Structural health monitoring of the Tamar Suspension Bridge

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Abstract

This paper presents experiences and lessons from the structural health monitoring practice on the Tamar Bridge in Plymouth, UK, a 335m span suspension bridge opened in 1961. After 40 years of operations the bridge was strengthened and widened in 2001 to meet a European Union requirement to carry heavy goods vehicles up to 40 tonnes weight, a process in which additional stay cables and cantilever decks were added and the composite deck was replaced with a lightweight orthotropic steel deck. At that time a structural monitoring system comprising wind, temperature, cable tension and deck level sensors was installed to monitor the bridge behaviour during and after the upgrading. In 2006 and 2009 respectively, a dynamic response monitoring system with real time modal parameter identification and a three-dimensional total positioning system were added to provide a more complete picture of the bridge behavior, and in 2006 a one day ambient vibration survey of the bridge was carried out to characterize low frequency vibration modes of the suspended structure. Practical aspects of the instrumentation and data processing & management are discussed and some key response observations are presented. The bridge is a surprisingly complex structure with a number of inter-linked load-response mechanisms evident, all of which have to be characterized as part of a long term structural health monitoring exercise. Structural temperature leading to thermal expansion of the deck, main cables and additional stays is a major factor on global deformation, while vehicle loading and wind are apparently secondary factors. Dynamic response levels and modal parameters show apparently complex relationships among themselves and with the quasi-static load and response. As well as the challenges of fusing and managing data from three distinct but parallel monitoring systems, there is a significant challenge in interpreting the load and response data firstly to diagnose the normal service behavior and secondly to identify performance anomalies.

Keywords: Structural Health Monitoring, Suspension Bridges, Environmental

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1. Introduction

Suspension bridge monitoring programs have historically been implemented for the purpose of understanding and eventually calibrating models of the load-structure-response chain. One of the earliest documented systematic bridge monitoring exercises, by Carder (1937), was conducted on the Golden Gate and Bay Bridges in San Francisco in an elaborate program of measuring periods of the various components during their construction to learn about the dynamic behaviour and possible consequences of an earthquake.

Interest and capability in suspension bridge monitoring picked up in the 1980s with examples of Deer Isle, Humber and Tsing Ma Bridges. (Bampton et al., 1986; Brownjohn et al., 1994; Chen et al., 2004). In some countries (Miyata et al., 2002; Authority, 1987; Koh et al., 2009), some form of monitoring of long span bridges is mandatory, and it is becoming routine for new long span bridges (LSBs) in China and Hong Kong to include comprehensive monitoring, with the most complex example soon to be operational in Hong Kong's Stonecutters Bridge (Structural Vibration Solutions, 2009).

When properly managed, such exercises can provide vital information about LSB performance. Capability to gather and interpret performance data is particularly important for these large structures, since despite advanced computational capabilities there are still surprises in real life operational conditions. Bridge engineers still get caught out due to lack of understanding of loading mechanisms, with famous examples provided by Tacoma Narrows Bridge (University of Washington, 1954) and London Millennium Bridge (Fitzpatrick and Smith, 2001), and more recently the Great Belt bridge, which exhibited excessive response due to vortex shedding (Larsen et al., 2000).

The most significant environmental loads on a long span suspension bridge are due to wind, hence suspension bridge structural health monitoring (SHM) programs have typically focused on wind effects. For example the Humber Bridge monitoring program (Brownjohn et al., 1994) was designed to calibrate wind load/performance simulation software, while a major concern in Asia has been performance during typhoons (Miyata et al., 2002; Xu et al., 2007).

Thermal effects are also a major concern for LSBs where variable solar heating with convection and conduction result in a complex expansion pattern that has to be accommodated by complex bearing arrangements, although relatively little attention is paid to thermal effects in the literature (Ni et al., 2005). Deformation monitoring for suspension bridges is thus a major exercise, for which a range of technology is used, with increasing use of GPS technology (Wong et al., 2001), along with some more conventional technology such as hydrostatic levelling devices and optic-based systems (Brownjohn and Meng, 2008).

Accelerometers are another conventional and widely used technology for suspension bridge SHM, used not only for measuring response levels but also for system identification. While there have been very many short-term vibration testing exercises for high resolution mapping of mode shapes on suspension bridges (Brownjohn et al., 2010), relatively few studies have used permanent dense arrays (Pakzad and Fenves, 2009) and real time modal parameter estimation remains a rarity. Such technology, as expected to operate on Stonecutters Bridge provides capability for real time tracking of modal parameters for the purpose of online condition assessment and indication of structural change (Siringoringo and Fujino, 2006).

Suspension bridge SHM research still presents significant challenges in instrumentation, data management and interpretation. The dominant application is confirmation of safety and serviceability of these lifeline structures and bridge operators take a great interest in these systems. With such an opportunity, SHM system designers need to pay attention to system efficiency, reliability and most importantly the way information is presented. This means that an appropriate level and type of instrumentation should be used, with downstream signal interpretation and data mining capabilities aimed at presenting the right level of information at the right time.

The Tamar bridge SHM system has progressively developed along these lines, combining different measurement technologies with sophisticated modal analysis and data mining techniques (Farrar and Worden, 2007; Cross et al., 2010). As the system has grown, so has understanding of the bridge response 'mechanisms'. These have had to be identified and understood as a prerequisite for reliable diagnosis of unusual performance. This then is the focus of this paper: a description of the technology and presentation, with some interpretation of performance of this rather unusual bridge.

2. Tamar Suspension Bridge

2.1. History/specification

The Tamar Bridge (Figure 1) forms a vital transport link over the River Tamar carrying the A38 trunk road from Saltash in Cornwall to the city of Plymouth in Devon. The bridge is owned, operated and maintained by the two local authorities, and has relied solely on toll income to cover all capital and operating costs.

The original bridge opened in 1961 and was designed by Mott Hay and Anderson as a conventional suspension bridge with symmetrical geometry, having a main span of 335 metres and side spans of 114 metres, and with anchorage and approach spans the overall length is 642 metres. Unusually for a suspension bridge of this era, the Towers were constructed from reinforced concrete, and have a height of 73 metres with the deck suspended at half this height. The Towers sit on caisson foundations founded on rock.

Main suspension cables are 350mm in diameter and each consists of 31 locked coil wire ropes, and carry vertical locked coil hangers at 9.1 metre centres.

The main cables are splayed at anchorages and anchored some 17 metres into rock. The stiffening truss is 5.5 metres deep and composed of welded hollow steel-boxes. The original 3-lane deck, spanning between cross trusses,

Figure 1: Tamar suspension bridge

was of composite construction with a 150mm deep reinforced concrete slab on five longitudinal universal beams and surfaced with 40mm of hand-laid mastic asphalt.

2.2. Upgrade

When opened in 1961 Tamar was, for a short time, the longest suspension bridge in the UK and was also the first to be built since World War 2. It was initially carrying approximately 4000 vehicles a day with a maximum gross weight of 24 tons, but in the late 1990's, after nearly four decades of use, it was found that the Tamar Bridge would not be able to meet a new European Union Directive that bridges should be capable of carrying lorries up to 40 tonnes weight. Since restricting use by such vehicles would damage the local economy, the bridge needed to be strengthened or replaced.

The appointed consultant (Acer, now Hyder) proposed replacement of the main deck with a lightweight orthotropic steel deck, and having investigated a range of traffic diversion options, proposed construction of temporary relief lanes cantilevered off the bridge truss, to act as a supplementary diversion route while the main deck was being replaced. As the design developed, it soon became apparent that the permanent addition of cantilever lanes offered a cost-effective improvement in terms of both capacity and safety, as well as satisfying the temporary diversion requirement. More background to the exercise is provided by Fish and Gill (1997) and the implemtation proceeded in three phases.

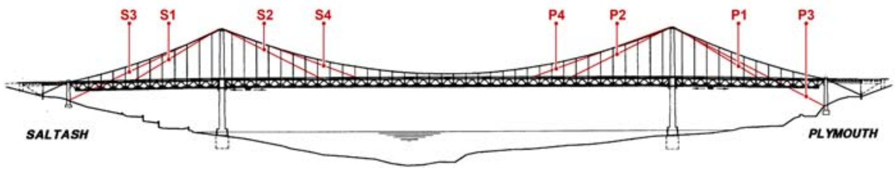


Figure 2: Arrangement of additional stays

Figure 3: Deck truss and additional longitudinal girder

2.2.1. Phase 1 Strengthening structure in advance of the addition of cantilever lanes.

Eighteen new diameter locked-coil cables with diameters of 102mm or 110mm were installed and stressed to supplement the original suspension system, primarily to help carry the additional dead load of the new cantilever lanes and associated temporary works (Figure 2). They also reduced the extent of truss strengthening required, and restored some 400mm of the original hogged profile of the main deck, which had been lost due to main cable creep over the previous 40 years.

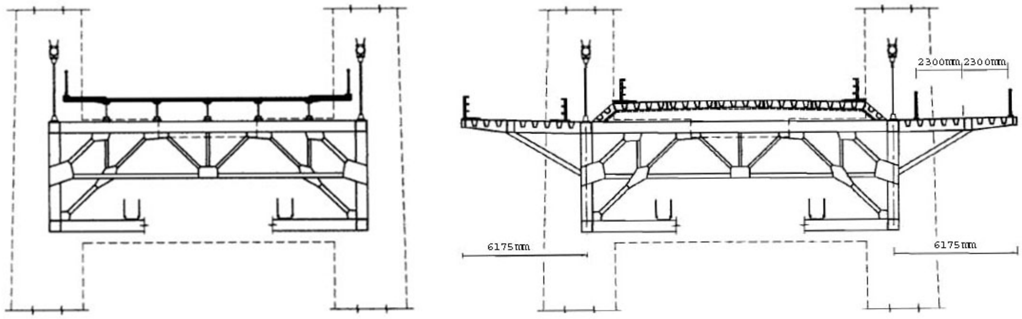
The truss was strengthened by the installation of supplementary inverted U-shaped parallel elements fitted below the bottom chord and by welding additional steel plates at key locations (Figure 3). Two of the additional cables run in these inverted U-girders.

2.2.2. Phase 2 Erection of the cantilever decks.

Pairs of prefabricated orthotropic panels, each typically 15m long and 3m wide were welded longitudinally to form the 6m wide cantilever sections, also surfaced with hand-laid mastic asphalt and when in place, traffic was diverted onto the new lanes (Figures 4 and 5).

2.2.3. Phase 3 Replacement of the main deck.

A new orthotropic steel deck was installed as shown in Figures 4 and 6. This was done in two stages, separating the deck into two longitudinal halves,



(a) before　　　　　　(b) after

Figure 4: Deck systems before and after strengthening and widening

Figure 5: Installing cantilever

Figure 6: Orthotropic main deck

to allow one lane of traffic to run on the main deck at peak times, which with the new cantilevers provided up to three vehicular lanes. Replacement panels, also fabricated in two halves were placed using travelling portal gantries.

Approximately 2,800 tonnes of structural steel was added together with 125 tonnes of cables, but when offset by the removal of the old main deck, the final weight of the suspended structure rose by just 25 tonnes to 7,925 tonnes. The deck replacement process was completed by December 2001 and the bridge now carries about 50,000 vehicles per day.

2.3. Maintenance and operation concerns

2.3.1. Present day performance

The main concerns for bridge operations are safety, not just in the bridge but in the adjoining Saltash tunnel. CCTV cameras and image tracking software are used to avoid and manage dangerous traffic situations and wind data are used to determine when the bridge should be closed to high sided vehicles.

The additional cables restored 400mm of the original hogged longitudinal profile of the main deck that had been lost due to main cable creep over the previous 40 years. As part of the ongoing structural assessment program, deck profile is checked at frequent intervals by surveying.

There are two aspects of structural performance resulting from the upgrade that have interested bridge management. The first is the behaviour of the bearings and the global deformation of the bridge and the second is the dynamic behaviour of the additional stay cables.

2.3.2. Bridge deformation and bearings

In the original configuration, bearings at the Saltash Tower comprising vertical swing links and lateral thrust bearings allowed for longitudinal movement and rotation about a vertical axis to allow deck sway, together with an expansion joint in the roadway. At the Plymouth Tower the arrangement was the same but with a link connecting the trusses either side of the Tower. Figure 7 shows the original arrangement of thrust bearings.

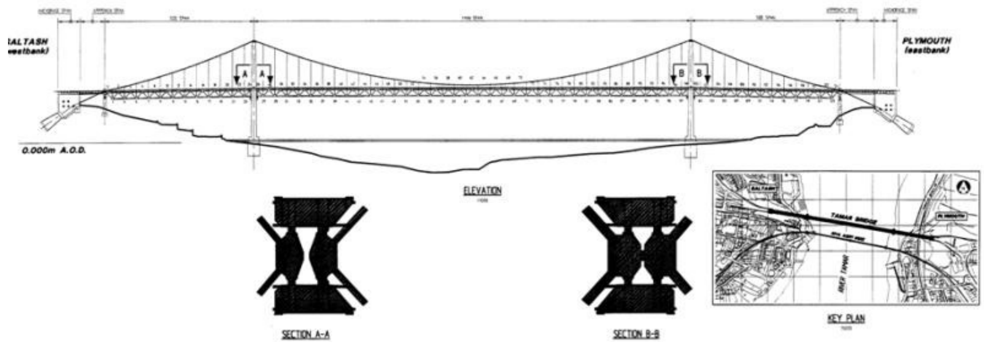


Figure 7: Bearing arrangement before upgrade

(a) Continued joint

(b) Bearing

Figure 8: Bearing arrangement after upgrade

During the upgrade, the link at the Plymouth Tower was severed, and the axial load path to provide longitudinal restraint on the main span is now via the cantilever deck sections which are continuous either side of the Plymouth Tower and incorporate movement joints at the Saltash Tower. While the outrigger truss provides vertical support for the cantilever (except around the Towers), longitudinal forces are transferred between deck and cantilever via plate shear boxes. Figure 8 shows the arrangement of bearings at the Plymouth Tower after the upgrade; the left view shows the swing links that restrain vertical and torsional motion of the deck and the unsupported continuous cantilever around the Tower, while the right view shows the thrust bearing with the severed longitudinal link. There is some uncertainty about how these new arrangements are working, and the rearrangement has resulted in a net movement of the deck away from Plymouth.

(a) (b)

Figure 9: Shear boxes shown behind the hanger sockets (left) and additional stay connection at deck level with strain-gauges (right)

2.3.3. Additional stay cables

On a number of occasions during and for a few years after the upgrading, and during certain wind conditions, but always accompanied by rain, certain of the additional stay cables exhibited vertical plane oscillations of up to 100mm amplitude. This phenomenon has been observed on a number of suspended span bridges worldwide and is currently a challenging research topic. The implications for the bridge are public concern and durability of the cable sockets and it was desirable that vibrations be eliminated.

P1 north and S2 south stay cables as shown in Figure 2 exhibited large amplitude vibrations during construction as tension varied, and after construction the longer cables (3 and 4) on occasion exhibited vibrations of 40mm amplitude.

Temporary dampers have now been installed on all 16 additional stays, and comprise full 300-gallon plastic water butt each containing a metal disc attached to the cable by tie rod and clamp. Figure 11 shows P3S (S denotes south side of bridge) undergoing dynamic testing, with a clear demonstraton of the effectiveness of the damper in controlling the vertical motion.

3. Static monitoring system

During the upgrade, environmental and structural monitoring equipment was installed, which supplemented with live loading information obtained from a weigh-in-motion system (Richard may answer it), allowed the project team to examine the behaviour of the structure under changing environmental and loading conditions.

3.1. Instruments

The Structural Monitoring System (SMS) installed by Fugro Structural Monitoring was used to monitor cable loads, structure and environment temperatures and wind speed and profile. The SMS has provided engineering information on the performance and condition of the bridge during and after the strengthening and widening.

In particular it was used to track deck profile and cable loads during the strengthening works. The sensors used in the SMS include (see Figure 10):

- Anemometers to measure wind speed and deflection
- Fluid pressure-based level sensing system measure deck vertical displacement
- Temperature sensors for main cable, deck steelwork and air
- Extensometers and resistance strain gauges to measure loads in additional cables
- Electronic distance measurement between Tower tops

The sampling frequency of the DAQ system was originally 1Hz but has been changed to 0.1Hz to save hard disk spaces since 30th April 2009. This modification was done after the event of the data loss between JAN to March 2009 caused by a shortage of computer storage space.

3.1.1. Temperature

Temperature sensors for steel and cable monitoring are Platinum Resistance Thermometers (PRTs) on stainless steel shim glued in place. Temperature sensors for air monitoring consist of temperature probes with radiation shields.

3.1.2. Strain gauges

Type 1,2,3 and 4 inclined cables are measured by resistive strain gauges attached to main tensioning bolts at deck anchor points, Figure 9-(b). Fixing is by epoxy (protected by foil-backed putty) or micro-welding (covered by butyl rubber and neoprene). Gauges are arranged in pairs 180 degrees apart around the bolt, each pair comprising an axial element and an element to measure hoop strain. The four gauges are connected to a full Wheatstone Bridge, with the hoop gauges providing the temperature compensation.

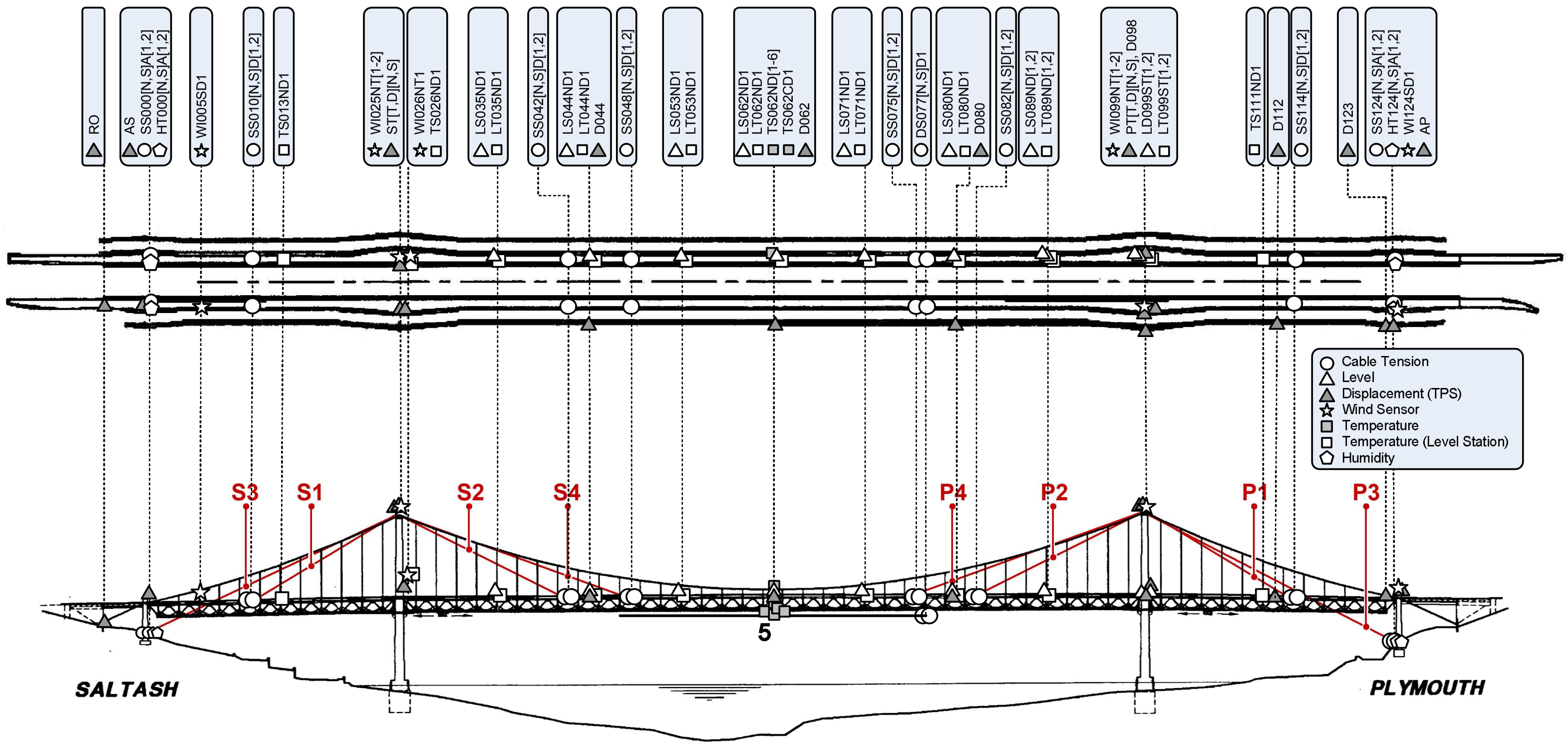


Figure 10: Sensors in Tamar Bridge

3.1.3. Wind sensors

Wind speed is measured mechanically at the top of the Saltash Tower (where direction is also measured), at the deck level of Saltash Tower and Saltash Approach.

3.1.4. Tower displacement measurements:

Tower separation is measured by an electronic distance measuring device (EDM) attached to the upper portal of the Plymouth Tower. This uses a laser reflected off a vertical array of mini-prisms on the wall of the Saltash Tower upper portal; the array size allows for in plane movement of the Towers.

3.1.5. Level sensing system:

This comprises a fluid manometer system with fluid-filled pipes along the main span. Heights are measured at level sensing stations (LSS) 1/8 span centres using pressure measurements on the fluid head. The system is based on a similar system installed by Fugro on bridges in the Lantau Fixed Crossing. The height measurements were specified to be accurate to +/5mm (Wong et al., 2001) and are updated very 10 seconds (but sampled at 1Hz). Fluid temperature sensors are also installed at each LSS.

3.2. Acquisition

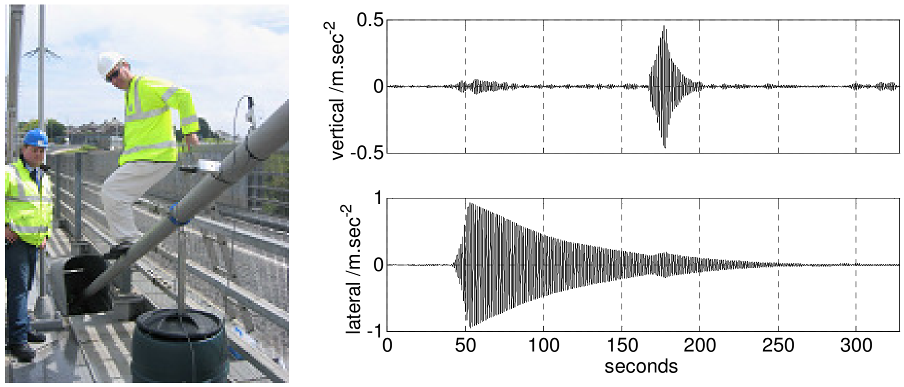
Visual displays from the SMS are provided via a mimic in the control room PC. The system stores time series as single comma-delimited ASCII data files for each day, and a smaller text file contains hourly values of mean, minimum, maximum and standard deviation. These files are available on a PC in the bridge control room and are accessed and downloaded using FTP.

The system was overhauled and upgraded in the summer of 2007, so that from September 2007 a complete set of 70 channels of response data has been available.

4. AVT

Facilities and staff from the Vibration Engineering Section (http://vibration.shef.ac.uk) were used an the ambient vibration survey on 28^{th} April 2006, using testing facilities as follows:

- A set of 16 QA700 and QA750 Quartz-flex servo accelerometers
- Data Physics Mobilyzer multi-channel data acquisition/spectrum analyzer system
- MODAL (Brownjohn et al., 2001) and ARTeMIS operational modal analysis (OMA) software



(a) Free vibration test

(b) Free decaying curves

Figure 11: Damper installed at cable 3S under test, showing water-butt damper

Overview of the AVT test is shown in Figure 12. To cover all the measurement points, eight roving tests were carried out on the measurement grid shown in Figure 13. Due to limited access across the bridge, all the measurement points were on the south side except for two locations on the north span where pairs of accelerometers sensing in vertical and lateral directions were located for the duration of the test as biaxial references.

The NExT/ERA (James III et al. 1995,) and eFDD (Brincker et al., 2010) methods were used for OMA and results using NExT/ERA are shown in Table 1. A complete description of the modal test is provided by Brownjohn et al. (2007). Assumptions of symmetry were used for estimating the mode shapes, while the frequency and damping values are no more than estimates, since the long term monitoring has shown significant variations in these values. Separate testing exercises have also been conducted on the stay cables, as illustrated in Figure 11. These tests show that for each cable, vibration modes exist in pairs of very close frequencies at dozens of harmonics of fundamental frequencies that are in the region of 1Hz. The cable parameters have been used to corroborate cable tension values from the Fugro system and to confirm effectiveness of the dampers.

5. Dynamic monitoring system (University of Sheffield)

An additional set of sensors was installed by University of Sheffield in 2006 to monitor dynamic behavior of the bridge deck and selected cables. In order to confirm satisfactory performance of the damper solusion, four stay cables, P4N, P4S, P1N and S2S were instrumented with a pair of accelerometers, one oriented horizontally and one in the vertical plane of each cable as shown in Figure 14.

(a) (b) (c)

Figure 12: AVT in progress, showing location of accelerometers next to shear box and data acquisition centre close to Tower.

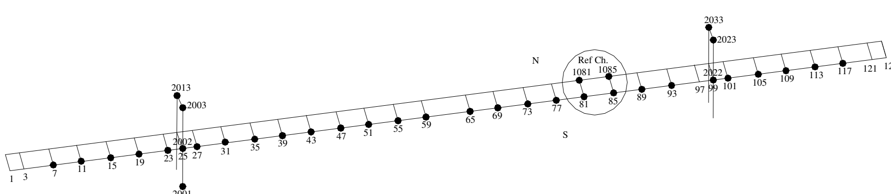
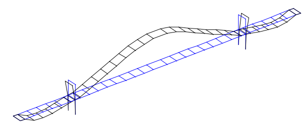
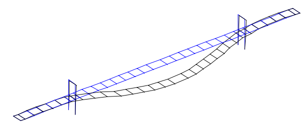
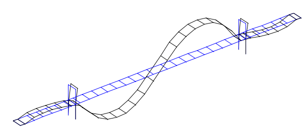
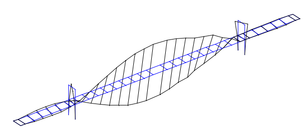


Figure 13: AVT measurement grid highlighting reference positions

Table 1: Modal parameters from AVT

Mode	Side span	Frequency (Hz)	Damping (%)	Mode Shape
VS1	strong	0.39	1.7	
LS1	neg	0.46	3.5	
VA1	weak	0.59	1.8	
LS2	N.A.[a]	*0.69*[b]	N.A.	
TS1	weak	0.73	1.1	

[a]NExT/ERA method missed this mode
[b]Estimated frequency from FEM model. Corresponding mode shape is given on the right.

Figure 14: Biaxial accelerometer on stay cable.

As well as the eight cable accelerometers, three accelerometers were installed at the deck section corresponding to hanger 69, the fourth hanger location from the midspan in the Plymouth direction. Because of the interest in the deck deformation, a set of three extensometers was installed at the Saltash Tower, where the continuity of the deck is broken to allow for expansion due to thermal and other causes.

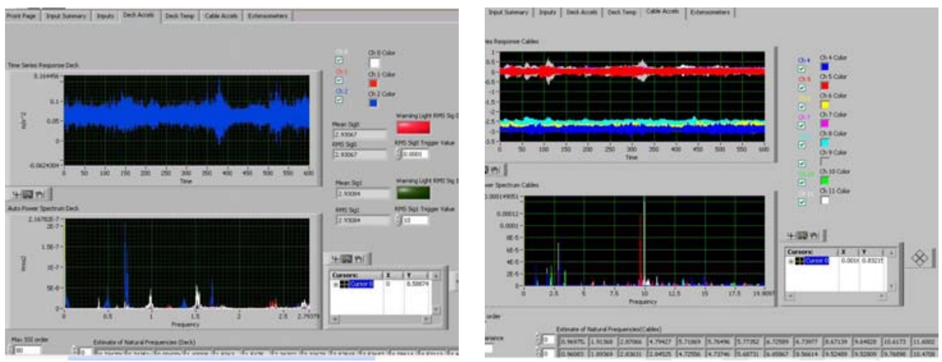
In the original form operating from 2006 to 2009, the Sheffield system recorded 64Hz-sampled time series in files at 10-minute intervals. The acquisition was managed using a virtual instrument (VI) written in LabVIEW. Figure 15 shows displays of time and frequency domain response for the deck and cable accelerometers respectively.

The VI carried out signal processing of the data, storing raw data and results from processing (discussed in the next section). It also managed emails of condensed summarised response parameters for a day of 10-minute time series. These summaries contain values of mean and RMS for all channels.

A significant feature of the VI was the incorporation of automated system identification procedures using the covariance-driven stochastic subspace identification (SSI) (Overschee and Moor, 1996 and Peeters and Roeck, 1999). For this project, the procedure was coded in MATLAB called from within the VI which saves and emails a separate daily log file of modal parameters (Brownjohn and Carden, 2008).

6. Observed performance up to 2009

Until 2009 data were managed through the daily log files of frequency, damping and RMS values generated by the Sheffield system as well as through the



(a) (b)

Figure 15: LabVIEW virtual instrument pages showing signals from deck (left) and cable (right) accelerometers in time and frequency domain.

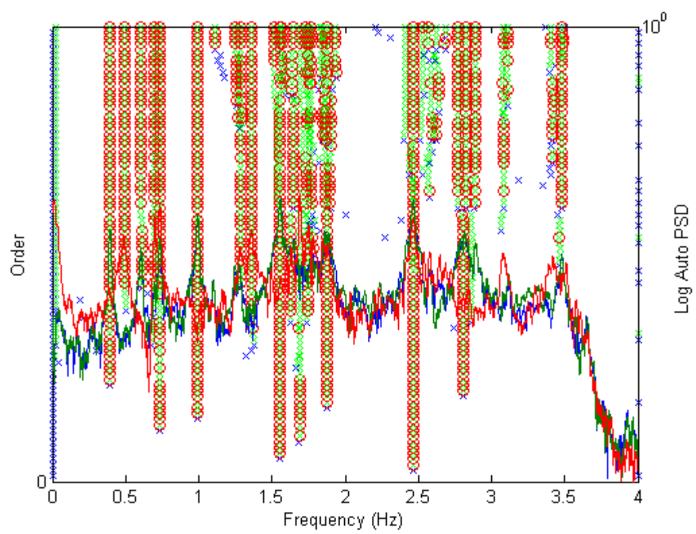


Figure 16: Acceleration auto spectrum and SSI stability plot from deck signals

hourly statistics from the SMS representing the wind and environmental parameters with the bridge static response in terms of the deck level and cable tensions.

Fusion of these data sets represented a challenge for a number of reasons not least because of the different sampling rates. For example, until July 2007 many of the SMS sensors were unserviceable, likewise the extensometers ceased to function correctly at about the same time. While the SMS logged more or less continuously throughout this period, there were several gaps in data from the Sheffield system. Hence the data were processed for specific months between March 2007 and December 2008 corresponding to the most complete data.

Samples of these data are presented in this section in order to indicate general trends and correlations leading to preliminary hypotheses about the performance mechanisms of the bridge. These observations helped determine the monitoring strategy for the subsequent system upgrades that included revision of the modal identification procedures and addition of a total positioning system (TPS).

6.1. Bridge performance summary, March 2007

Figure 17 shows the most obvious influence of structural temperature on bridge performance. Tensions in additional stay cable S2S (one of the short cables connecting the Saltash Tower to main span) and opening across the bearing between the main span and Saltash side span have strong negative correlation with temperature. This immediately suggests that temperature rise extends the main span horizontally thus closing the expansion joint and slackening the stay cable. The variation in cable first mode frequency is directly linked to the cable tension variations.

Figure 18 examines the correlations between the additional stay tensions and the mean deck temperature. Some of the 32 load cells in the 16 cables were unserviceable at this time but the effect is clear. First, the Plymouth end no. 2 and 4 cables (P2S/N, P4S/N) tighten as the corresponding Saltash end cables slacken, which supports the hypothesis of the extending deck girder. Second, the relationship is not linear but involves varying time delay (so the orbits of the tension data points are not elliptic with time) and offsets; this suggests that temperature is a major factor in performance, but not the only one.

Examination of complete sets for later months with all load cells operating shows seasonal effects in the correlations shown in Figure 18. In particular for No. 1 cables (P1S/N, S1S/N) which connect the Towers to the sidespans, the correlations have samm but varying average slopes and reverse for some periods.

Since the extensometers failed at the same time the deck level sensors became available, correlations of deck level with extension could not be examined. Displacement data are available from the electronic distance measuring sensor (EDM) aligned between the Tower top portals, but these signals clip at the lower range so are not a reliable data source.

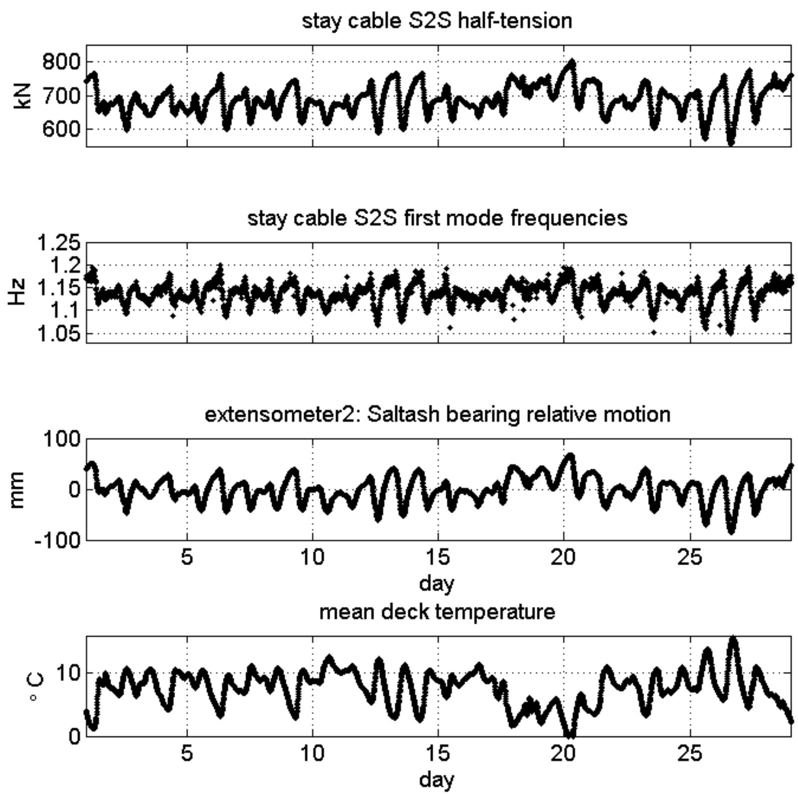


Figure 17: Static response, march 2007

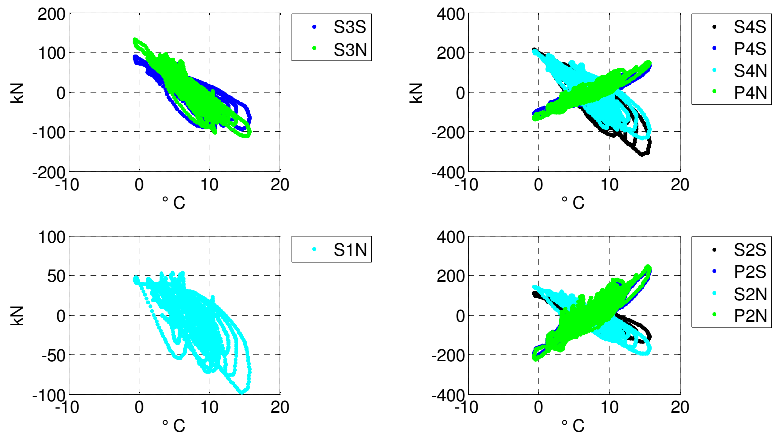


Figure 18: Cable Tensions and mean deck temperature, March 2007

6.2. Bridge performance, September 2007

Figure 19 shows variation of 1-hour average displacements and mean deck temperature in September 2007. At this level of resolution the daily variations of the deck levels appear to follow the deck temperature cycle but another possibility is average vehicle loading. At low resolution, analysis of this effect requires hourly vehicle crossing statistics according to total vehicle weight, which are provided only approximately by data as shown in Figure 26.

Figure 20 zooms in on one day of data (bold lines in Figure 20) sampled at 1Hz (right) in an attempt to show effects at high time resolution. This range of one-hour mean displacements is relatively low for the day studied but still the instantaneous values cover twice this range, with the likelihood that the large excursions are caused by passage of heavy vehicles.

Unfortunately the level sensing instrument itself samples at 0.1 Hz and a heavy vehicle moving at 30mph takes only 2 level sensing samples to cross the main span so the zoomed time series in the lower view of Figure 20 (corresponding to the bold part of the trace in the upper plot) cannot confirm quasi-static effects of vehicles.

To study the thermal effects further, Figure 22 (left) shows deck levels vs. mean deck temperature (zeroed for the lowest temperature). The correlations are very clear although some of them are positive i.e. the deck rises at the Plymouth end as temperature increases. If the bridge behaviour were dominated by the main cables, as in a normal suspension bridge, the deck would be expected to rise and fall with the same sense as the main cables contract and expand with temperature variations.

Figure 21 shows the first principal component of the deck level data over a period of 18 months. Like a mode shape, this is the dominant behaviour of the deck, with absolute scale varying with time, to which is added smaller effects.

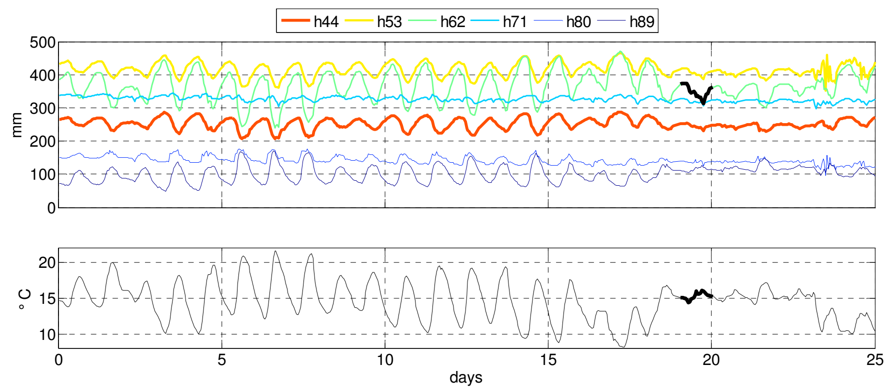


Figure 19: Levels and deck temperature

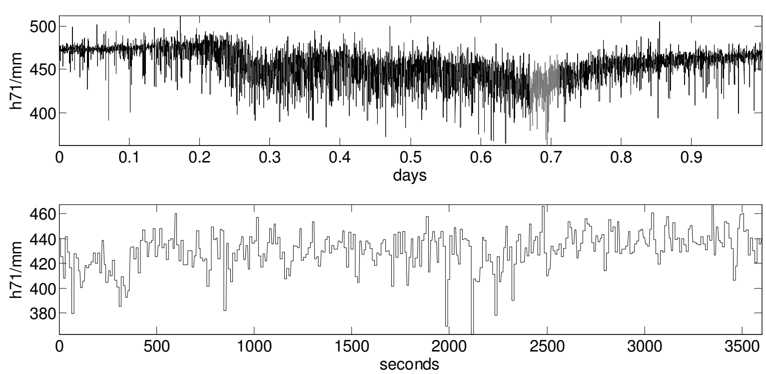


Figure 20: Level time history (detailed)

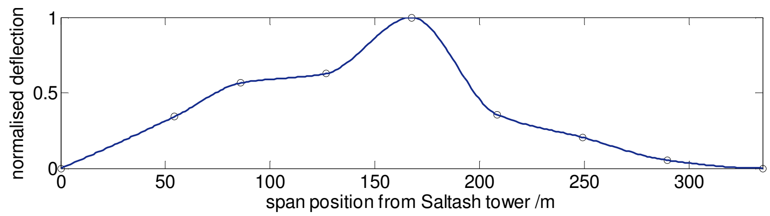


Figure 21: 1st PCA of level sensors at Deck

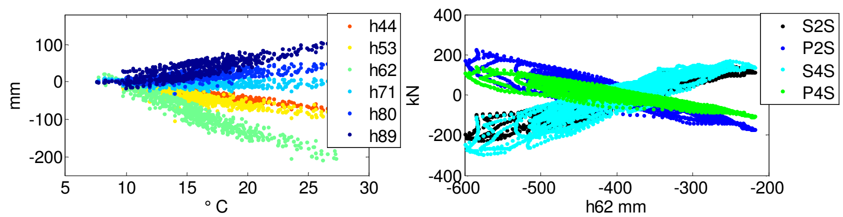


Figure 22: Correlation with deck level

The bias towards the Saltash side is consistent with the comparatively low or opposite sign deflections observed at the Plymouth side in Figure 21.

Figure 22 (right) shows that the cable tensions have the same form of correlation with deck level as with deck temperature, confirming the dominant effect of temperature.

6.3. Bridge performance, January 2008

The final form of loading to consider is wind. This is typically the major concern for a long span bridge but at 335m span Tamar Bridge is not likely to be as susceptible as the major UK suspension bridges such as Seven, Humber and Forth Road Bridge. Monitoring of Humber Bridge showed significant wind effects on static rotation and lateral drift, neither of which could be detected by the level sensing system at Tamar Bridge.

Dynamic effects of wind buffeting would appear in the variance of displacement in the low frequency modes, and thanks to the good performance of the servo accelerometers, double integration of response in either time or frequency domain provide a reliable interpretation of dynamic displacements.

The double-integrated acceleration signals of January 2008 (the windiest in the period 2007-2008) are shown in Figure 23 along with representative wind speed data, suggesting that significant vertical response is only engaged when wind speeds exceed 25m/sec.

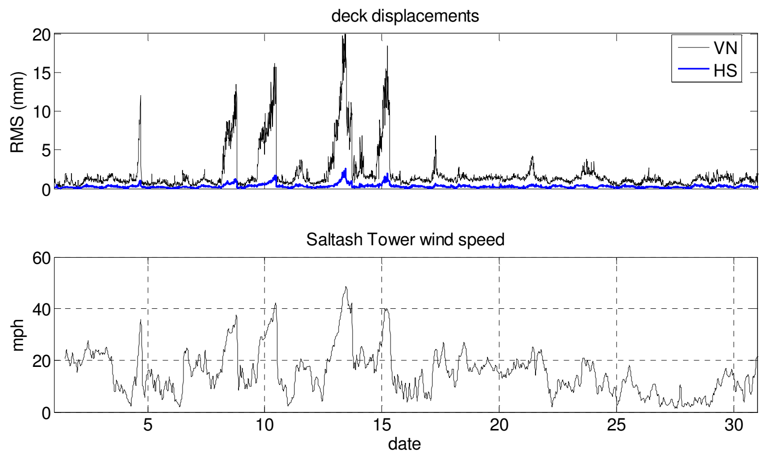


Figure 23: Deck Displacement and Wind speed

6.4. Loadings

Figure 24 shows variations of temperature until early 2009. Temperature change between winter and summer is about $13°C$ which is relatively small to the daily temperature changes. Figure 25 shows the wind rose plot of wind measurement until early 2009 on the top of Saltash Tower. Length of each spoke represents frequency of winds blowing from a particular direction and each spoke is broken down into color-coded bands to show distribution of wind speeds. In addition to the the traditional predominance of southwesterly winds, the south east quadrant has a strong contribution. Finally, Figure 26 shows numbers of HGVs, provided separately by the bridge management team emphasises the daily/weekly pattern and the strong peak loadings in early morning and/or late afternoon.

6.5. Modal Properties

Interpretation of dynamic behaviour is based on the unique (at the time) real time system identification procedure which reported deck and cable mode frequencies and damping ratios at 10-minute intervals with the modal information from the ambient vibration survey. These frequencies can be related to characterised vibration modes. Cable vibration mode frequencies have been shown to correlate strongly with cable tensions (as expected) but the big surprise is the large variation of deck modal frequencies, presented in Figure 27.

The modal identification by SSI intermittently fails to identify modes (due to the stringent quality criteria set in the automated procedure), but the figure shows five modes up to 8Hz. The modes around 0.4Hz and 0.6Hz are first symmetric and anti-symmetric vertical modes, while the mode close to 0.5Hz is the first lateral mode. All modes have significant variance but the lateral mode stands out, with a range exceeding 0.1Hz. The causes of the mode frequency

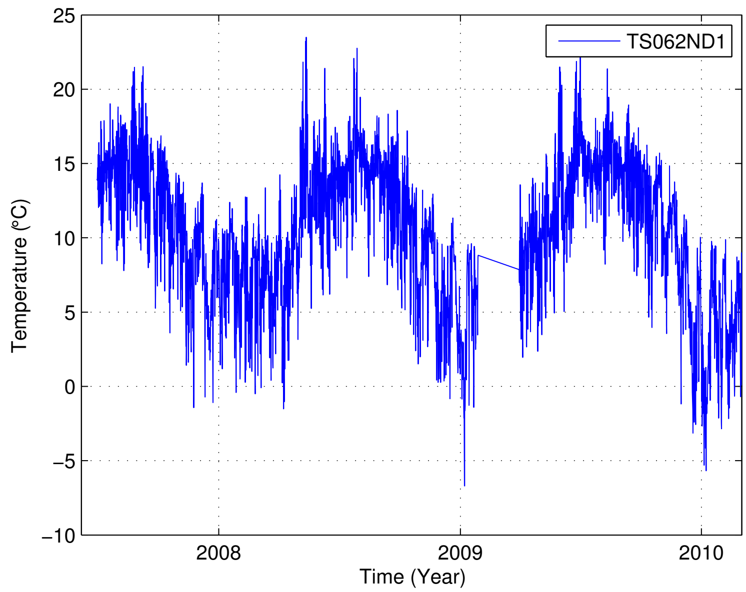


Figure 24: Temperature at Deck

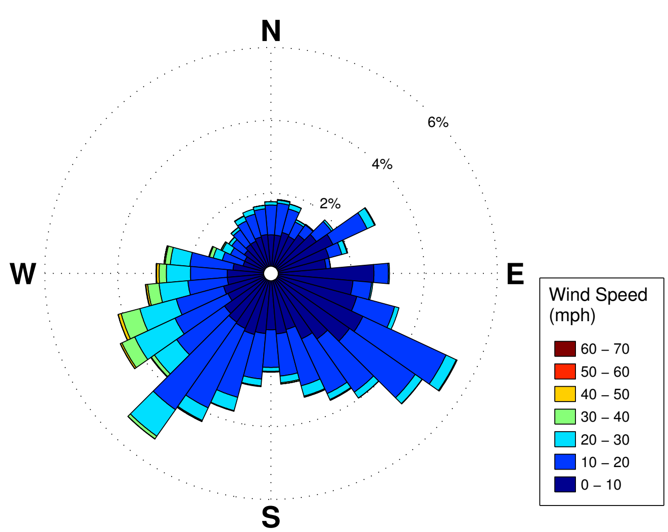


Figure 25: Wind Rose at Top of Saltash Tower

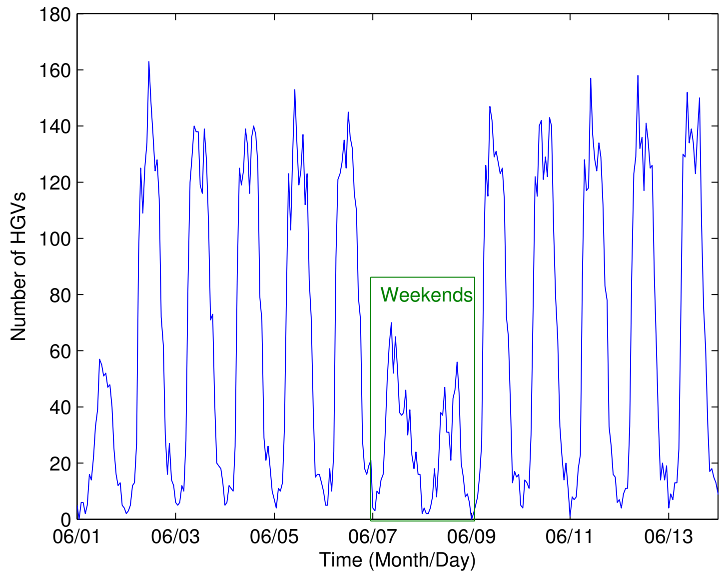


Figure 26: Typical traffic Loading

variation have yet to be established. There are also strong daily variations in cable mode frequencies that correlate almost perfectly with the tension values from the SMS.

Figure 28 shows the 1st deck frequency versus RMS of the vertical deck acceleration. For low wind-speeds between [0, 20] mph, there is a clear linear relationship. As the wind speed increases (>20 mph), and as response amplitudes rise, frequencies moves away from the linear relationship toward to right showing an evelope of exponentially decaying curve.

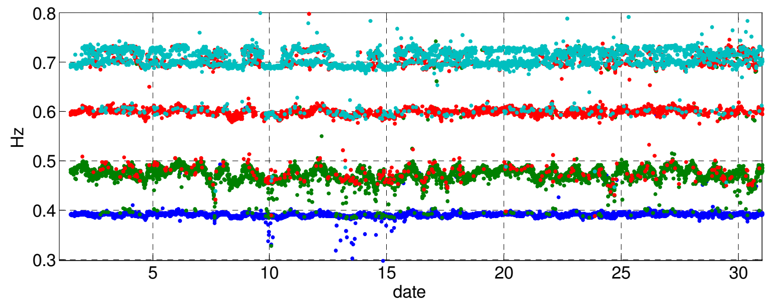


Figure 27: Deck mode frequency variation in September 2007

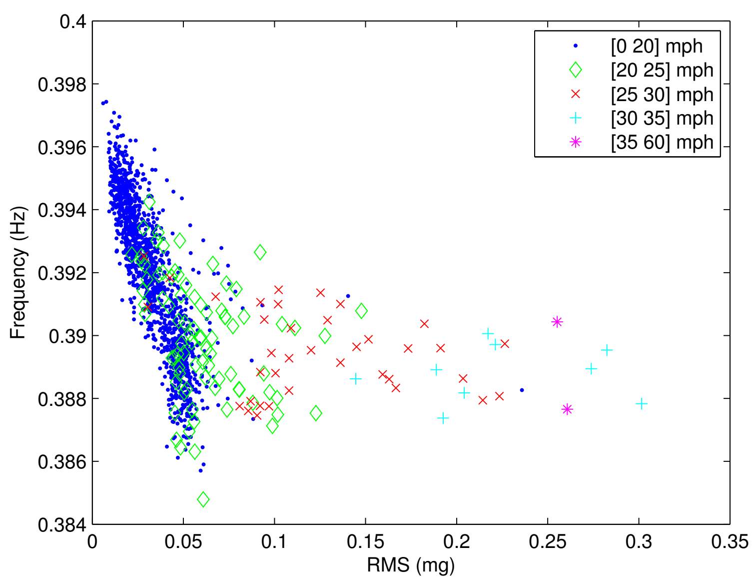


Figure 28: Frequency dependency on RMS and Wind Speed

7. Total Positioning System (TPS) for Deflection Monitoring

The preceeding discussion has indicated that thermal effects on bridge deformation appear to be the strongest environmental influence on bridge behaviour, and it is highly likely that this deformation leads to secondary effects on cable tension and even, through nonlinear effects, on modal parameters. Because proper characterisation of deformation is a prerequisite to understanding the mechanisms at work in the bridge, a number of possibilities were considered for detailed study of global bridge deformation. First, since is only the slowly varying movement that is of interest there is no need for fast-sampled intertial based systems, nor for high-speed GPS sensing. GPS was considered but ruled out due to the requirement for multiple high-cost sensors which would in any case be challenged to resolve the low levels of displacement experienced on the bridge. Of the methods available for the task (Brownjohn and Meng, 2008), total positioning system (TPS) technology offered the best solution, and a system was designed and installed on the bridge in 2008.

The TPS Leica TCA 1201M (Figure 29a) was installed at the roof of Tamar Bridge Office as shown in Figures 30 and 31. Fifteen reflectors (Figure 29b) are installed on the bridge including the south ends of the bridge deck and on tops of both Towers. Reflectors are located off the fence so that pedestrians couldn't access.

Figure 33 represents the range of deformation, so far observed, of key reflectors in the vertical plane of the bridge. Clearly the axial extension is accompaied by lowering of the deck biased to the Saltash end. Except for Plymouth tower, other deformations are small and mainly vertical. For logistical reasons it has not been possible to recover motion across the Saltash expansion joint using TPS.

(a) (b)

Figure 29: Total Positioning System, Leica TCA1201M and Reflector

Figure 30: Birdview of Tamar Bridge

Figure 31: TPS for Tamar Bridge Monitoring

Figure 32: Reflector on the bridge deck

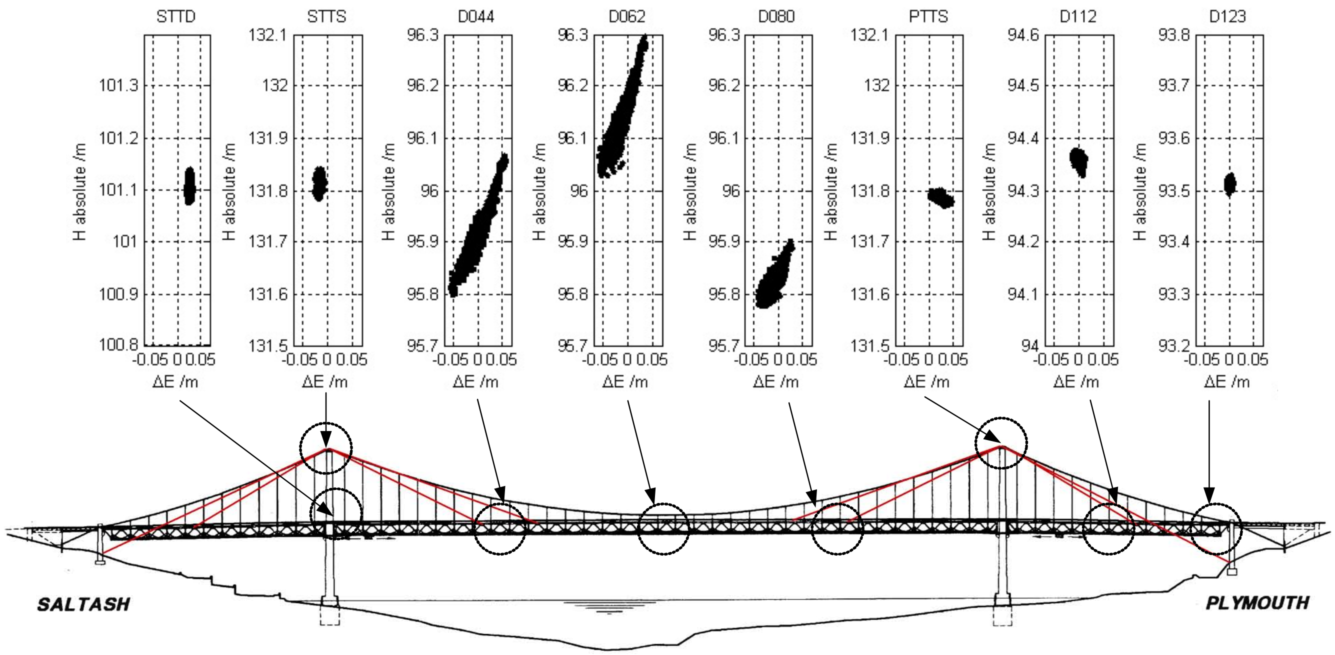


Figure 33: Longitudinal (Easting) and Vertical (Height) movements of Bridge

8. Conclusions

The paper describes the multi-component instrumentation of Tamar Bridge and presents selected performance observations. Tamar Bridge turns out to be rather complex structure, a hybrid of cable stay and suspension bridges, with unusual boundary conditions. As such the behaviour of the bridge has been a challenge to unravel and the mechanisms at work are still far from clear, What does seem to be apparent is that thermal effects are the major driver of deformation. There are surprisingly large variations of modal frequency, particularly for the fundamental (symmetric) lateral mode and these could result from a mixture of effects including varying deformation, wind, response levels and traffic loads. In fact the modal parameter variations are the most mysterious which in turn suggests that peeling off the various contributions could lead to some interest performance diagnosis possibilities. Even after four years of study with various configurations of the monitoring system, data are insufficient for identifying the 'normal' behaviour leaving a great challenge in detecting performance anomalies. In fact almost every aspect of the bridge performance is in some sense anomalous, and present research focuses on opposite extremes of data-driven assessment tools and validated finite element model simulations.

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