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Structural Health Monitoring of short to medium span bridges in the United Kingdom

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Abstract. Historically the UK has been a pioneer and early adopter of experimental investigation techniques on new and operation structures, a technology that would now be descried as 'structural health monitoring' (SHM), yet few of these investigations have been enduring or carried out on the long span or tall structures that feature in flagship SHM applications in the Far East.

Keywords: bridge structural health monitoring

1. Introduction

1.1 Overview of UK bridge structural monitoring practice into 21st Century

Structural Monitoring (SM) is defined in BD79/13 of the UK Design Manual for Roads and Bridges (Highway Agency UK, 2013) as:

"*periodic* or *continuous observation* and *recording* of *information* pertaining to structural behaviour, in order to detect *deterioration* or distress should it occur, to determine the extent, severity and rate of deterioration, and to determine whether a critical limit state or other criteria are at risk of being reached".

Such monitoring is routine, if low-tech, for operational dams and is a different type of monitoring compared to measurements settlement or (peak) construction-induced vibrations during major constructions. For other classes of civil structure e.g. bridges, monitoring is relatively

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rare, but is beginning to be used more widely due to demonstrable benefits.

SM as described in BD79/13 is specifically intended as a tool for assessing sub-standard highway structures, and ranges from periodic visual observations to remote logging, whereas structural *health* monitoring SHM is a term usually associated with elaborate and high-cost implementations on flagship structures such as long span cable-supported bridges. It is arguable that the SHM system designed for the Queensferry Crossing (the replacement for the Forth Road Bridge) is the UK's first true SHM system of the type now commonplace in the Far East and increasingly in North America where dynamic response is monitored along with quasi-static thermal and structural response to operational loads from traffic and weather conditions.

The beginnings of 'formal' i.e. periodic bridge SM in the UK are difficult to define, due to the blurred distinction between (e.g.,) load testing campaigns (Thomson 1981) and repeated measurements. Among the load testing studies, the significant contributions were post-war studies on road bridges, specifically arches (Chettoe and Henderson 1957, Davey 1953). For rail bridges comprehensive studies of loads (Hayward 2014) led to the development of loading standards although relatively little literature is available in public domain. One example is a combined model, analytical and field measurement on the Brunel's Royal Albert Bridge that runs parallel to Tamar (road) Bridge and predates it by 100 years, (Leeming and Whitbread 1974). A more recent (but still rare for UK) public domain report (Spencer *et al.* 2012) is of strain measurements for an overbridge on the (London) Docklands Railway have been reported.

In in the UK there is an established track record of both short term dynamic investigations, also called modal tests, and the usually distinct longer term SM investigations of quasi-static response to operational loads. Both types of investigation were undertaken by Transport and Road Research Laboratory (TRRL) in the 1960 and 1970s, with research activities developing in the 1980s onwards mainly involving the University of Bristol and more recently a number of monitoring specialists.

TRRL bridge monitoring projects up to 1974 (Mortlock 1974) were focused on temperature effects on longitudinal movement across expansion joints and included:

- Medway Bridge from 1964 to 1966, studying creep and shrinkage in concrete using vibrating wire (strain) gauges (VWGs);
- Hammersmith Flyover from 1964 to 1965 using thermocouples and a single displacement transducer between the two continuous but multi-span 249 m and 377 m sections.
- Beachley Viaduct/Wye Bridge (an extension of the Severn Bridge on the original M4, now the M48) from 1966 to 1967 to determine the range of temperature and amount of movement experienced by the steel box deck;
- Coldra viaduct from 1967 to 1971. This has 254 m long pre-stressed and precast concrete recast beams with cast in-situ slab. Eight thermocouples were installed and two displacement transducers were installed across the single expansion joint.
- Mancunian Way, from 1967 to 1971. The structure comprises twin 985.3 m pre-stressed concrete boxes with cantilevered carriageways and expansion joints at extreme ends. Thermocouples and heat flow meters were installed on a box, with one expansion joint monitored with a pair of displacement transducers.
- Around 1987, vibrating wire gauges (VWGs) were used in monitoring of new post-tensioned concrete box girder bridges in Cardiff, comprising glued segments, to study the shrinkage effect (Barr *et al.* 1997).

The TRRL research on performance of expansion joints continued with a seven year program of performance monitoring (Price 1984), using a combination of measurements and visual inspections, and the interest continued with subcontracted studies of expansion joint performance (Johnson and McAndrew 1993).

Orthotropic steel bridge decks such as used in Severn and Humber suspension bridges (among many other examples) were prone to fatigue due to stress concentrations due to detailing. Orthotropic deck use in these bridges was supported by monitoring of deck sections inserted into a trail section of carriageway of the A40 trunk road as part of a 9 year investigation into fatigue of steel decks (Beales and Cunninghame 1992, Nunn and Morris 1974).

More conventional monitoring projects (by 21st century standards) have involved cable-stayed bridges prone to vibrations induced by vortex shedding. Hence, wind and vortex-induced acceleration response measurements were made on Wye Bridge in from 1977 to 1978 and Erskine Bridge from 1979 to 1981 (Hay 1984). After the opening of Kessock Bridge in 1983, TRRL monitoring vortex-induced vibrations and vertical tuned mass damper (TMD)s were installed (Cullen Wallace 1985). The bridge was further monitored by a team from University of Bristol in 1992 when it was found that the TMDs had stopped working (Owen *et al*. 1996). University of Bristol researchers collaborating with Politecnico di Milano were also heavily involved in studies of the wind-induced static and dynamic response of the Humber Bridge in 1990 and 1991 (Brownjohn *et al.* 1994). The purpose of this exercise was calibration of software to be used to simulate performance of the proposed 3 km span Stretto di Messina suspension bridge. A simpler program of monitoring of Humber Bridge using updated technology such as GPS began in 2011 (Brownjohn *et al.* 2015) and provided data for replacement of the old A-frame rockers with pendles for the main span (Hornby *et al.* 2012).

The refurbishment of the Severn Bridge and the companion Wye Bridge and Beachley Viaduct between 1985 and 1991 involved numerous monitoring works. These included strain measurements on a dummy inclined hanger and program of measurements of main span acceleration and winds by TRRL before (1982-1984) and after (1989-1990) the strengthening and hanger replacement (Flint *et al*. 1992). The strengthening also included strain, load and deflection monitoring of the jacking posts used to carry loads from the saddles during reinforcement of the steel towers, and profile measurement of the adjacent Wye Bridge using a level sensing system (Evans 1992).

Monitoring of response to wind was also carried out on the Second Severn Crossing during and after construction (Macdonald *et al.* 1997) since the bridge was prone to response due to vortex shedding (Macdonald *et al.* 2002).

Wind effects on stay cables were a concern for the Flintshire Bridge (Dee Crossing), completed in 1998 (Curran and Tilly 1999), so these were instrumented with both accelerometers and load cells. For a heavy vehicle load test, VWG strain gauges were installed to track static strains, with accelerometers and electrical resistance strain gauges to track dynamic effects.

Acoustic emission monitoring has been introduced to the UK in the last 20 years following TRRL trials on their site and on Huntingdon railway viaduct (Cullington *et al.* 2001). The technology has proven to be highly effective for tracking strand breaks in main cables of Forth Road Bridge and Humber Suspension Bridge (Barker *et al.* 2013) and also during investigation of the Hammersmith flyover, described in this paper.

Expertise in bridge monitoring projects has been accelerated in the UK in the start of the 21st century with a number of contractors (e.g., Strainstall, Datum -using technology from US Bridge Diagnostics Inc. and MISTRAS) and consultants e.g., Flint and Neill (F&N). F&N in particular have engaged in monitoring at various levels, from specification to operation. As part of the Wye

Bridge strengthening beginning in 1987, a novel level sensing system was designed to track automatically levels during rearrangement of the stay cables. The sealed system relying on pressure changes rather than fluid movements was successful and incorporated in F&N SHM system specification for the Lantau fixed crossing and installed by Fugro Structural Monitoring Ltd. The system was trialled on the Erskine Bridge and operated for several years on Tsing Ma Bridge, but while it had superior resolution compared to GPS, the system had inferior frequency response and reliability, and was eventually replaced. The same system was installed on the Tamar Bridge as part of the system installed by Fugro to track the strengthening and widening of the Tamar Bridge, completed in 2001, and continued to operate for several years.

Clearly, suspension bridges are popular candidates for monitoring. Apart from the two comprehensive long-term monitoring exercises of Humber Bridge, there have been long-term deployments of wireless sensing in the anchorage (Hoult *et al*. 2008) and short term GPS deployments (Ashkenazi and Roberts 1997), while GPS has also been deployed on Forth Road Bridge for evaluating traffic-induced quasi-static deformations (Roberts *et al*. 2012).

Finally, an elaborate SHM system, modelled on the systems operating on the Lantau Fixed Crossing, is at the time of writing currently being installed by Strainstall on the new Queensferry Bridge (Scullion 2016), which is intended to replace the Forth Road Bridge. While the Queensferry SHM system aims to provide information during construction and from early life, the old bridge experienced problem of wire breaks (tracked by acoustic emission monitoring) and more recently by fatigue failure of a truss end link, resulting in complete closure of the bridge over Christmas 2015. The link is one of a pair that forms an articulated linkage between the ‘truss’ girder and tower, designed to take live loads. While the other three main girder truss end links exhibit no damage, all are being monitored to characterize the movement and loading, since there is uncertainty about the actual loads carried by the truss end links and the state of the bearing pins, so monitoring technology developed for Queensferry and Humber Bridges was being applied. As part of the investigation load tests were undertaken with a fleet of heavy lorries and the SHM system, which has been referenced in Scottish Parliamentary proceedings (Scottish Parliament 2016) has been key to resolving the problem and allowing the bridge to reopen to all traffic.

2. UK bridge monitoring case studies

The few recent public domain publications on UK bridge monitoring, including those by the first author, originate from academic curiosity even if they have outcomes of practical benefit. There have also been a number of dynamic investigations on footbridges relating to vibration serviceability (Fletcher and Parker 2003, Pavic *et al*. 2002) but these are short term measurement ‘campaigns’.

Hence we present three recent examples of practical UK bridge structural monitoring in the 21st century, initiated by operator requirements, are described here, aiming at identifying the practical benefit to operators. The case studies concern ‘quasi-static’ monitoring primarily due to thermal effects and not involving dynamic response.

2.1 Cleddau Bridge

The Cleddau Bridge (Fig. 1) located in South-West Wales was opened to the public in 1975. Being 819.4 m long, the seven-span steel box-girder bridge spans North to South between Pembroke Dock (at its North end) and Nyland (South end) across the estuary of the River Cleddau.

The bridge had daily traffic of 12,300 vehicles in 2013 (Department for Transport 2015), and has served more than 5 million vehicles since opening.

The bridge rests on six piers across the River Cleddau and on an abutment at each end. Each pier is designed as a fixed column; top ends of the piers are pinned to the bridge and the bottom ends fixed on bedrock. Bridge spans range from 76.8 m to 213.4 m with the longest span having a suspended portion that is hinged at its southern end via rocker bearings, and propped on two roller bearings at the northern end. As the bridge is aligned along the north-south axis, the east-facing and west-facing sides of the bridge are directly exposed to the sun during hours of sunrise and sunset respectively. This leads to strong temperature gradients in the transverse direction, i.e. across the box girder. The temperature differentials cause one side of the bridge to expand much more than the other and thereby create significant plan bending (i.e., bending about the vertical axis).

2.1.1 Monitoring of the Cleddau Bridge

Monitoring was initiated in October 2011 since roller bearings of the bridge, which had been in operation for nearly forty years, were visibly close to the end of their service life. Fig. 2 shows the worn-out teeth of the rack and pinion at one end of a bearing at the commencement of monitoring. The hole in the pinion, which is centred on the main cylindrical axis of the bearing, has also been damaged, as can be seen from Fig. 2(c). The forces on the bearings from plan bending are thought to have accelerated the deterioration process that may have been initiated by a lack of appropriate protection for the bearings from the harsh environment. A combination of the effects of corrosion, fatigue and excessive forces led to the fracture of a flange, which in turn initiated the monitoring of the bearings. The monitoring was started initially to detect adverse behaviour of the bearings. It was then continued in order to understand bearing performance under operational and environmental loadings so as to inform the design of new replacement bearings, which were installed in May 2014.

Fig. 1 The Cleddau bridge (looking from east of Pembroke dock) (left) and its geographical location (right). (Courtesy: Pembrokeshire County Council)

2.1.2 Monitoring system

The monitoring system consists of a number of sensors for measuring temperatures and displacements. Twelve one-wire digital temperature sensors were installed a few meters away from the centre of the suspended section of the bridge, on the inside of the box-girder - three sensors on each of its faces, as shown in Fig. 3 (left), to record surface temperatures every minute. Fig. 3 (right) indicates the locations of sensors measuring structural displacements.

To measure the gap between the suspended span and northern section of the bridge, two linear pull-wire potentiometers connect to centre of the inner and outer faces of each bearing. Inner in this context refers to the bearing face that is closer to the vertical centre line of the girder. The potentiometers are setup to measure longitudinal movements at the two ends of each roller bearing, with sensor installed approximately 500 mm from the outer ends of each bearing. These two measurements are further referred to as the “east gap” and “west gap” measurements.

Analysis of the initial monitoring data indicated that the bearings twist and lock due to thermal effects. The pinions at the ends of the roller bearings were observed to be restricting the twisting movements, and hence large forces were thought to be transferred via the flanges of the bearings to the teeth of the pinions and eventually to the racks. These forces are likely to have caused the damage shown in Fig. 2. The pinions and the racks were removed in order to release the twisting constraint.

2.1.3 Analysis of measurements

Fig. 4 (left) shows temperature variations over two years collected with a temperature sensor on the top face of the box-girder. Measured temperatures reflect seasonal trends. The measured temperatures from all sensors for three consecutive days in April 2013 are plotted in Fig. 4 (right).

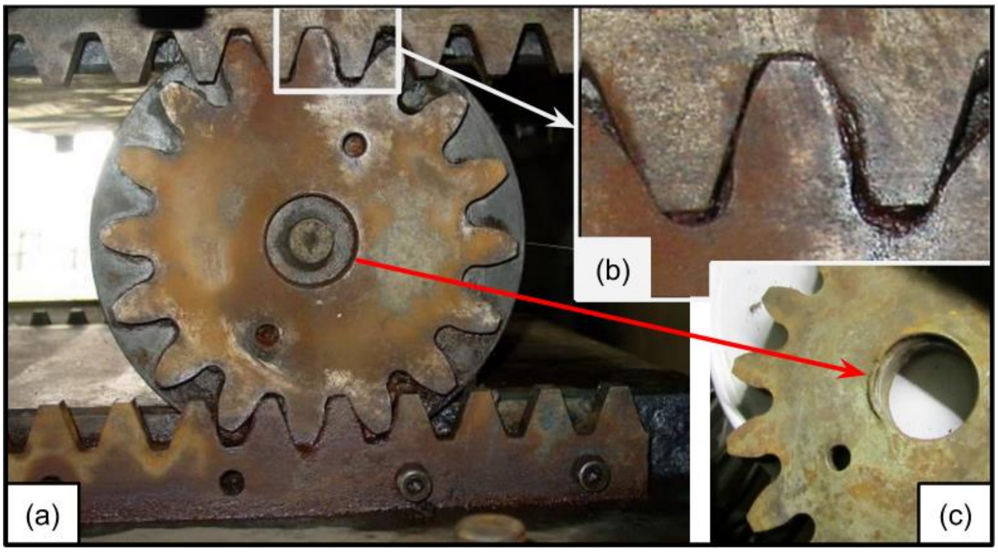

Fig. 2 Damaged pinion: (a) front view, (b) closer look at worn out teeth and (c) central support. (Courtesy: Pembrokeshire County Council)

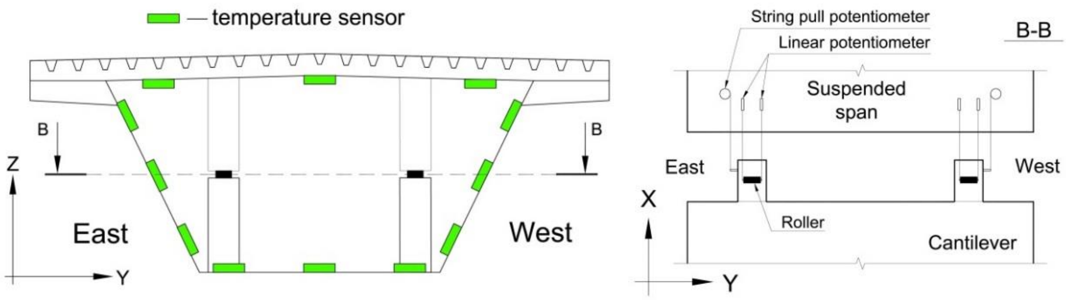

Fig. 3 The location of one-wire digital temperature sensors (left) and displacement sensors (right). In the figure, x-axis is aligned longitudinally along the bridge and z-axis is the vertical axis aligned with gravity

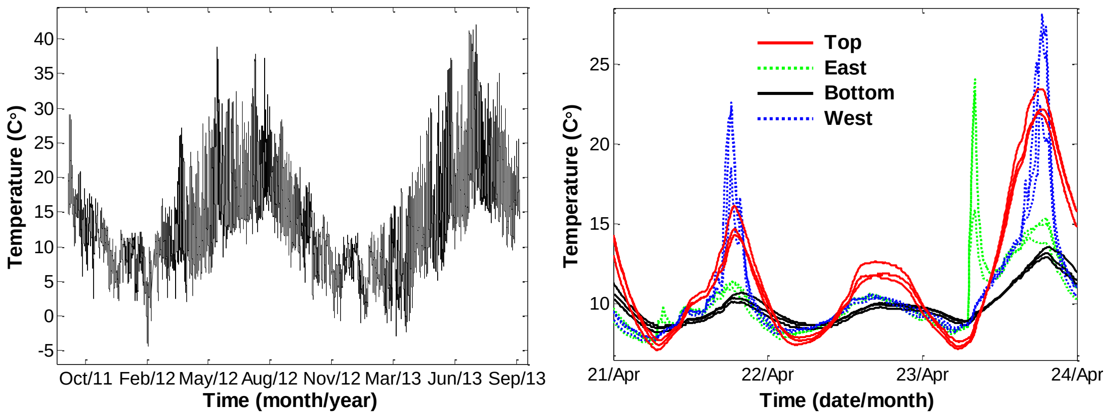

Fig. 4 Temperature measurements from the top surface of the girder for two years (left), and time-histories of measurements from all twelve sensors over a three-day period (right)

Variations of temperature across the width of the box-girder similar to that on April 23 can produce plan bending of the bridge, which then leads to plan twists at bearings. The magnitude of the twists is the difference between the displacement measurements at the inner and outer ends of the bearings. Fig. 5 (left) shows the time-history of the differences between raw measurements taken at the inner and outer ends of the west bearing. The figure plots results from measurements taken at a frequency of 1 Hz. The data are extremely noisy, making visualizing of trends difficult so Fig. 5 (right) plots the same data as in Fig. 5 (left) but after smoothing. While smoothing reduces the magnitudes of the peaks, the resulting plot is sharper and also more useful for illustrating comparisons with predictions from numerical models. If there is no plan twist, the differences should be zero. However, the difference between measured movements of the inner and outer ends varies from around -0.23 mm to +0.23 mm in the smoothed data-set.

The mean distance travelled by a roller bearing is computed as the average of measured movements at its inner and outer ends. On April 23, the bearing moved about 60 mm over the course of a day primarily due to thermal effects. Note that this ignores vehicular effects as the bearing movements may have to be measured at a much higher sampling rate than 1 measurement per second to capture accurately the movements due to vehicular passage.

The sample rate for bearing displacements is sufficiently high to reveal that bearing movements are seldom smooth (Fig. 6). Movements happen incrementally and friction plays a significant role. Bearings are seen to lock briefly and then to release especially during periods when temperatures change rapidly. A certain slip force is required to initiate the release. This phenomenon has also been observed in previous research (Emerson 1976). In Fig. 6, the abrupt drops in east-gap displacement that take place every 300 to 600 seconds are representative of this phenomenon.

2.1.4 Results from monitoring

Measured temperature gradients are observed to be significantly different from the code-specified temperature distribution scenarios. Therefore the scenarios given in the Eurocodes alone are insufficient to study the performance of the Cleddau Bridge. For this structure, temperatures vary not only along the depth of the box-girder but also along its breadth. Transverse temperature gradients are however not captured accurately in the Eurocodes. These results show that performance assessment of bearings using temperature gradients given in the design codes can be unreliable. This is primarily because the codes were developed to take into account the uncertainties in thermal variations during the design of a structure, while a structure's in-situ performance will depend actually upon the experienced daily and seasonal temperature variations. The understanding of temperature variations and bearing movements obtained from monitoring was subsequently employed to choose appropriate replacement bearings for the structure. More specifically, the new bearings were designed to accommodate the range of observed movements.

2.2 Hammersmith flyover

The Hammersmith Flyover opened in 1961 with 16 spans in a continuous 622 m elevated section with typical span 42 m. Details of design and construction are provided by (Rawlinson and Stott 1962); the construction used post-tensioned precast concrete segments (Fig. 7) and was designed to be maintenance free, featuring electric roadway heating to 'combat winter hazards to traffic,' although the system was decommissioned in 1962 owing to high energy costs. The flyover now carries 70,000 vehicles/day.

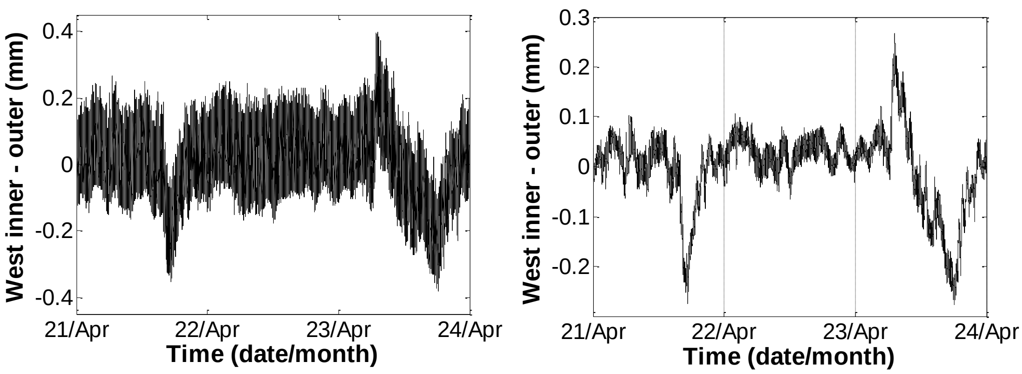

Fig. 5 Raw (left) and smoothed (right) time-histories of differences between the movements measured at the outer and inner ends of the west bearing

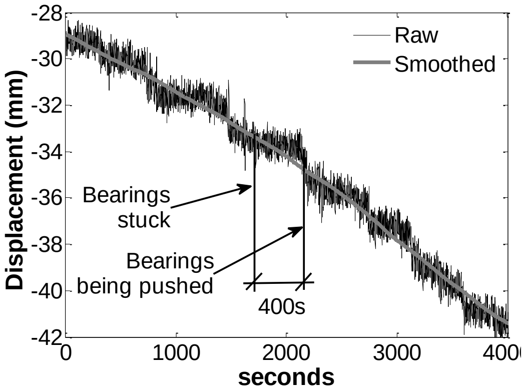

Fig. 6 Measurements over a one-hour period showing bearing locking and release (right)

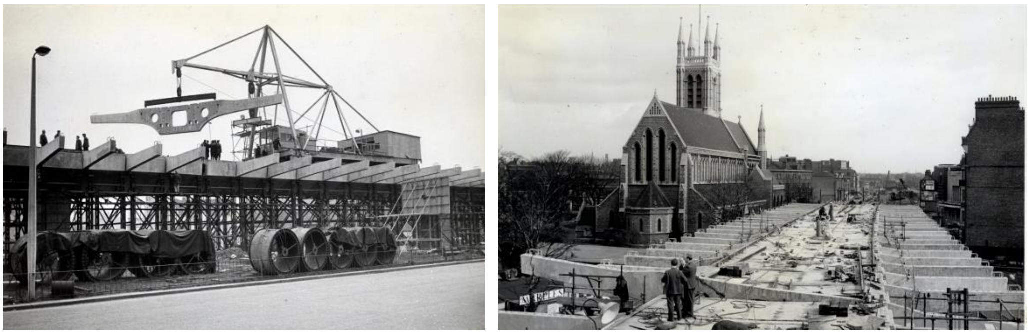

Fig. 7 Flyover construction showing cantilever unit being dropped into place between beam segments

A series of measurements (effectively monitoring) of cable tensions, temperatures and movements were made during construction (Wroth 1962) with the aim of identifying the behaviour of the 'prestressing strand in service conditions' and the extent of thermally induced movement of the superstructure and behaviour of the roller bearings. The measurements showed these movements to be 20% less than designed for.

Post-tension special inspections (PTSI) were carried out by the previous owner in the decade before ownership was transferred to Transport for London (TfL) in 2000. In 2006 the PTSI to date was reviewed and the structure assessed for capability to carry 40 tonne vehicles.

An enhanced PTSI was done in 2008 and in 2010 systems monitoring structural health and consequently acoustic emission were installed to monitor the deteriorating health of the structure.

The AE monitoring used seven systems with 600 sensors to detect wire breaks (Fig. 8) and to drive a deterioration model for predictions on remaining life before loss of reserve capacity.

Different aspects of the structural health monitoring system (Fig. 9) have different purposes. Strain and rotation at top and longitudinal movement at the bottom of each of the 15 piers were measured. Free movement of bearings is intended to prevent development of bending moments due to thermal expansion of the elevated section so these measurements would help to identify any 'locked in' forces, which could be having a further negative effect on structural capacity.

Fig. 8 Failed wire strands (left) and acoustic emission sensor (right)

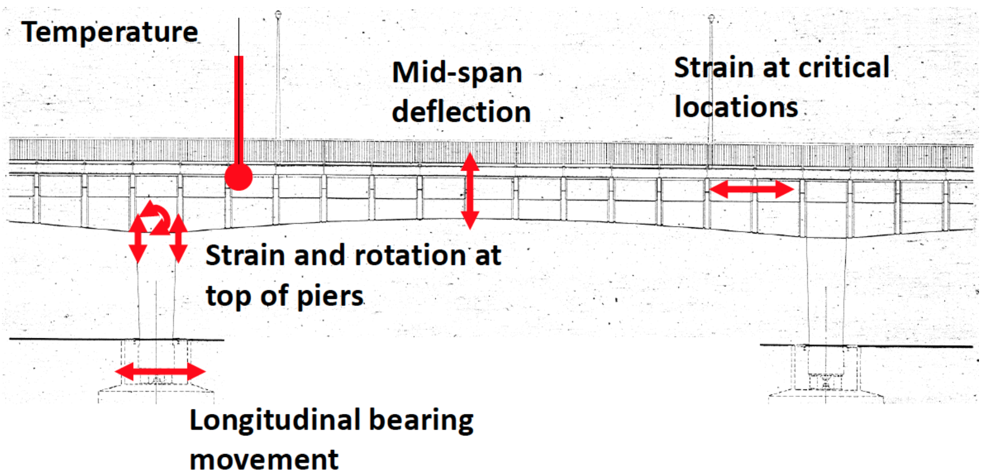

Fig. 9 Instrumentation summary

Fig. 10 Base of pier and extensometer for bearing movements

LVDTs were also placed across segment joints between adjacent beam segments (across cantilever units as shown on Fig. 7) at critical locations. These would be able to check for joint opening caused by loss of compression as strands failed. Loss of compression in a span would also result in increased mid-span vertical deflection in that span, which was monitored using three total stations and multiple prisms inside the structure.

Since any progressive changes would be partially obscured by effects of diurnal and seasonal temperature changes, temperatures of air, of roof and of floor concrete were measured inside the box as well as external (in shade) air temperature for use in the analysis of the data.

Two useful observations were made using the SHM data: first, there was no anomalous mid-span deflection and no opening of critical joints, suggesting that there was no significant loss of compression, and hence assuring of the structure's short-term safety; second, the bearings at some piers were not moving as expected, but the strain data did not provide enough information to quantify locked in forces.

As an interim measure the critical span was propped in 2011 Fig. 11 (left), and following further intrusive investigations a decision was taken to close the flyover to all traffic on 23rd December 2011, resulting in severe traffic disruption. During the closure, more intrusive investigations were carried out to determine with greater accuracy and resolution the condition of the post-tensioning system. Subsequent refinement of the condition factors allowed the bridge to reopen with one lane in each direction to light vehicles (7.5 tonne limit) only while emergency strengthening works were undertaken.

The emergency strengthening works involved the construction and installation of an external post-tension system across the top of five critical piers Fig. 11 (middle) to restore compression and full load carrying capacity to those areas of the structure. All lanes were reopened to all traffic on 30th May 2012 in time for the London 2012 Games, while a second project was initiated to complete strengthening of the remainder of the structure (Fig. 11). Structural health monitoring, including AE, continued in the interim and provided data to the deterioration model. The deterioration model continued to predict the date at which zero reserve capacity would be available, and thus provided a deadline for the completion of the strengthening works.

The second project, implemented between October 2013 and November 2015, installed a new post-tensioning system throughout the structure to supplement the emergency works, as well as undertaking a number of refurbishment activities including replacement of bearings and expansion joints. Extensive monitoring was used during the construction to provide assurance of structural safety and to measure the effects of construction; for example, stressing of tendons and jacking of the structure for bearing replacement.

Fig. 11 Temporary propping during investigations (left), new external post-tensioning across critical spans (middle) and additional strengthening (right)

During the works the AE system became less effective due to noisy construction activities. Monitoring of critical segment joints became the primary warning system as any excessive opening or unpredicted behaviour would suggest a loss of compression. High frequency (10 Hz) measurements were taken in order to be able to detect live load effects, although this was less effective than expected.

A reduced monitoring system remains in operation to monitor bearing movement and to detect any opening of critical segment joints. This provides further assurance of the success of the works, which included many innovations and non-standard construction.

2.3 River Trent Bridge

The River Trent Bridge is located in the UK East Midlands between Nottingham and Derby. It was built in the mid-sixties and spans in the North-South direction. It is a part of the M1 motorway and serves as the crossing over the floodplain adjacent to the River Trent. According to the Department for Transportation (Department for Transport 2015), the annual average daily traffic flow across the bridge in 2013 was estimated to be 136,000 vehicles.

The bridge has 21 spans in total, each supported on either side by 8 piers (Fig. 12). The distribution of piers may be represented using a grid layout, where the vertical rows are labelled alphabetically from A to V in Fig. 12, and the horizontal rows are labelled numerically from 1 to 8. The 97.3 m long floodplain#2 section on the left side in Fig. 12 (bottom) consists of four deck-on-pier elements, which are connected together with pre-cast slabs. Remaining spans in the 78.2 m floodplain#3 section are made of deck-on-beam systems. Decks at each end of the bridge are supported on abutments. Piers are supported on individual footings, which are cast on concrete piles, four piles for the deck-on-pier system and three piles for the deck-on-beam system.

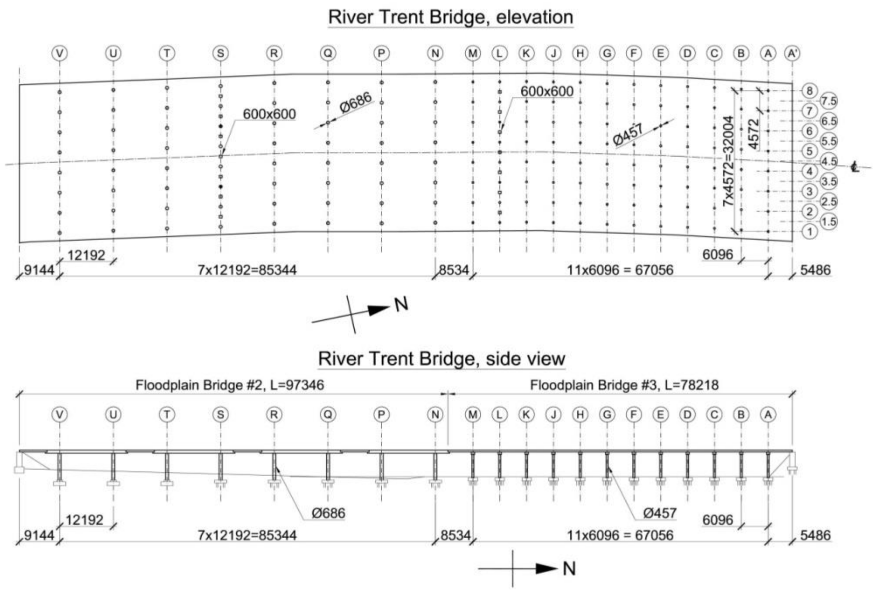

Fig. 12 Sketch of the River Trent Bridge: elevation (top) and side view (bottom)

Fig. 13 Alkali-silica reaction on a footing of a pier of Floodplain #2 (left) and piers subjected to full scale tests on the grid-line S (right). (Courtesy: Highways England)

The cement and the aggregate used in the construction was prone to alkali-silica reaction (ASR) as was later found out through testing; Fig. 13 (left) shows ASR on the surface of the footing of a pier. The Highways Agency conducted a series of full-scale load tests on certain footings suffering from ASR (Fig. 13 (right)). Loading tests demonstrated that footings can carry much higher loads than they were designed for, however, in order to detect any change in the bearing capacity of the piers due to ASR, a monitoring system was installed on the bridge in 2004.

2.3.1 Monitoring

The purpose was to provide support to regular inspections and to detect changes in the structural performance of the piers. The system comprises 150 VWGs and 8 thermocouples (TH). VWGs and thermocouples are referred to as VW-i and TH-i, respectively, where *i* is a number of a specific sensor. VWGs are installed on all piers between grid-lines A and S (Fig. 12).

All sensors except VW-3 are installed on the west faces of piers, approximately 300 mm from the connection to the beam or slab. Temperatures within the concrete deck slab, abutment and air can be measured once every hour, although for most of the monitoring period, between 2004 and 2014, only one measurement per day at 4AM was recorded.

2.3.2 Thermal effects

The bridge is oriented in a north-south direction hence in mornings and evenings, the east and west sides of the bridge respectively are exposed to the sun. In the afternoons the deck of the bridge receives significant solar radiation. There are no large obstacles adjacent to the bridge, hence temperature distribution is expected to vary equally along the length. Temperature distribution along the depth and width of the bridge depends on the location of the sun. Thermal images of the east side of the bridge with piers on grid-line 1 close to the front are shown in Fig. 14. These images show that the surface temperature of the east side of piers is up to 10℃ warmer than that of the west side.

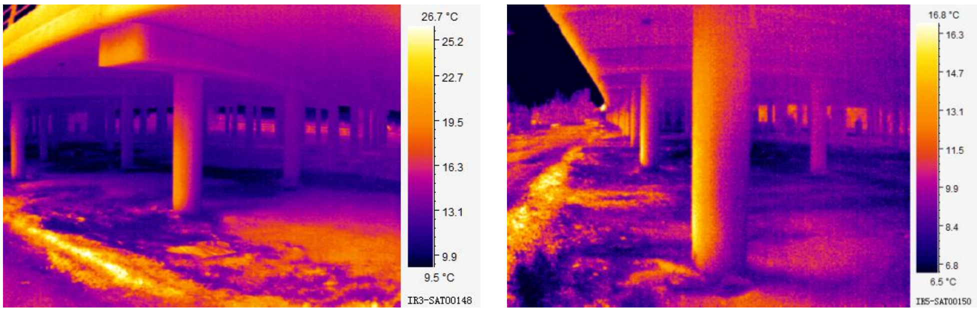

Fig. 14 Thermal images of the River Trent Bridge east face on the morning of 9th April 2014

These temperature variations have a significant effect on bridge behaviour, as indicated by collected measurements that have to be pre-processed carefully prior to interpretation. Strain measurements are particularly noisy and contaminated with outliers. The outliers are removed with the interquartile range technique (IQR) (Kromanis and Kripakaran 2014).

Fig. 15 plots temperatures measured with sensors TH-1 and TH-2, and strains measured with sensor VW-32 over a week in late April 2011. While sensor TH-1 measures the ambient temperature, TH-2 measures temperatures within the slab. Sensor VW-32 is installed on a pier located on grid location (C,8) where strains are expected to increase when temperature rises.

Fig. 15 illustrates that the time lag between the peaks measured by i) TH-1 and VW-32 is 12 hours, and ii) TH-2 and VW-32 is 8 hours. These time lags are due to thermal inertia effects in concrete. This example shows that measurement collection frequency has to be sufficiently high to account for thermal effects. If measurements are collected once per day, thermal effects cannot be understood fully and quantified accurately.

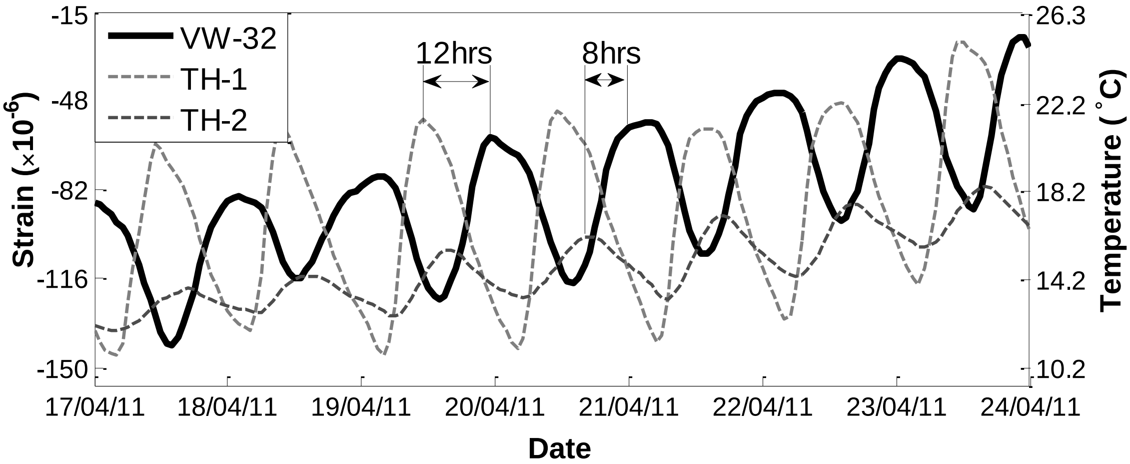

Fig. 15 Time histories of temperature and strain collected over one week

2.3.3 Measurement interpretation

Given there are 150 strain sensors and 8 thermocouples, visual interpretation of the raw data is extremely difficult. Computational approaches are therefore employed for integrated analysis of measurements. First sensors are clustered into groups according to their spatial locations. Then a method of regression-based thermal response prediction (Kromanis and Kripakaran 2014) is used to create regression models that take temperature measurements as input and predict response. These models are then employed to compute prediction error (PE) series, which are simply the difference between predicted and measured response. The PE series is visualized and further analysed using anomaly detection techniques to identify anomaly events such as sensors turning faulty.

For illustrative purposes PE VW 15, PE VW-23, PE VW-26 and PE VW-36 are shown in Fig. 16. PE VW-15 and PE VW-23 remain stationary and no obvious shifts or drifts in their signals are observed during the monitoring period. However, PE VW-26 and PE VW-36 drift albeit gradually indicating drifting sensors. This is confirmed when PE VW-26 and PE VW-36 are analysed using anomaly detection techniques. In fact analysis of all measurement clusters that include sensors VW-26 and VW-36 indicate presence of anomaly events (Kromanis 2015).

2.3.4 Conclusions from the monitoring

Real-life strain signals are likely to be noisy and have outliers. Therefore, effective pre-processing of data is crucial to data interpretation. For the River Trent Bridge, thermal inertia effects create complex relationships between temperature and strain measurements. Capturing these relationships requires an appropriately high measurement collection frequency. Computing techniques can help in clustering measurement sets that upon further examination using anomaly detection techniques can pick up anomalous events, which in this case turn out to be faulty sensors.

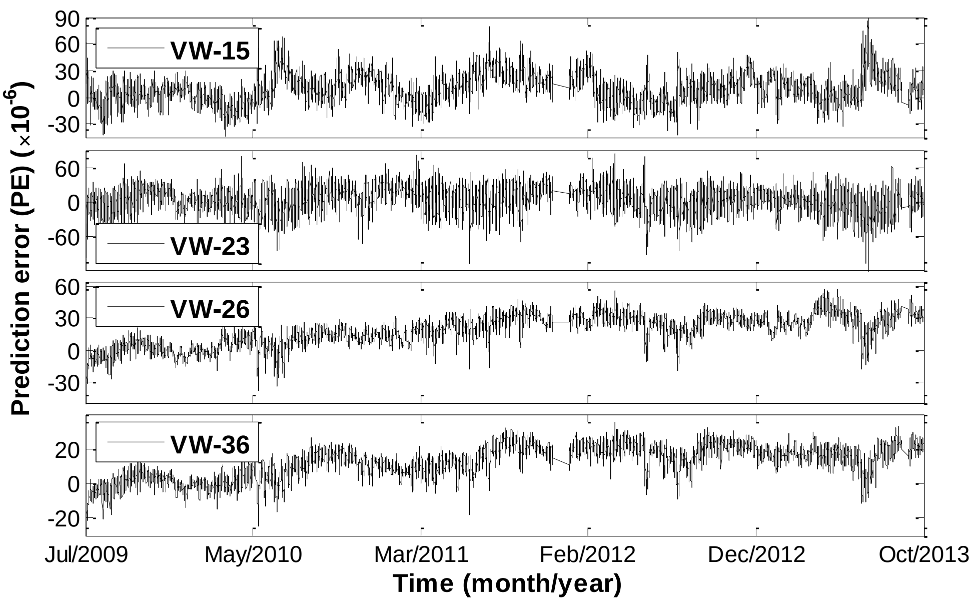

Fig. 16 Prediction error signals for sensors VW-15, VW-23, VW-26 and VW-36

3. Conclusions

The three case studies exemplify present UK concerns and approaches to highway bridge monitoring. They show that the concerns that drove the studies by Transport (and Road) Research Laboratory half a century ago, i.e. thermal expansion and behaviour of movement joints have not disappeared, if anything they are more pressing as structures age and demands increase. The technology for monitoring has also improved, moving from analog recording to remotely accessed digital acquisition. The ‘revolutionary’ technologies of fibre optics (for strain gauging) and low power and low cost wireless data transmission are slowly moving out of academic applications towards commercial application in bridge monitoring and the authors expect to see these supplemented by non-contacting measurement technologies.

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