Using GPS to Measure the Response of the Forth Road Bridge to Wind and Temperature Loading

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Abstract

The interaction of wind and temperature loadings with the response of a bridge is investigated using Real-Time Kinematic (RTK) GPS. A two-day continuous test was carried out on one of the longest suspension bridges in the UK, the Forth Road Bridge. High grade geodetic GPS receivers sampling at a minimum rate of 10 Hz were installed at key locations on the bridge deck and also at two sites on top of the southern cable-supporting tower. Weather measurements (wind speed and direction, air temperature and humidity) were collected by a digital weather station set up at the midspan of the bridge and sampling over 30 seconds periods. To characterise the effect of wind and temperature loadings, apart from detecting the bridge's 3D natural vibration frequencies and deflection amplitudes, a novel hybrid low-high pass filter has been developed to increase the filter's stability and its capability to isolate the aerostatic movements of the bridge from the aerodynamic displacements. This paper demonstrates that careful data processing and analysis are essential, but given this, RTK GPS can be employed to measure both aerodynamic and aerostatic displacements of long suspension bridges induced by environmental factors.

Keywords**:** Real-Time Kinematic (RTK) GPS; Wind and temperature loadings; Aerostatic and aerodynamic displacements; Structural health monitoring; Digital signal processing (DSP)

1. Introduction

Wind loading is a major consideration during the design, construction and operational life of any bridge since it can cause aerostatic and aerodynamic stability problems of the structure [1]. The actual displacements induced by wind are often difficult to measure. However, this paper presents data obtained by GPS measurements, relating wind speed and direction, and air temperature with three dimensional (3D) movements for a long span suspension bridge.

The predicted mid-span lateral deflection of the Forth Road Bridge in the UK under extreme wind loading of 161km/h is 7.163m and the maximum vertical deflection due to live load bending, torque and temperature change of +30°F is 4.115m [2]. However, significant discrepancies can exist between the predicted values – for example from wind tunnel tests or from design calculations under wind action, and those of field measurements for the lateral displacements and vibration amplitude. For Japan's Akashi Kaikyo Bridge, a lateral displacement of 5.41m and vibration amplitude of 2.56m were predicted under wind load, but the actual values have been found to be 5.17m and 0.78m respectively [3]. Knowing the actual response of different components of a bridge as-built under wind action can provide invaluable information for future design, and provide a database of knowledge for structural health monitoring.

Monitoring the wind response of supporting towers is of particular significance in suspension bridge design. Less attention is paid to the wind analysis of supporting towers due to their higher stiffness when compared with the bridge deck [4]; their high slenderness characteristics make wind tunnel tests very expensive and it may still be very difficult to measure the actual structural response.

While studying the responses of a bridge to wind loadings, the wind speed and direction, and the acceleration and displacement of the responses need to be monitored. In the past a range of sensors (accelerometers, velocity gauges, strain gauges and anemometers) have been employed to observe wind-induced movements from an operational bridge [3,5] and relate them to design factors. Each of these techniques presents challenge. Obtaining relative displacement by double integral of accelerations which are measured by an accelerometer requires extensive post-processing. Furthermore this procedure can produce large position inaccuracies due to the amplification of any original sensor errors, such as scale factor error and random noise [5] - an intrinsic difficulty in the realisation of this technique for real-time structural monitoring. An accelerometer may have difficulty to accurately measure aerostatic movement of a bridge, but most importantly, relative measurement sensors do not have a solid reference system and therefore they cannot be employed to measure long-term structural displacements such as the foundation settlement, or gradual horizontal movements of the supporting towers.

Network, telecom and various in-built or mounted non-destructive intelligent sensors such as fibre optic strain sensors represent recent developments in this challenging area of structural health monitoring [6]. However, there are many practical difficulties in the applications of various smart sensors for structural health monitoring, such as the effect of measurement errors, flexibility, robustness and duration of these sensors' life spans.

GPS positioning has been proved to be a viable technology for monitoring the operational performance of various structures without interference to traffic flow or damage to the structure [5]. It has long-term stability, capacity in fully automated data collection and high three-dimensional (3D) positional accuracy. Operation is weather-independent. It gives absolute measurement and can be referred to any chosen datum; steady improvements in space segments have led to improved reliability.

Real-Time Kinematic (RTK) is a very appealing GPS positioning mode for the civil engineering community [7]. RTK GPS is a process where GPS signal corrections can be transmitted *via* a radio link or other means in real-time from a reference receiver or a network of reference stations at known location(s) to one or more remote rover receivers (in this instance those installed on the monitored bridge). These measured corrections can then be used to compensate for common errors of reference and rover receivers, such as satellite clock error, atmospheric delay and satellite orbital error, thus increasing 3D positioning accuracy to within a centimetre.

Using RTK GPS to monitor large suspension bridges and high-rising buildings/towers is a recent research focus [6, 8, 9, 10, 11]. Some researchers have introduced their preliminary results from using GPS to monitor the responses caused by wind loadings [12, 13]. Other work has shown the difficulties applying the technique to high rise buildings, as there is a necessity for a static reference station at approximately the same height as the measurement position on the structure to achieve sufficient accuracy [14].

The paper below outlines how RTK GPS has been used to monitor the Forth Road Bridge. The configuration and layout of a monitoring system is briefly introduced. Using data collected from different bridge sites, the wind-temperature-structure interaction is investigated. To generate more tractable results, a novel low-high pass filter has been developed. This overcomes stability problems of a low-pass filter when it is employed to detect aerostatic movements of the bridge and mitigate GPS multipath. Using this new filter, very slow movements with high amplitudes can be determined from displacement-time series. Through this real-life exercise, the paper demonstrates the advantages and feasibility of RTK GPS for addressing wind and temperature induced bridge dynamics. The potential application for the use of high precision GPS measurements of bridge displacements and the derived dynamic parameters under significant wind loading is explored.

2. Forth Road Bridge

The Forth Road Bridge spans the Firth of Forth linking Fife and the North of Scotland with Edinburgh and the south. It runs approximately north-south, and provides a vital strategic link to the rest of the UK for industry, and for tourism in the north and east of Scotland. It was opened to traffic on 4th September 1964. At that time it was the largest suspension bridge in Europe and the fourth largest in the world, with a main span of 1,005m and an overall length of 2.5km. During the last 40 years, traffic load on the bridge has risen steadily from 4 million vehicles p.a. in 1964 to over 24 million in 2004. It is one of the busiest estuarial crossings in the UK. Not only has the traffic volume increased, but in 1964 the heaviest commercial vehicles weighed 24 tonnes while the current limit is 44 tonnes. To cope with these steadily increasing traffic loads, an upgrading scheme of the two main towers was undertaken in 1994 and completed in June 1997 at a cost of £12.75m. Recent major road works have focused on repairing the damage caused by heavy traffic, and consultation is underway to discuss the need for a second crossing to alleviate the traffic loads [15]. This has become more pressing given recent findings of corrosion in the main cables.

The Forth Road Bridge is a stiffened deck structure designed to carry lateral loads in the structure, and differs fundamentally from other structures such as the Humber Bridge [17] that were designed with aerodynamic deck structures that would be subject to lower wind load.

3. Experimental method

In 2005 (from February 8th-10th) a major monitoring of the bridge using GPS technology led to 46.5 hours of continuous GPS code and carrier phase measurements, referred to subsequently as raw data, being collected from a total of 9 receiver sites [16]. Figure 1 shows schematically the instrumentation layout which consists of two reference stations (Ref1 and Ref2 as shown in Figure 2a set up near the bridge, two tower stations (A1 and A2) on each side of the southern supporting tower of the bridge, five monitoring sites on the bridge deck at quarter (B and E), three eighths (C), and half (D and F), of the full span. Two other receivers were available to be installed on two 40 tonne lorries to track their locations when the bridge was closed to carry out a known-load test that took place during a one hour period in the early morning of 10th February. The GPS data sampling rate was set to 10 Hz for A1, A2, B, C, D and F, and at E 4 Hz for GPS and 200 Hz for an Inertial Navigation System (INS) unit. AC power was supplied continuously at all bridge sites during the test. Specially designed clamps used to mount GPS antennas were fixed on the monitoring sites (bridge handrail). At site F (Figure 2b) on the west side of the bridge, a mini digital weather station was installed to measure temperature, wind speed and wind direction, and humidity all at a sampling rate of 30 seconds, as shown in Figure 2c. Wind speeds were averaged over this 30-second period by the weather station. Two 128Mb or 256Mb memory cards were used at each GPS site. This configuration could log raw GPS data for more than 10 hours continuously at a sampling rate of 10 Hz. When the card in the GPS receiver was full, data logging was interrupted for a few minutes and a new card was replaced to continue the data logging. During the total of 46.5 hours of field work around 5 GB of raw data were collected.

Additionally, during the (approximately) two-day period, the bridge was also closed for 10 minutes to allow an exceptional load (a 100 tonne lorry) to pass the bridge, and this event was recorded.

4. Data Processing and Analysis

To analyse global structural deformation under a meaningful engineering coordinate system, Cartesian coordinates in WGS84 (a GPS system) are converted to those in a Bridge Coordinate System (BCS) which uses the bridge main axis (approximately N-S for the Forth Bridge) as its *longitudinal* direction, the perpendicular direction to it in horizontal plane as its *lateral* direction and the *vertical* direction to complete a right-handed 3D coordinate system.

Structural deformation analysis requires extraction of structural dynamics through the design and implementation of an appropriate digital filter. In this work a Chebyshev type digital filter is used. Its input parameters include passband and stopband frequencies, passband ripple in decibels, filter order, and data sampling rate. An 8-order Chebyshev-type digital filter can be used to detect responding frequencies between 1.0 to 3.0 Hz. (For other frequency bands it is only necessary to adjust the input parameters to the filter.) Coordinate or acceleration data are filtered either to reduce the noise level or to split the measurements such that only the real signals to the allotted frequency band are output for further analysis. A narrow moving window FFT is then applied to the filtered measurement outputs to further extract both the vibration frequencies located within the designated frequency band and the corresponding vibration amplitude of the frequencies. [18]

In engineering design the concept of an "initial position" – perhaps under the action of self-weight alone – is used to give design comparisons. For example the effect of wind load on this initial position can be evaluated and displacements relative to this "at-rest" condition can be calculated. However, when working on a real structure under the actions of traffic, thermal and wind loadings, there is rarely a good measurement of an at-rest or initial position, and it is more relevant to report relative movements – which we do below. In the case of the work described below, it is possible to identify clearly the effects when, say, the lorry trials were undertaken, as the bridge was closed and vertical movement would be predominantly due to the live load action. However when the lateral load is considered, there is a cyclical motion about some mean position that will be determined by the prevailing wind conditions; the bridge was never stationary during the period of testing.

5. Results

5.1. *Lateral movement due to wind*

The polar coordinate plot of wind speed and direction in Figure 3 shows that the wind directions were mainly from the West to the North quadrant and the maximum wind speed reached 104km/hour (28.8m/s). The mean wind speed was about 60km/hour – near gale conditions - for much of the period. Table 1 lists the maximum and minimum 3D coordinates in the above-mentioned fixed BCS at the different bridge sites during the observation period. Tower top site A2 has been excluded as it is very close to A1. The maximum lateral movement of the deck at midspan reached 1.162m; the maximum vertical movement of the deck was 1.193m at midspan. Comparing the design (Table 2) and actual measured bridge movements, it can be seen that even under today's much higher density of traffic loading and a significant wind force that the peak-to-peak vertical deflexion of the bridge at midspan only reaches approximately one third (1.193m) of its designed maximum movement (3.292m) under live load. Even though the absolute displacements are different, the ratio of the vertical movements measured at quarter span B and midspan D match well with that of the designed values, which are 0.88m and 0.92m, respectively.

The maximum lateral movements measured by GPS differ from those predicted by the designers Not only is there a larger absolute displacement difference, but also there appears to be a larger ratio discrepancy for the lateral movements between measured and designed values at quarter span B and midspan D, which are 0.74m and 0.62m. However, it should be noted that results given in Table 1 represent a range of movement rather than a single movement, and conclusions about displaced shape are difficult.

Since the ongoing assessment of the bridge began, finite element modelling has developed significantly, and results from such models have shown remarkable agreement between the measured data with that predicted under the known load tests mentioned above [16]. Unfortunately the prediction of lateral displacements under wind loading is not available to the authors, nor is it in the public domain. A separate study is beyond the scope of this paper.

The maximum longitudinal movement during the observation period reached 0.253m on the top of the tower; and the maximum vertical deck movement was 1.193m

5.2. *Range of bridge movements*

General movement trends can be observed in Figure 4 - the scatter graphs of six bridge sites for horizontal (*Lat-Long*) and vertical sections (*Lat-Vert*). The shapes of the scatter of horizontal movements at the opposite midspan sites D and F during the 46.5 hour observation period are nearly the same but slight difference exists for symmetric quarter span sites B and E, demonstrating different local longitudinal movements. However, the similar shapes in the vertical section confirm that the differences in the horizontal plane between two quarter span were actually caused by the small differences in the local longitudinal movements.

Two representative sites F and A1 (Tower East) are chosen to investigate the correlations between 3D displacements and wind and temperature loadings during the entire observation period and the estimated cross correlation coefficients are listed in Table 3.

For bridge deck site F and tower top A1, the magnitudes of lateral movements are very different. The peak-to-peak movements for F and A1 are 1.142m and 0.060m, respectively. However they both show good levels of correlation with measured wind speed, which are 77% for site F and 66% for A1. Wind loadings also caused movements in the longitudinal and vertical directions for both sites, but due to the differences in their supporting substructures a more flexible site F has higher correlation with wind loading than tower top A1 in longitudinal direction. For example, when wind loading from the west was present the bridge deck displaced towards the east side of the bridge which caused the two supporting towers moved closer to the central span – but of course the west tower moved slightly more than the east tower. As a typical visual example, Figure 5 shows the apparent correlation between lateral movement and wind speed at site F for a period of 21 hours when wind speed reached the highest level during two days observation.

Further analysis was conducted to establish the relationship between lateral movement and wind speed for both midspan F and tower site A1 as shown Figures 6a and 6b. Leastsquares fitting can be used to obtain a best fit curve to establish a relationship between wind loading and response, assuming a quadratic form. For both sites, the lower wind speeds have higher correlation with the lateral movements as discussed by [2]. Figure 6c and 6d show the breakdown for the two distinct wind regimes experienced during the test period. For approximately the first 26 hours the wind was from a mean bearing of 263.2°(Standard Deviation 11.8°) and the lateral displacement against wind speed is shown in Figure 6c, while for the remainder of the period the mean bearing was 189.4° (standard deviation 20.4°) (Figure 6d).

5.3. *Temperature*

During the two days test the maximum temperature variation was about 5.5ºC. As with the wind loadings, temperature changes had different effects on the sites F and A1. When the air temperature increased the expansion of the bridge deck led to a reduction in height measured - i.e. the bridge sagged at midspan. It should also be noted that there was little solar heating effect at this time of year at these latitudes. The sagging of the deck with increased temperature is expressed as a negative cross correlation between vertical displacement and temperature for site F, which is shown in Figure 7 where all 46.5 hours measurements are used. However, due to vertical expansion of the tower when the temperature rose, the GPS receiver on top of the tower picked up the height increase due to this maximum 5.5 degrees temperature changes during two days observation, presented as a positive correlation of 13%. These correlations are also listed in Table 3. In the longitudinal direction, the bridge deck and supporting towers behaved in a different way in response to the temperature effect and showed a slightly different level of positive correlation with the changes of the temperature, demonstrating that when temperature rose the whole bridge deck dropped and the tower top was drawn towards midspan. The temperature has less effect on the lateral movement of the bridge deck but when temperature increased the unbalanced thermal effect on the slender supporting towers causing lateral bending.

The temperature effect on the bridge can be seen from Figures 8a and 8b in which the vertical movements at site F and the longitudinal movement at the tower site A1 are plotted against the temperature change for the time series of 46.5 hours. The resulting relationship – expected to be linear [2] – shows that the vertical movement at midspan is approximately six times the longitudinal movement at the tower.

5.4. *Interaction of Structural Components Due to Live Load*

The interaction between different components of the bridge is also analysed. This can be carried out very easily and precisely since the data sets collected from different sites are precisely synchronised to same GPS time frame. In practice, this characteristic can be employed to address the interaction between substructures caused by different loadings such as wind, temperature and traffic. Figure 9 shows the relationship between the relative longitudinal and vertical displacements at Tower East site and midspan Site F during the known loading lorry trials carried out between 1.00am and 2.00am on 10th February 2005. To match the displayed order of magnitude of the vertical displacement, the longitudinal displacement has been amplified by 4 times. From this figure it can be found when the two lorries were passing through the deck they forced the bridge to move downwards and at same time due to the increased cable force the two towers move towards midspan. A high correlation coefficient of 0.80 is obtained from these two data sets.

6. Conclusions

This paper demonstrates the feasibility of using RTK GPS to monitor both quasi-static and aerodynamic displacements induced by wind and temperature actions. Taking the Forth Road Bridge as an example, the authors introduce the instrumentation layout for field data collection. The comparison of designed and measured lateral and vertical peak-to-peak movements of the bridge at midspan under known (measured) conditions gives clear feedback to the design process.

The results show there are high cross correlations between lateral displacements and wind loading both at the high cable-supporting towers and the more flexible midspan sites; these two are taken as key representative sites. Temperature loading has medium level positive correlation with longitudinal movements at both sites and clear and expected negative correlation with the vertical displacements.

The authors use a low-high pass filter to extract very low structural aerostatic movements caused by wind loading. It has been successfully used to ascertain both the 3D main vibration frequencies and their amplitudes. The characteristics of the 3D movements at different bridge sites are further analysed.

This work described provides a basis for obtaining excellent comparisons of design and as-built conditions, and enables engineers to assess the outcome of designs in which stiffness is a key characteristic.

Acknowledgement

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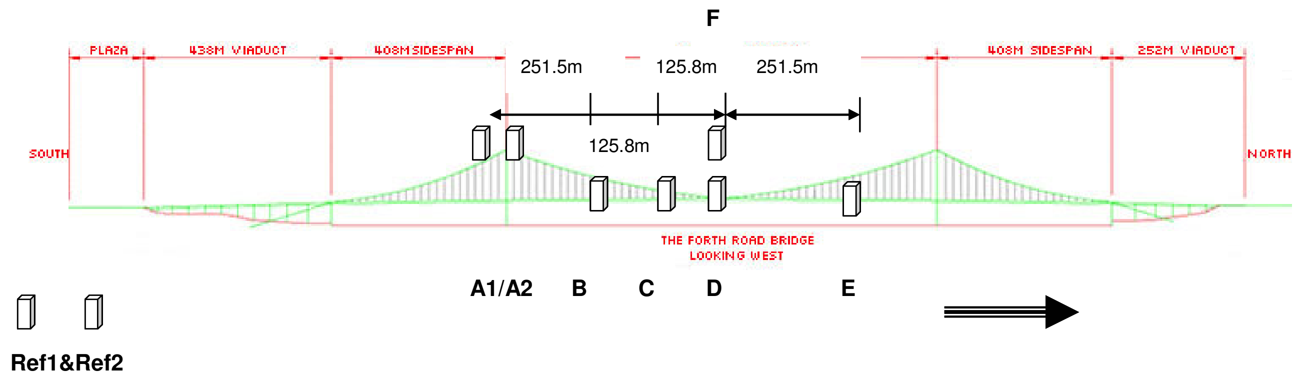


Figure 1 Instrumentation layout on the Forth Road Bridge.

Figure 2a

Figure 2b

Figure 2c

Figure 2a Two reference stations installed close to the Forth Road Bridge

Figure 2b Installation of a GPS antenna on the bridge handrail by a clamp (Site F)

Figure 2c Weather station set up on site F.

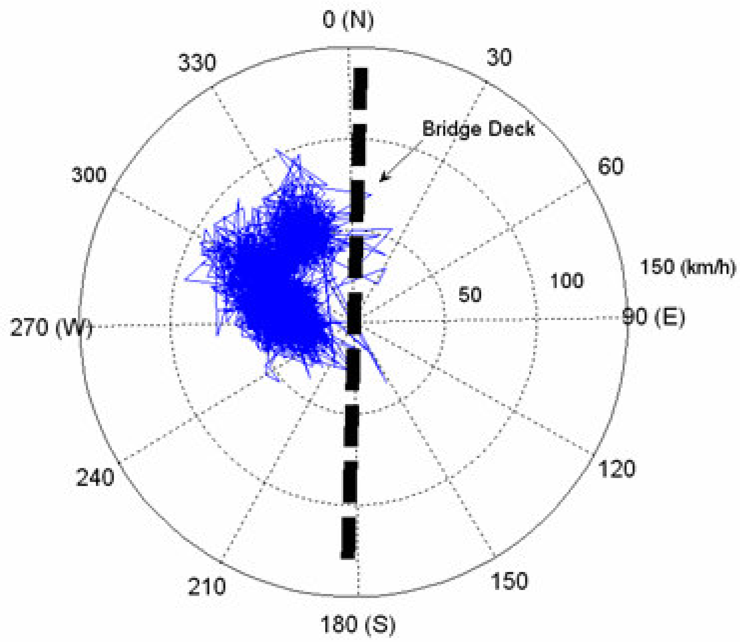
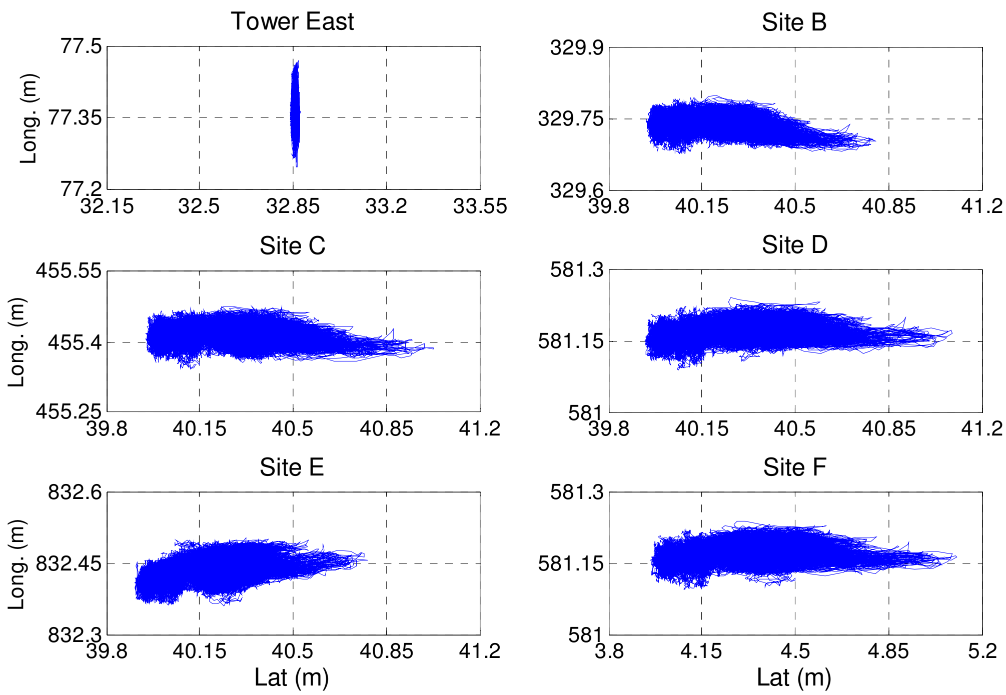


Figure 3 Polar plot of wind speed and direction.



Tower East
Site B
Site C
Site D
Site E
Site F
Long. (m)
Lat (m)
77.5
77.35
77.2
32.15
32.5
32.85
33.2
33.55
329.9
329.75
329.6
39.8
40.15
40.5
40.85
41.2
455.55
455.4
455.25
581.3
581.15
581
832.6
832.45
832.3
3.8
4.15
4.5
4.85
5.2

Figure 4a

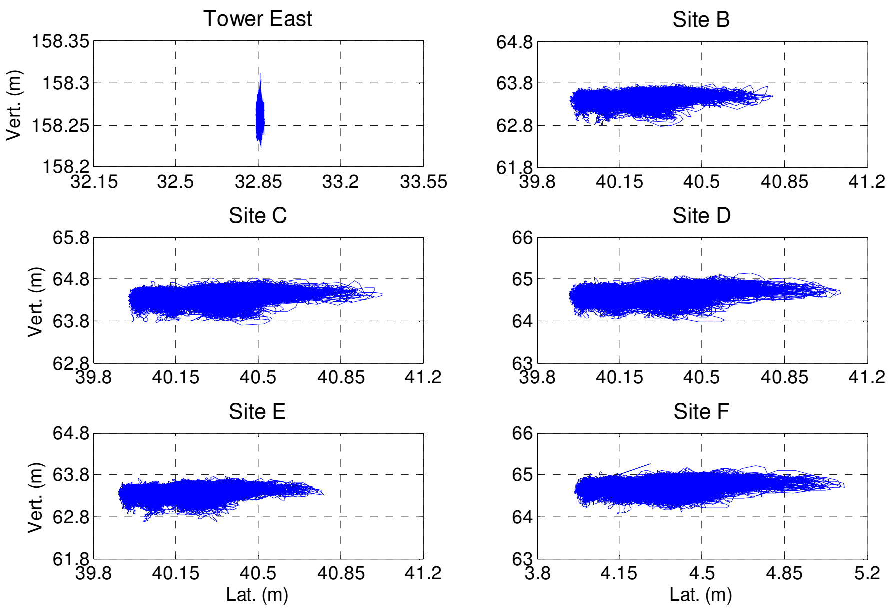


Figure 4b

Figure 4 46.5 hours plane (*Lat-Long*) and vertical (*Lat-Vert*) scatter graphs of six sites on the Bridge.

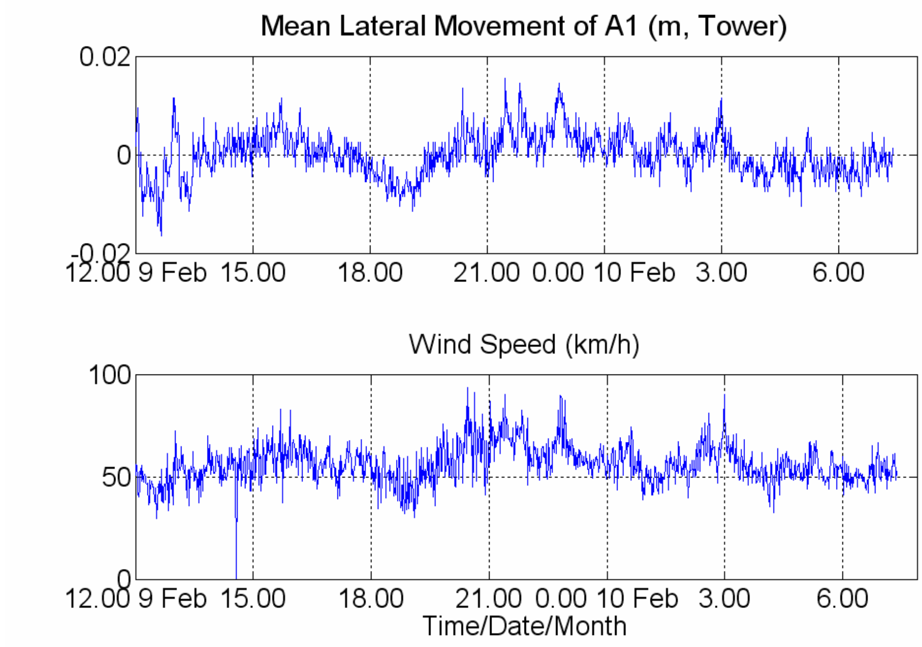
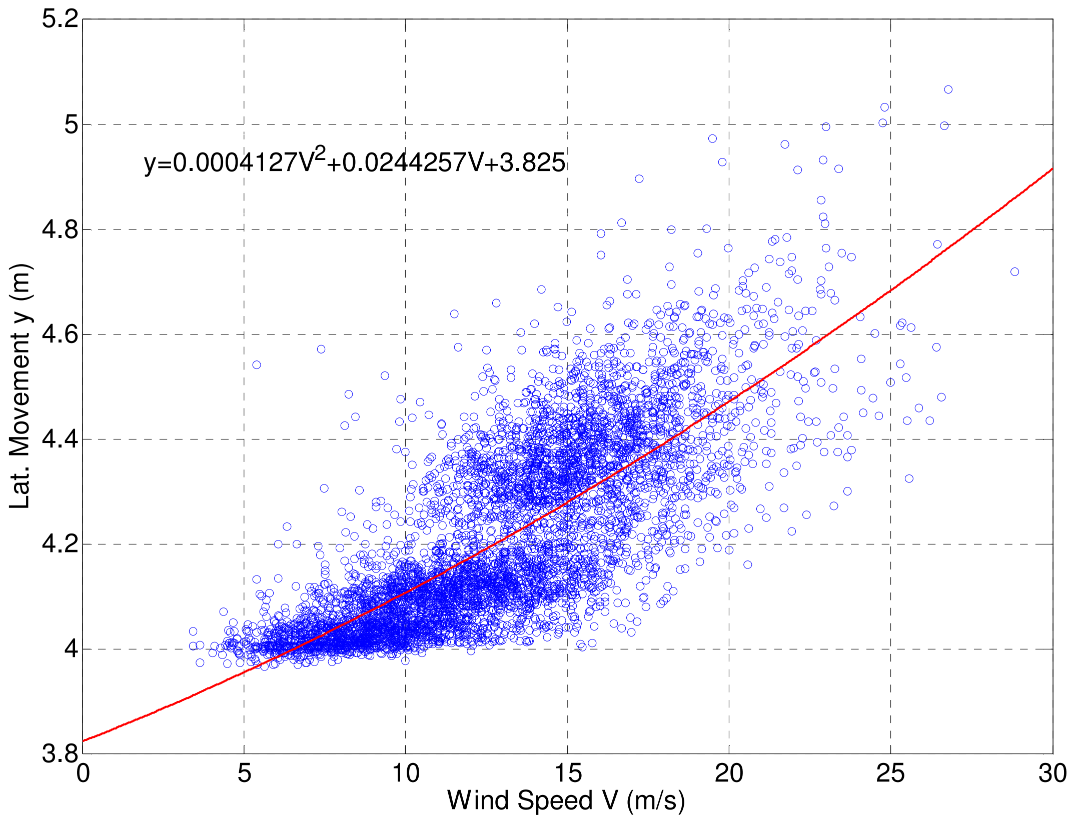
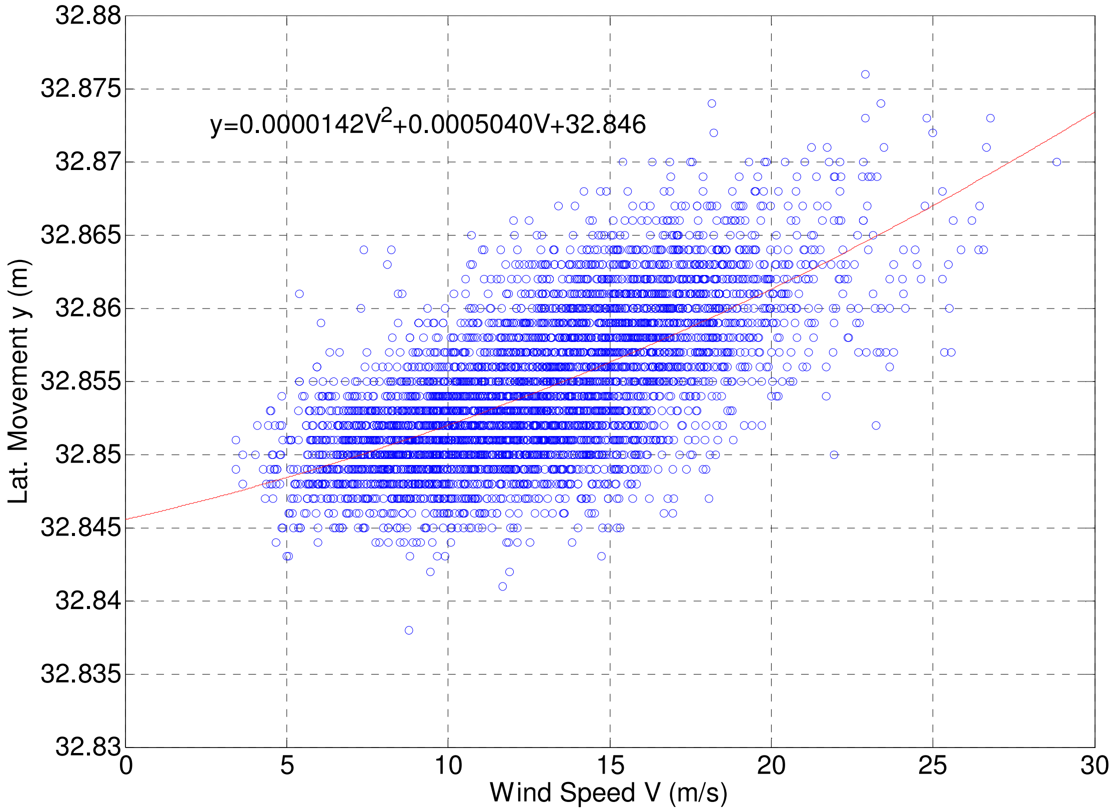


Figure 5 Time series of mean lateral movement and wind speed



y=0.0004127V^2+0.0244257V+3.825
Lat. Movement y (m)
Wind Speed V (m/s)
5.2
5
4.8
4.6
4.4
4.2
4
3.8
0
5
10
15
20
25
30

Figure 6a



y=0.0000142V^2+0.0005040V+32.846
Lat. Movement y (m)
Wind Speed V (m/s)
32.88
32.875
32.87
32.865
32.86
32.855
32.85
32.845
32.84
32.835
32.83
0
5
10
15
20
25
30

Figure 6b

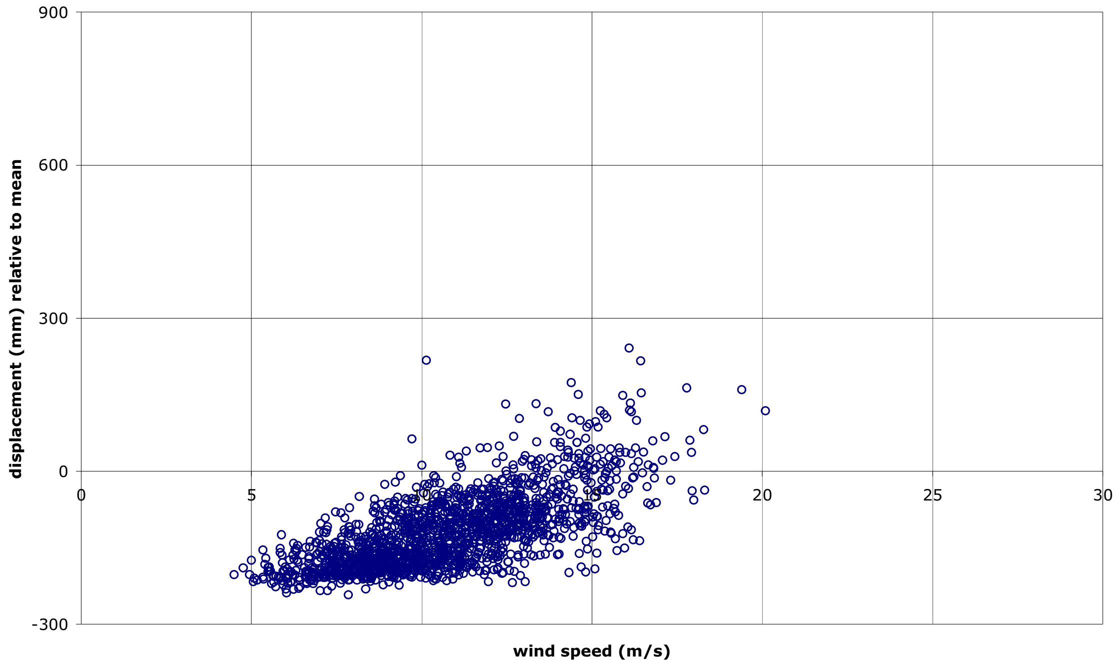

displacement (mm) relative to mean
900
600
300
0
-300
0
5
10
15
20
25
30
wind speed (m/s)

Figure 6c

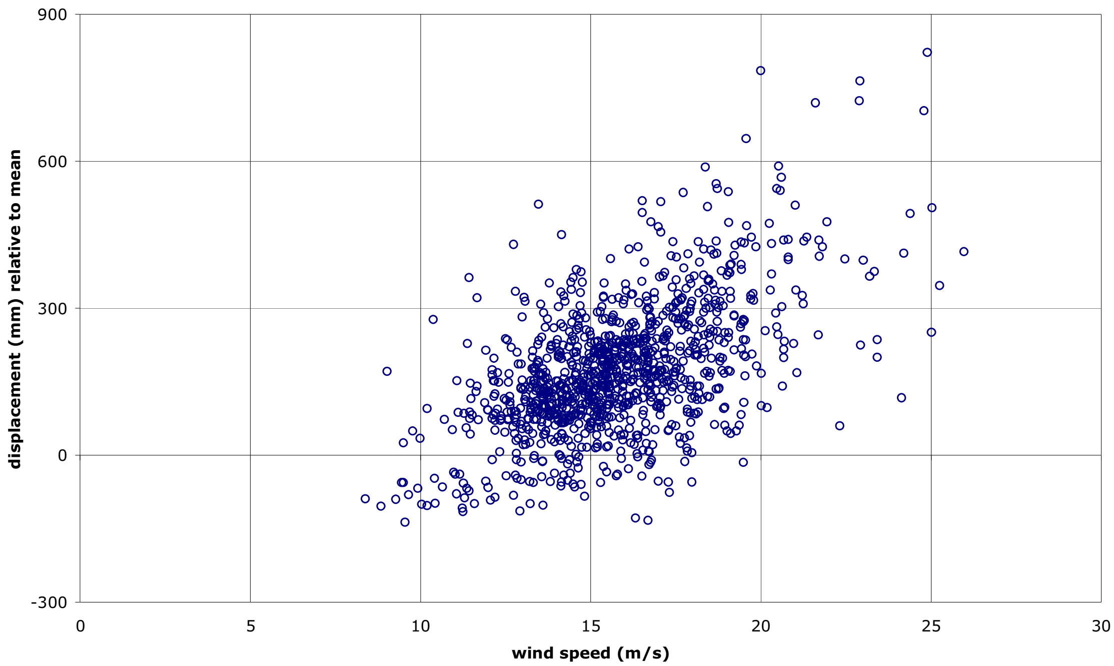


Figure 6d

Figure 6

(a) Lateral movement-wind interaction at site F

(b) Lateral movement-wind interaction at site A1

(c) Lateral movement – wind speed at site F for first 26 hours

(d) Lateral movement –wind speed at site F for final period (19hrs approx.)

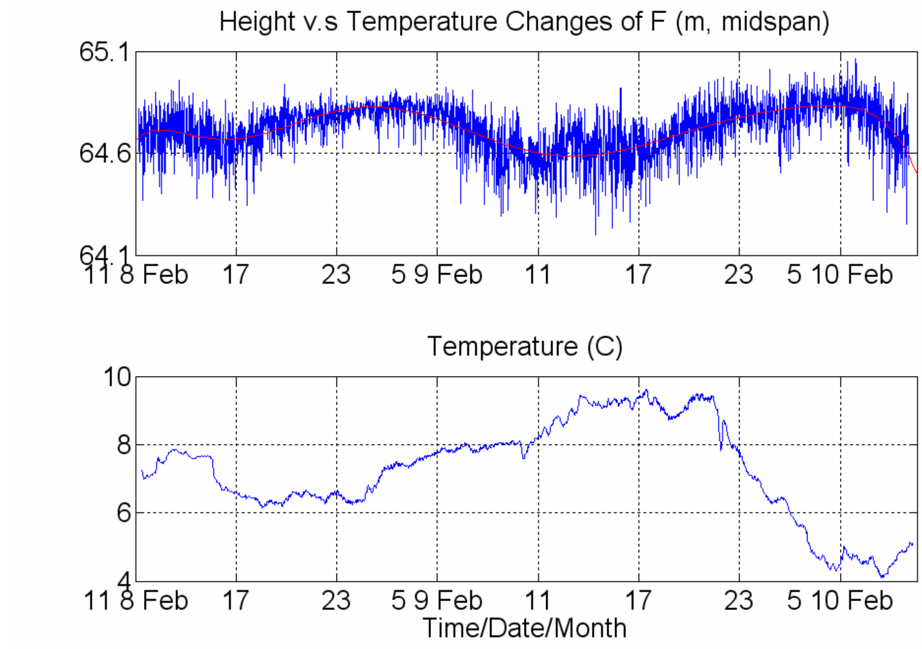
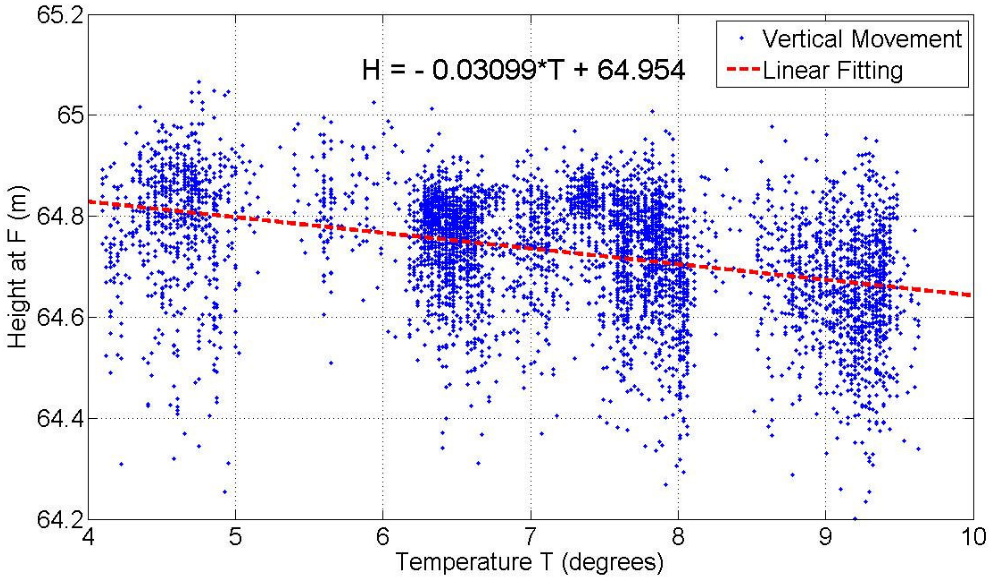


Figure 7 Time series of height and temperature changes



65.2
65
64.8
64.6
64.4
64.2
Height at F (m)
4
5
6
7
8
9
10
Temperature T (degrees)
H = - 0.03099*T + 64.954
Vertical Movement
Linear Fitting

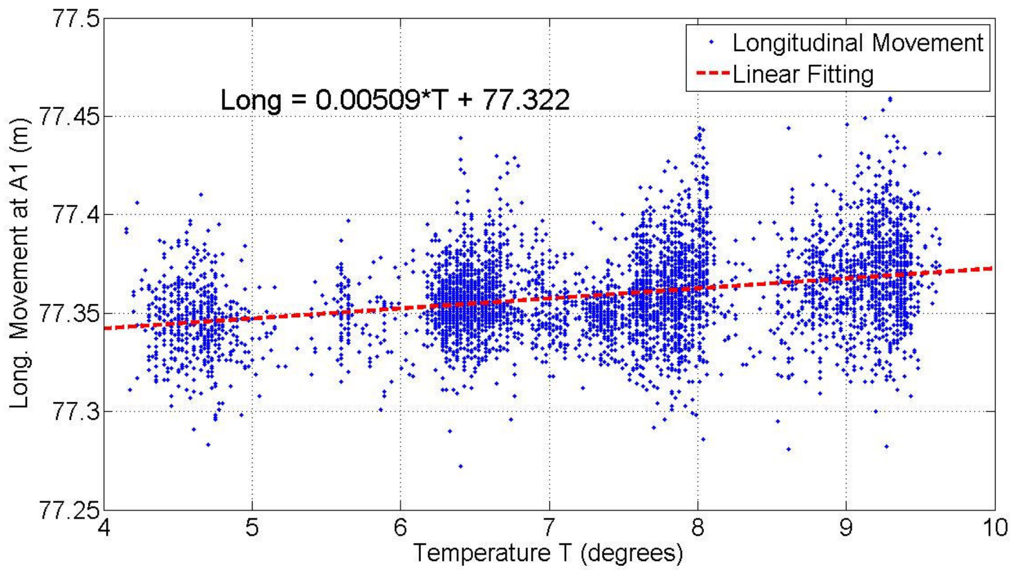


Figure 8 (a) Negative correlation between vertical movement and temperature change at site F (b) Positive correlation between longitudinal movement and temperature change at site A1.

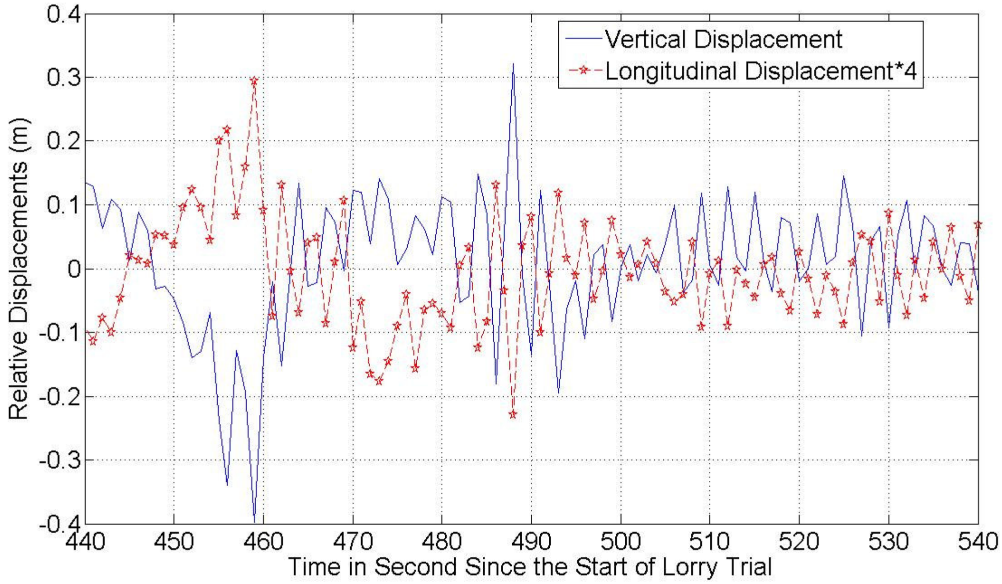


Figure 9 Correlation between Longitudinal and Vertical Displacements at Tower East and Midspan West.