 0.5 Hz sinusoidal motions with amplitudes of 5 mm, 10 mm, 20 mm and 40 mm generated by the table in the vertical direction are compared with those recorded by the GPS. With the filtering, the vertical displacement time histories recorded by the GPS become very close to those generated by the motion table. The measurement errors are all less than 8% even for the sinusoidal motion of 5 mm amplitude only. However, in consideration that the 3σ value of the background noise after the filtering is 3.66 mm in the Z-direction, it will be conservative to say that the performance of the GPS is satisfactory in the vertical displacement measurement when the amplitude of 0.5 Hz sinusoidal motion is not less than 10 mm. 20 mm sinusoidal motions in the vertical direction generated by the table with frequencies of 0.025 Hz, 0.1 Hz, 1 Hz and 1.8 Hz, respectively, are also compared with those recorded by the GPS with the filtering. Similar to the case of horizontal motion, the GPS performance is the worst when the motion frequency is 1.8 Hz. The measurement errors in the average maximum peak displacement, the average minimum peak displacement and the standard deviation displacement are all more than 22%. For the motion frequency not greater than 1 Hz, the measurement errors in the displacement standard deviation are less than 12%. Clearly, the measurement accuracy of the GPS in the vertical direction is much lower than that in the horizontal direction. Only if 14% measurement error is acceptable, can one say that the performance of the GPS is satisfactory in the vertical displacement measurement when the frequency of 20 mm sinusoidal motion is not greater than 1 Hz.
Wind-Induced Vertical Dynamic Response of Tsing Ma Bridge Deck

The measurement accuracy of the GPS is also assessed using wind-induced vertical dynamic displacement response of the Tsing Ma bridge deck measured during Typhoon Victor in 1997. The input dynamic displacement time history to the motion simulation table in the vertical direction was extracted from the measured vertical acceleration response of the bridge deck at mid-span through double integrations. The vertical dynamic displacement time history of 1800 seconds has a maximum value of 32 mm. Since the first and second symmetric modes of vibration of the bridge were found to occur at a frequency of 0.139 Hz and 0.241 Hz, respectively (Xu et al., 1997), a band-pass filter of lower and upper cut-off frequencies of 0.05 Hz and 0.6 Hz was applied to both the input vertical dynamic displacement time history and the GPS-recorded vertical dynamic displacement time history. The results show that the wind-induced vertical dynamic displacement response of the bridge deck recorded by GPS is similar to that generated by the table. The measurement errors are all less than 7%. The GPS has an ability to track dynamic displacement responses of a long span cable-supported bridge in the vertical direction.

Application of GPS Technology to Tsing Ma Bridge

Background

The Tsing Ma Bridge in Hong Kong is the longest suspension bridge in the world carrying both highway and railway (Figure 4). It is the key structure of the most important transportation network in Hong Kong that links the Hong Kong International Airport to the commercial centers of Hong Kong Island and Kowloon. The Tsing Ma Bridge is also located in one of the most active typhoon prone regions in the world. Therefore, the Hong Kong Highways Department installed a Wind And Structural Health Monitoring System (WASHMS) and a Global Positioning System-On-Structure Instrumentation System (GPS-OSIS) in the Tsing Ma Bridge in 1997 and 2000 respectively (Wong et al., 2000, 2001).

![Figure 4: Distribution of Anemometers in Tsing Ma Bridge](image-url)

As shown in Figure 4, there are a total of six anemometers installed in the Tsing Ma Bridge with two ultrasonic anemometers at the middle of main span of the bridge, two mechanical anemometers at the middle of the Ma Wan side span, and one mechanical anemometer each on the top of the Tsing Yi tower and the Ma Wan tower. To prevent disturbance from the bridge deck on natural wind, the anemometers at the deck level are respectively installed on the north side and the south side of the bridge deck via a boom of 8.965 m long from the leading edge of the deck. The displacements of the Tsing Ma Bridge in longitudinal, lateral and vertical directions are mainly monitored by GPS. The components of the bridge implemented with the GPS receivers include bridge towers, main cables and bridge
deck. There are totally 14 GPS monitoring stations installed in the mentioned three major components and their distribution is shown in Figure 5.

Although the WASHMS and the GPS-OSIS installed in the Tsing Ma Bridge have demonstrated various degrees of successes in providing useful field measurement data for research and verification of original design, wind-resistant performance assessment of the bridge based on the data from the WASHMS has not been explored in detail. In the following, strong wind data and wind-induced bridge displacement response data recorded from both anemometers and GPS stations during the period from June 2003 to September 2005 are analyzed. The statistical relationships between strong wind and wind-induced bridge displacement response at the locations of GPS stations are then established for different displacements and wind directions. The obtained statistical relationships shed light on displacement behavior of the bridge under strong winds and provide the information for comfort and deflection assessment of the bridge under winds, which are associated with the comfort of road vehicles and human bodies. The statistical relationships can also be taken as reference to verify wind tunnel test results. Furthermore, the statistical relationships will be extended through the computer simulation to assess wind-resistant performance of the bridge under extreme wind speeds.

**Statistical Analysis of Strong Wind Data**

Only two types of relatively strong wind events are considered: typhoons signal No. 3 and above hoisted; and strong monsoons issued by the Hong Kong Observatory (HKO). The original wind data recorded by anemometers at a sampling frequency of 2.56 Hz are then pre-processed. The main functions of the data pre-processing are (1) to eliminate unreasonable data with abnormal magnitude for propeller anemometer at the Ma Wan tower; (2) to decide which anemometer, on the south or on the north, at the deck level shall be considered for each typhoon and monsoon events; (3) to eliminate unreasonable data with abnormal magnitude for ultrasonic anemometers at the middle of the main span; and (4) to obtain 10-minute and hourly mean wind speeds and wind turbulence components from the original wind data recorded by the ultrasonic anemometers.

From statistical analysis of strong wind data, the mean wind direction, mean wind speed, mean wind incidence, turbulence components, turbulence intensities, wind spectra, and integral length scales are obtained for the time interval of both ten-minutes and one hour. Only the statistics of wind data from the two ultrasonic anemometers at the middle of main span of the bridge deck are presented in this paper because they are used for the establishment of statistical relationships between wind speed and wind-induced bridge displacement response. Figure 6a depicts the frequency distribution of ten-minute mean wind direction within a 22.5° sector from typhoon wind records. It can be seen that the dominant wind direction is in the sector of N. Figure 6b shows the distribution of ten-minute mean wind
speed. It can be seen that the maximum ten-minute mean wind speed measured at the bridge deck level is 22.67 m/s when the wind blows from SW.

![Polar plot of mean wind direction](image1)

![Polar plot of mean wind speed](image2)

(a) Polar plot of mean wind direction  (b) Polar plot of mean wind speed

Figure 6: Statistics of Typhoon Wind Data from Ultrasonic Anemometers

Turbulence intensity is defined as the ratio of the standard deviation of fluctuating wind to the mean wind speed for a given duration. Turbulence intensity reflects the intensity of fluctuating wind and it is an important parameter in the determination of wind-induced dynamic response of long span cable-supported bridges. The turbulence intensities will be highly unreliable at very low mean wind speeds and shall be eliminated. Therefore, the threshold value of 5 m/s ten-minute mean wind speed is applied to the wind data from the ultrasonic anemometers at the deck level. Figure 7 shows the relationship between ten-minute mean wind speed and longitudinal turbulence intensity for the sectors of N and SSE for the typhoon events. It can be seen that turbulence intensity varies with mean wind speed. The mean value of the turbulence intensities over the concerned mean wind speed range is 0.153 with a standard deviation of 0.038 for the N-direction and 0.108 with a standard deviation of 0.029 for the SSE-direction.

![Variation of Turbulence Intensity with Ten-minute Mean Wind Speed](image3)

(a) N direction  (b) SSE direction

Figure 7: Variation of Turbulence Intensity with Ten-minute Mean Wind Speed (Typhoon Events)

**Statistical Analysis of Bridge Displacement Data**

The original bridge displacement data recorded by GPS stations at a sampling frequency of 10 Hz are pre-processed first. The main steps of the pre-processing of GPS data include (1) to convert the GPS data from the HK80 geographic coordinate to the Universal Transverse Mercator (UTM) grid coordinate; (2) to compute the bridge displacement...
coordinate with respect to the reference coordinate measured during the installation of the GPS systems; (3) to eliminate unreasonable data with abnormal magnitude caused by an abrupt change in the number of satellites or unsatisfactory geometry configurations; (4) to obtain bridge displacement responses in three orthogonal directions; (5) to filter displacement response time histories using a low-pass filter of upper frequency 1 Hz to maintain high quality data for subsequent analysis; and (6) to decompose mean and dynamic displacements from the total response measurements recorded by GPS.

As the mean displacement response of the bridge recorded by GPS may be affected by GPS background noise and temperature in addition to mean wind, the effects of GPS background noise and temperature on the bridge mean displacement response shall be assessed. Furthermore, the dynamic displacement response of the bridge recorded by GPS may also be affected by moving road vehicles, moving trains, and GPS background noise in addition to wind. In this regard, calibration tests using the 2D motion simulation table were first carried out on the site of Tsing Ma Bridge in September 2005. The calibration tests included static tests and dynamic tests to find the possible effects of GPS background noise on the bridge displacement responses. The calibration results show that the ten-minute mean value of GPS background noise averaged in one hour is 0.07 mm in the horizontal direction and 0.11 mm in the vertical direction only. The averaged ten-minute standard deviation of GPS background noise is 1.62 mm in the horizontal direction and 3.67 mm in the vertical direction. Therefore, the effect of GPS background noise on the mean displacement response is small and can be neglected when strong wind events are considered. The standard deviations of the dynamic displacement response less than 4.5 mm and 9.7 mm in the respective horizontal and vertical direction are not included in the statistical analysis to avoid the effect of GPS background noise on dynamic displacement response.

To assess the effect of temperature on the mean displacement response of the bridge, the measurement data from both the temperature sensors and GPS stations were selected for the forty day with only small wind. By examining the variation of deck longitudinal, lateral and vertical mean displacements and cable longitudinal, lateral and vertical mean displacements, and tower longitudinal and lateral mean displacements with bridge effective temperature, one could find that temperature affects mainly the deck and cable mean displacements in the vertical and longitudinal direction, the tower mean displacement in the longitudinal direction. On the other hand, by examining the variation of deck longitudinal, lateral and vertical mean displacements and cable longitudinal, lateral and vertical mean displacements, and tower longitudinal and lateral mean displacements with mean wind speed, one could find that mean wind affects mainly the lateral mean displacements of the bridge deck, cables and towers. These observations provide a way of distinguishing temperature effects from wind effects on bridge mean displacements.

The statistical analysis of dynamic displacement response of the bridge revealed that the dynamic displacement responses of the bridge deck in the vertical direction, the cables in the vertical and longitudinal direction, and the bridge towers in the longitudinal direction within the frequency range less than 0.08 Hz are pre-dominated by railway-load effects. Therefore, wavelet decomposition is applied to the time histories of the relevant dynamic displacement responses of the bridge components to eliminate railway-load effects. The remaining concern is how to eliminate highway-load effects on the dynamic displacement response of the bridge. In this regard, the measurement data from both weight-in-motion (WIM) sensors and GPS stations were examined. Part of the measurement data which meet the requirements of weak wind and no passing trains were selected. The variation of dynamic
displacement response of the bridge with the traffic flow rate (TFR) was then assessed. It was found, for instance, that the average standard deviation of dynamic displacement response of the bridge deck in the mid-main span over the period of high TFR is about 5 mm in the lateral direction and about 11 mm in the vertical direction. By assuming wind- and traffic-induced displacements are uncorrelated with each other, the standard deviation of wind-induced dynamic displacement response ($\sigma_w$) can be determined by:

$$
\sigma_w = \sqrt{\sigma^2_{WT} - c^2}
$$  \hspace{1cm} (3)

where $\sigma_{WT}$ is the standard deviation of the total dynamic displacement response induced by both highway load and wind load; and $c$ is the highway-induced displacement standard deviation. The highway-induced standard deviation is often small or even can be neglected when the total displacement standard deviation is large enough. By Equation (3), less than 5% error is presented in wind-induced displacement standard deviation if the total displacement standard deviation is greater than or equal to $3.28 \times c$.

**Statistical Relationship between Wind and Wind-Induced Bridge Displacement**

The relationships between ten-minute mean wind speed and ten-minute wind-induced mean, dynamic, and total displacement responses of the bridge were explored, but only the relationship between ten-minute mean wind speed and ten minute wind-induced total displacement of the bridge deck and main cables is presented in this paper. All valid 10-minute mean wind speeds higher than 5 m/s and the corresponding 10-minute total displacements of the bridge deck and main cable are considered for the time period from June 2003 to September 2005. The power law function expressed by Equation (4) is adopted to fit the measurement data.

$$
\hat{D} = D \pm m \sigma_{D} = a \bar{U}^b
$$  \hspace{1cm} (4)

where $\hat{D}$ is the wind-induced total displacement of the bridge deck or main cable at a given section in a given direction; $\bar{D}$ represents the corresponding wind-induced mean displacement; $\sigma_{D}$ is the corresponding displacement standard deviation; $m$ is the statistical peak factor; $\bar{U}$ denotes the corresponding mean wind speed; $a$ and $b$ are the two parameters to be determined. 

Because of the nature of the problem, only the positive and non-zero value of $b$ in Equation (4) reflects the fact that the larger is mean wind speed, the larger wind-induced total displacement is. Hence, in the curve fitting the one-tail testing hypothesis concerning the value of parameter $b$ in the regression model is performed with a $t$-test significance 0.05 ($p$-value) and the two-tail testing hypothesis concerning the value of parameter $a$ in the regression model is performed with a $t$-test significance 0.05. The relationships between the 10-minute mean wind speed and the lateral total displacement recorded at the four positions of the bridge deck and the main cable are explored by using the regression model given in Equation (4). The results of the regressions for the mid-main span and the main cable in the lateral direction in the N direction sector are shown in Figure 8.

It is noted that the estimated exponents in the power law functions obtained for the lateral direction of the mid-main span and the main cable are 1.822 and 1.741 respectively. This means that the wind-induced total displacement in the lateral direction is approximately proportional to the squares of wind speed. When the mean wind speed is taken as 10 m/s, the lateral total displacements of the bridge deck at the mid-main span and the main cable estimated using the power law functions are -191.29 and -179.73 mm respectively. These results are similar to the summarization of the lateral mean displacement and the triple of the lateral standard deviation.
The statistical relationships between wind and wind-induced total displacement response at four locations of the bridge deck and main cable are also explored for wind direction sector NNW. The regression factors determined using the least-squares method with p-value less than 0.05 and R2 greater than 0.6 are accepted. The curve fitting fails for the measurement data from the deck section in the Ma Wan side span because the R2 is less than 0.6. Figure 9a illustrates the statistical relationships obtained by Equation (4) with the regression parameters obtained for the three locations of the bridge deck and the main cable in the lateral direction. It is noted from the figure that the total displacements are almost symmetric with respect to the middle of main span. It is also observed that the total displacement of main cable is similar to that of the mid-main span.

The statistical relationships between wind and wind-induced total displacement response of the bridge deck at the middle of main span for different wind directions are also investigated. The regression factors determined using the least-squares method with the p-value less than 0.05 and the whole regression with R2 greater than 0.6 are accepted for six wind directions only. By using Equation (4) and the regression factors obtained, the total displacement response at the middle of main span in the lateral direction for the six wind direction sectors are plotted in Figure 9b. From the figure, it can be noted that the total
displacement responses reach their most negative values in the N direction sector, whereas the most positive total displacements come from the SSE direction. It can also be observed that the relationship between wind and wind-induced total displacement response varies with the wind direction.

Wind-Resistant Performance Assessment of Tsing Ma Bridge

Background

There are three levels of wind-resistant performance assessments for the Tsing Ma Bridge. The first level is related to comfort and safety of road vehicles and trains running on the bridge in crosswind. In this regard, the comfort and deflection of the bridge shall be examined, respectively, in terms of the maximum acceleration and displacement responses of the bridge against the maximum allowable acceleration and displacement of the roadway and the railway. A high wind management system is introduced for traffic management of the Tsing Ma Bridge for the safety of road vehicles. The operation guidelines adopted for the bridge during high wind period specify that both the upper and lower decks shall be closed for all road vehicles except trains when the hourly-mean wind speed recorded on site is in excess of 165 km/hour. The second level of wind-resistant performance assessment is associated with the serviceability limit state (SLS) of the bridge, in which the bridge shall behave elastically and is expected to be serviceable immediately without the need for any repair. To examine the serviceability of the bridge, buffeting analysis of the bridge is necessary and the maximum total displacement response of the bridge under serviceability design wind speed shall be examined against the maximum allowable displacement. The third level refers to the ultimate limit state (ULS) of the bridge, in which the bridge may undergo large deformation in the post elastic range without substantial reduction in strength and the damage level of the bridge will be considered as economically and technically feasible to repair. In this level, the aerodynamic instability should be avoided and the critical wind speed should be confirmed.

Wind-resistant performance assessment of the bridge based on the data from the WASHMS has not been explored in detail. The number of sensors in the WASHMS is always limited for such a large structure and the locations of structural defects or degradation may not be at the same positions of the sensors. Possibility exists that the worst condition may not be directly monitored by the sensors, and the structural performance assessment could not rely on the data from the sensors only. Therefore, for the accomplishment of a complete wind-resistant performance assessment of the bridge, the WASHMS shall be combined with advanced computer simulation in an evolutionary way. For this purpose, a SHMS-based computer simulation of wind-induced responses of the bridge has been presented by the authors and verified by the measurement data to some extent. Nevertheless, this paper only discusses the application of GPS for wind-resistant performance assessment of the bridge in terms of bridge displacement response. Since the design hourly mean wind speed of the bridge is 50 m/s at the deck level for SLS and 65 m/s for ULS, the statistical relationships presented in the last section cannot be used to assess wind-resistant performance of the bridge under extreme wind speed because the maximum wind speed recorded by the WASHMS so far is much lower than the design wind speeds. The statistical relationship presented in the last section is therefore extended through the computer simulation to consider extreme wind cases. The statistical relationships presented in the last section is also extended for other deck sections of the bridge through the computer simulation in order to have a complete assessment
of the comfort and deflection of the bridge associated with the comfort of road vehicles and human bodies.

**SHMS-based Finite Element Model**

To facilitate a complete wind-resistant performance assessment of the bridge, a structural health monitoring system (SHMS)-based finite element model (FEM) is needed for a long suspension bridge so that stresses/strains in all important bridge components can be directly computed and some of them can be compared with the measured ones for verification. However, the currently-conducted buffeting analyses of long span bridges are often based on a simplified spine beam FEM of equivalent sectional properties (Xu et al., 2000). Such simplified model is effective to capture the dynamic characteristics and global structural behaviour of the bridge under strong winds without heavy computational effort. However, local structural behaviour linked to stress and strain, which is prone to cause local damage, could not be estimated directly. On the other hand, with the rapid development of information technology, the improvement of speed and memory capacity of personnel computers (PC) has made it possible to establish a SHMS-based finite element model for a long suspension bridge. In this regard, a SHMS-based finite element model has recently been established for the Tsing Ma Bridge, as shown in Figure 10, with significant modelling features of the bridge deck included for the good replication of geometric details of the as-built complicated deck (Zhang et al., 2007). The bridge was modeled using a series of beam elements, plate elements, shell elements, and others. The finite element model contains 12898 nodes, 21946 elements (2906 plate elements and 19040 beam elements) and 4788 Multi-Point Connections. The SHMS-based model has also been updated using the measured first 18 natural frequencies and mode shapes of the bridge with the updated parameters being material properties only because the geometric features and supports of bridge deck have been modelled in a great detail in the proposed SHMS-based FE model. It turns out that the updated complex FE model could provide comparable and credible structural dynamic modal characteristics.

![Figure 10: SHMS-based Finite Element Model of Tsing Ma Bridge](image)

**Computer Simulation of Buffeting-induced Responses**

Based on the established SHMS-based FEM, a numerical simulation procedure for buffeting induced responses of long suspension bridges has been proposed (Liu et al., 2007). Significant improvements of the proposed procedure are that the effects of spatial distribution of both buffeting forces and self-excited forces on a bridge deck structure are taken into account, as opposed to lumping all buffeting forces and self-excited forces at the centre of elasticity as in the case of an equivalent beam finite element model. Local strains and stresses in structural members of the bridge deck, which are prone to cause local damage, are
predicted directly using the mode superposition technique in the time domain. The field measurement data including wind, displacement, acceleration and stress recorded by the anemometers, GPS stations, accelerometers and strain gauges in the WASHMS installed on the Tsing Ma Bridge have been analyzed. The buffeting–induced displacement and acceleration responses at the locations of GPS stations and accelerometers have been computed using the mode superposition method and compared with the measured results. The buffeting-induced stress responses at the locations of strain gauges installed in the Tsing Ma Bridge have been computed through the modal stress analysis and compared with the measured ones. The comparative results show that the computed displacement, acceleration and stress time histories are similar in both pattern and magnitude with the measured ones. The SHMS-based finite element model of the bridge and the proposed computer simulation of buffeting-induced response offer a good tool for the extension of the statistical relationship presented in the last section to consider extreme wind cases.

**Extension of Statistical Relationships and Performance Assessment**

The statistical relationships between wind speed and wind-induced total displacement as developed before are extended for the extreme wind cases through the computer simulation. Figures 11a and 11b illustrate the extended statistical relationships between wind speed and wind-induced total displacement in the lateral and vertical direction of the bridge at the mid-main span for the SSE wind direction (the wind direction almost perpendicular to the bridge deck). As shown in the figures, the computed results match well with the measured data within a low wind speed range. When the mean wind speed is beyond about 15 m/s, only computed results are available. When the hourly-mean wind speed reaches 50 m/s, the total displacement response of the bridge at the mid-main span in the SSE wind direction is 2305 mm in the lateral direction and 860 mm in the vertical direction. Figure 12 further depicts the variations of the lateral and vertical total displacement responses along the bridge deck for wind velocity normal to the bridge deck but under different mean wind speeds. The maximum total displacement response of the bridge occurs at the mid main span.

For the Tsing Ma Bridge, the design hourly-mean wind speed for SLS is 50 m/s for a 120 year return period. The maximum allowable total displacement of the bridge in the lateral direction under the serviceability design wind speed is 2.9 m. Since the time period for collecting wind data on the site is not longer enough to examine the design wind speed for a
120 return period, the design wind speed of 50 m/s for a 120 return period is used for preliminary wind-resistant performance assessment. As shown in Figure 11a, the total displacement response of the bridge at the mid-main span is 2305 mm in the lateral direction under the design wind speed of 50 m/s at the deck level. To compare the computed maximum total displacement with the maximum allowable total displacement, the lateral total displacement movement of the bridge is within the tolerance movement according to the design criterion with respect to SLS. Similar exercises can be performed for other wind-resistant performance assessments of the bridge against other design criteria. One important point is that such exercises shall be updated as more data are available so as to yield an evolutionary wind-resistant performance assessment of the bridge.

![Graphs showing total displacement response](https://via.placeholder.com/150)

**Figure 12: Variation of Total Displacement Response along Bridge Deck**

### Conclusions

A systematic investigation on the application of GPS for wind and structural monitoring of long span cable-supported bridges has been carried out by Research Centre for Urban Hazards Mitigation of The Hong Kong Polytechnic University. An advanced 2D motion simulation table has been designed and manufactured for the assessment of the dynamic displacement measurement accuracy of GPS. A series of field tests were carried out in an open area using a motion simulation table and a GPS system in either a horizontal plane or a vertical plane. The test results showed that the background noise was dominated mainly by low frequency components. The background noise measured by the moving receiver was very close to that measured by the stationary receiver. For 1D sinusoidal in the horizontal plane and in the vertical direction, the GPS could measure dynamic displacements accurately if the motion amplitude was not less than 5 mm in the horizontal direction or 10 mm in the vertical direction, provided that the motion frequency was less than or equal to 1Hz. The test results demonstrated that GPS could trace wind-induced dynamic displacement responses measured from a long-span bridge deck in the vertical plane with satisfactory results.

To apply GPS technology for wind-resistant performance assessment of long suspension bridges, the Tsing Ma Bridge in Hong Kong has been taken as an example. The calibration tests using the 2D motion simulation table were carried out first on the site of the Tsing Ma Bridge to find the possible effects of GPS background noise on the bridge displacement measurements. The results demonstrated that the effect of GPS background noise on the mean displacement response could be neglected, but the threshold values should be applied to dynamic displacement responses to eliminate GPS dynamic background noise. The effect of temperature on the mean displacement and the effect of highway and railway loads on the dynamic displacement were then figured out. The results showed that the mean
wind would affect mainly the lateral mean displacements of the bridge deck, cables and towers. The effects of railway load on dynamic displacement responses could be eliminated by filtering out low frequency responses. The effects of highway load on dynamic displacement responses could also be statistically extracted so that wind-induced dynamic displacement response of the bridge could be eventually obtained.

Strong wind data and wind-induced bridge displacement response data recorded from both anemometers and GPS stations during the period from June 2003 to September 2005 have been analyzed for the Tsing Ma Bridge. The statistical relationships between mean wind speed and wind-induced bridge displacement response have been worked out. The obtained statistical relationships shed light on displacement behavior of the bridge under strong winds and provide the information for comfort and deflection assessment of the bridge under winds associated with the comfort of road vehicles and human bodies. The statistical relationships can also be taken as reference to verify wind tunnel test results. The statistical relationships obtained based on the measurement data could not be used to assess wind-resistant performance of the bridge under extreme wind speed because the maximum wind speed recorded by the WASHMS so far is much lower than the design wind speeds. The statistical relationships have been thus extended through the computer simulation to consider extreme wind cases. The statistical relationships have also been extended for other deck sections of the bridge through the computer simulation to have a complete assessment of the comfort and deflection of the bridge associated with the comfort of road vehicles and human bodies. Some examples have been given and such exercises could be updated as more data are available so as to yield an evolutionary wind-resistant performance assessment of the bridge.

Acknowledgements

The work described in this paper was financially supported by the Research Grants Council of Hong Kong through several CERG grants, The Hong Kong Polytechnic University through its niche area project on performance-based structural health monitoring of large civil engineering structures, the Hong Kong Highways Department through a contract research on bridge health and engineering. The support from the Lantau Fixed Crossing Project Management Office, the Hong Kong Highways Department, to allow the authors to access the measured data for academic purpose only is particularly appreciated. Any opinions and conclusions presented in this paper are entirely those of the authors.

References


