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Ambient vibration re-testing of the Humber Bridge

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ABSTRACT: An ambient vibration survey of the Humber Bridge was carried out in July 2008 by a combined team from the UK, Portugal and Hong Kong. The exercise had several purposes that included the evaluation of new technology for instrumentation and system identification and the generation of an experimental data set of modal properties to be used for validation and updating of finite element models for scenario simulation. The exercise was conducted as part of a project aimed at developing online diagnosis capabilities for three landmark European suspension bridges. The paper describes the equipment and software technology used for the exercise and presents some of the results.

1 BACKGROUND

Among all types of civil infrastructure, Structural Health Monitoring (SHM) applications predominate in bridges, and there is a long of their application in short-term monitoring and assessment for management of short span bridges in Canada, USA, continental Europe and the Far East, and for long term monitoring of a number of landmark structures such as the Rion-Antirion, Confederation and Tsing Ma bridges.

For long span suspension bridges (LSSBs), structural monitoring exercises have been primarily concerned with effect of earthquakes and wind. Despite new computational capabilities in aero-elasticity, there are still surprises in real life operational conditions and the strategic importance and capital investment in these structures justifies the expense of monitoring systems to evaluate effects like vortex shedding and their mitigation measures.

Among these exercises, a joint study[1] of the Humber Bridge in 1990/1991 by teams from Italy and the UK was an early example demonstrating the potential for tracking and characterizing operational performance, although such data were not interpreted in real time (i.e. were post-processed). Many LSSB monitoring projects[2] have followed the Humber exercise but there is little public indication of their capability for structural diagnosis, apart from some academic post-processing exercises and certainly no clear evidence of real-time capability. Unless successes are being hidden, the information that could be obtained by timely and efficient data mining procedures is largely lost.

What would operators want to know about performance of their LSSBs? Among the greatest current concerns are apparently fatigue, condition of bearings and hangers and deterioration of the main cables. While some of these concerns can only be addressed by local detection procedures our experience is that global monitoring that includes both static and dynamic data can provide a wealth of information capable of addressing many of the key concerns.

This is the background to a project funded by EPSRC that aims to provide real-time online structural diagnosis capability in three major suspension bridges: Humber Bridge and Tamar

Bridge in the UK and Fatih Bridge in Turkey. In each case, a combination of static and dynamic instrumentation will be used to record loading and performance, which will be interpreted by system identification, data mining and computer simulations. The computer simulations, based on numerical or FE models, will be used to provide target practice for pattern recognition systems, and will be validated by field testing of the prototypes.

2 THE HUMBER BRIDGE AND PREVIOUS TESTING

The Humber Bridge (Figure 1), which was opened in July 1981, has a main span of 1410m with side spans of 280m and 530m. After Severn and Bosporus Bridges, Humber is the third bridge designed by Freeman Fox and Partners having aerodynamic steel box decks and inclined hangers. The spans comprise 124 units of 18.1m long 4.5m deep 140 prefabricated sections 28.5m wide including two 3m walkways. The top of the box section constitutes an orthotropic plate on which mastic asphalt surfacing is laid, and the sections have four internal bulkheads. At the end of each span is a pair of A-frame rocker bearing that provide restraint in three degrees of freedom. The slip-formed reinforced concrete towers rise 155.5 above the caisson foundations and carry the two main cables which have a sag of 115.5m. These cables each have sectional area of 0.29m^2 and consist of almost 15,000 5mm 1.54kN/mm^2 UTS wires grouped in strands.

The geometry of the main cable, along with the arrangement of the hangers and bearings are key influences on the dynamic properties of the bridge, as was discovered in previous testing in July 1985[3].The testing was motivated by a requirement to validate FE procedures[4] for suspensions bridges using the classical SAPIV software developed at Berkeley. The 1985 testing used only three accelerometers, several km of cables, an analog tape recorder and a two-channel spectrum analyser. By replaying the acceleration signals through the analyser, using a form of the ‘peak-picking’ procedure to recover frequencies, damping ratios and modulus ratios then laboriously piecing together the mode shape components over a period of six months, it was possible to identify over 100 vibration modes of the main span, side spans and towers up to a frequency of 2Hz. The same modelling technology was then applied to the two Turkish suspension bridges to study the effects of differential support excitation during earthquakes.

The 1985 testing showed that modes existed in which the main span oscillated independently of the side spans, and some modes with only sidespan movement. Because of the signal quality degradation resulting from measurement process, the modal participation among the spans appeared to be uncertain. Moreover, with only three accelerometers it was impossible to resolve torsional mode shapes from vertical mode shapes when they occurred at close frequencies. After, 23 years the original signals and resulting digital mode shapes were no longer available, just the values in published papers and reports. Because of the quality uncertainties and lack of digital data for the EPSRC project, a retest of the bridge was necessary.

Figure 1 Views of Humber Bridge from North and South

3 PLANNING AND TEST PROGRAM

The objective of the test was a set of vibration modes with enough resolution to capture dynamic effects that could be affected by structural changes of interest to the bridge operators, for example the bearings were believed to have partially seized. Without student resources for lengthy post-processing of data a different strategy was required for the system identification, making use of autonomous recorders with precise timing of GPS-synchronised clocks and a range of operational modal analysis (OMA) procedures. The system for using autonomous recorders pioneered by researchers at FEUP in Portugal [5,6] was adopted and a team from FEUP brought their recorders and assisted in the testing and post-processing.

Between FEUP and University of Sheffield ten GEOSIG recorders were available. These recorders used either internal force balance accelerometers, external Guralp CMG5 accelerometers of a triaxial arrangement of QA750 accelerometers (Figure 2). With up to five days of measurement available with a maximum of 10 hours per day due to recorder batteries, an optimal plan was formulated that involved separate recordings (setups) to cover 76 positions. Sensor locations for one of the setup is shown in Figure 3, in each of these two pairs of triaxial recorders would be maintained at two permanent reference locations, leaving the remaining three pairs to rove either deck or in East/West tower pylons. There being an average 5km round trip to visit the opposite side of the bridge, teams worked independently on each side of the deck and communicating via text messages. Each measurement generated one hour of 30-channel (10 in each direction) acceleration records, in four 15-minute segments. The entire day of measurements was pre-programmed into each recorder, leaving 10-minute periods between measurements to move the six rovers. Several cross-calibration measurements were made to test the synchronisation and relative calibrations of the recorders by positioning them together. For sidespan measurements an extra pair of recorders was kept as a sidespan reference and on the final day, a single pair of recorders was kept in the main span with a pair of recorder left on the top of each tower (one on each pylon) and the remaining pairs roved in the tower.

Figure 2 Recorders, sensors and GPS antennae during cross-calibration measurement

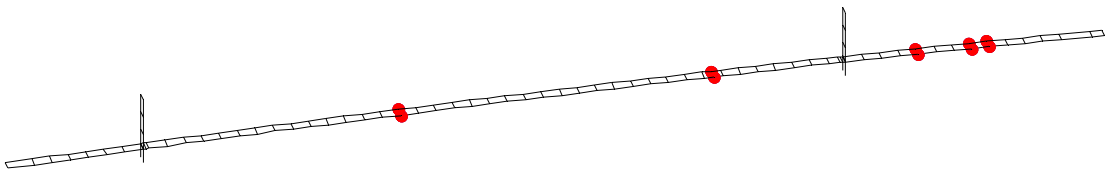

Figure 3 Sensor locations for setup24: sidespan measurement

Bicycles were used to speed the moving process and with constant pedestrian traffic on the walkways where the recorders were located the recorders had to be guarded constantly. Heavy rain occurred on the fourth day of the test but the recorders (if not the researchers) worked perfectly throughout. Measurements on the tower tested the limits of the recorders: the pier movements were expected to be very small and at the resolution limits of the sensors, also

satellite visibility was reduced below the deck and was not possible inside the towers. Hence measurements inside the tower were a test of the synchronisation memory of the recorders.

For the 1985 measurements, accelerometers were kept inside the deck. While avoiding problems with weather, this made movement of the sensors difficult. In the 2008 test, an independent recording using NI-USB 9239 acquisition unit and four QA750 accelerometers was left running inside the deck near the centre span for the duration of the measurements.

4 SYSTEM IDENTIFICATION PROCEDURES

A range of OMA procedures including ERA[7], SSI[8] and FDD[9] were used to estimate modal properties, with the major challenge being the merging of data from 76×6 degrees of freedom (two triaxial recordings from each side of the bridge, excluding duplications).

For the GEOSIG data the first step in the procedure was a gathering of up to four segments from each of the ten separate recorders into single data file for each of the 28 measurements. Separate approaches were then used by the FEUP and Sheffield teams for the preliminary initial results reported here.

The FEUP preliminary analysis procedure used a combination of MATLAB scripts and spreadsheet macros to assemble auto-and cross spectra from which modal ordinates could be identified, with frequencies estimated from the auto-spectra.

The Sheffield procedure used MODAL[10] system identification software to first create cross-power matrices for each setup. 'DOF' text files were created for each setup to map channels to degrees of freedom in the bridge geometry. The whole set of cross-powers was merged into one large matrix with as many columns as reference channels and as many rows as measurement channels. The impulse response functions generated by the inverse FT of this matrix were used to identify modes using the eigensystem realisation algorithm (ERA). Possible variables in this NExT/ERA[11] procedure include the number of averages used in the cross-power generation (and hence frequency/time resolution), the shape of the Hankel matrices (number of lags used) and the number of poles chosen in the identification. Some studies of these were carried out by the Hong Kong researchers, and the optimal mix used in the results presented.

Due to limitations of MATLAB computer memory and the complicated nature of the three-dimensional modes, the procedure was applied to subsets of e.g. vertical or torsional only data. In these cases, 5×5 cross-power matrices were used, taking advantage of assumed symmetry or asymmetry with respect to centre line of the deck (i.e. slaving nodes). The nature of the vertical/torsional modes was first identified by analyses with 10×10 matrices and no assumption of symmetry, and the mix of lateral and torsional response in predominantly lateral or torsional response was checked by slaving lateral DOFs (setting them opposite either side). Participation of the towers was studied using data from the final day of measurement.

Data from the recorder kept inside the deck were used to track variability of modal parameters and response levels throughout the measurements using stochastic subspace identification (SSI) procedures developed and applied successfully on Tamar Bridge.

5 RESULTS

Figure 4 shows auto power spectra of vertical and torsional acceleration at location h21. There is a high modal density up to approximately 5Hz after which response levels drop and modes are indistinct. The 1st torsional mode appears to occur within a few mHz of 6th vertical mode, and this has consequences on the ability to discriminate the two modes.

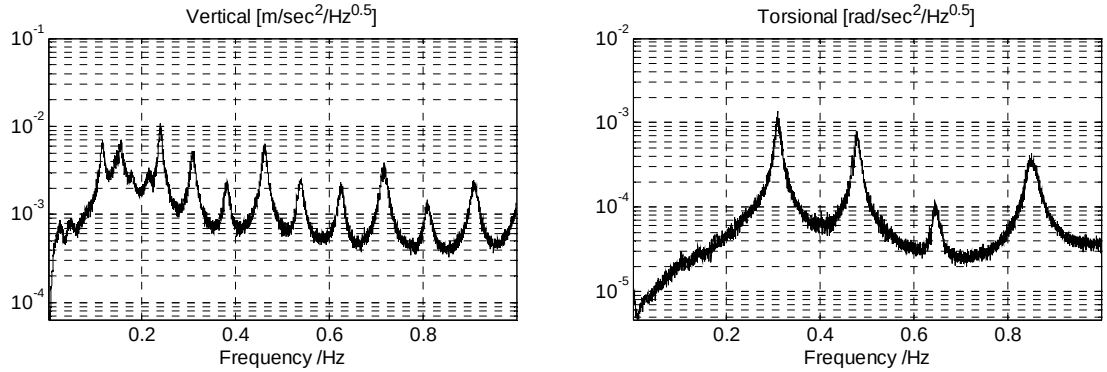

Figure 4 Vertical and torsional acceleration response auto spectral densities

Figure 5 shows the SSI stabilisation plot for the four reference locations. Two distinct but close modes are visible around 0.31Hz. Figure 6 shows the first 12 modes identified from the subset of 68 pairs of vertical deck responses. Three of the first four modes involve significant sidespan motion and the stronger of the two 0.31Hz shown has both vertical and torsional character

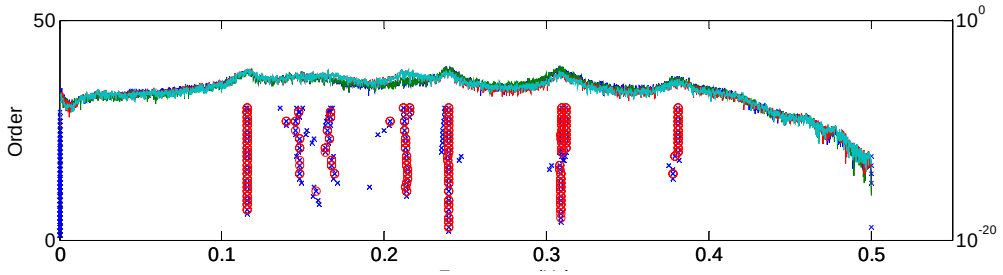

Figure 5 SSI stabilization plot from four vertical (reference) channels

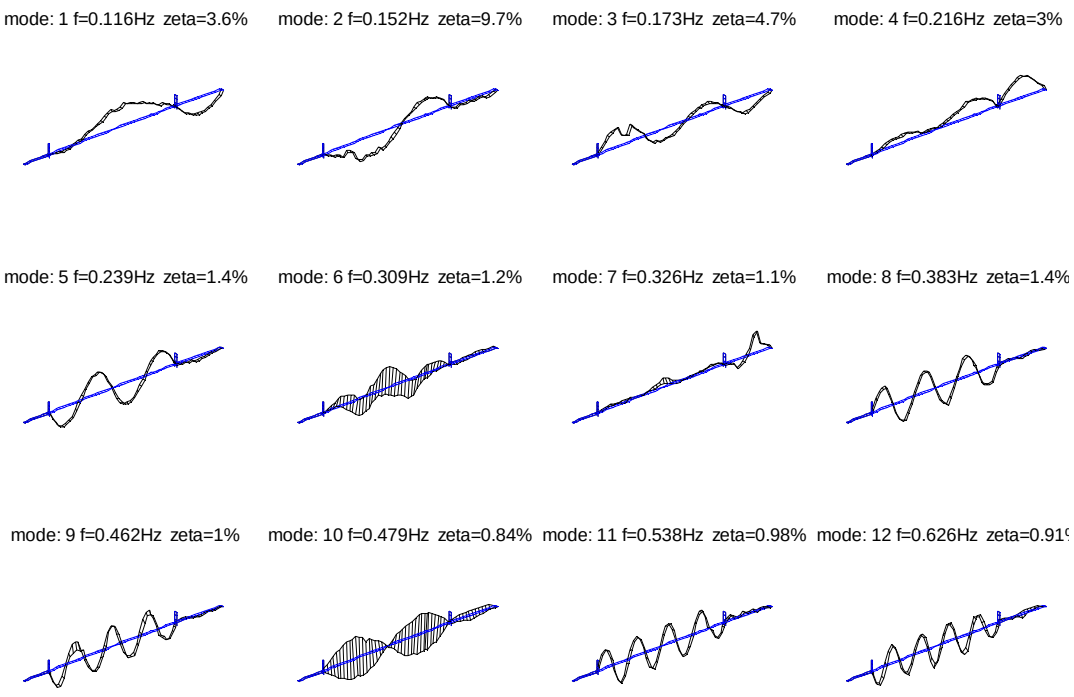

Figure 6 Relatively convincing vertical and torsional modes straight out of ERA

Figure 7 shows the first four torsional modes, including one in the longer sidespan, obtained from differencing the 68 pairs. The dual of the 0.309Hz mode shown (first symmetric torsonal mode), is the third symmetric vertical mode. Figure 8 shows identified lateral modes; these include modes that are predominantly torsional. The first mode is the most interesting dynamically due to its apparent high damping and response to wind. There are several other weak modes including one at approximately 0.23Hz.

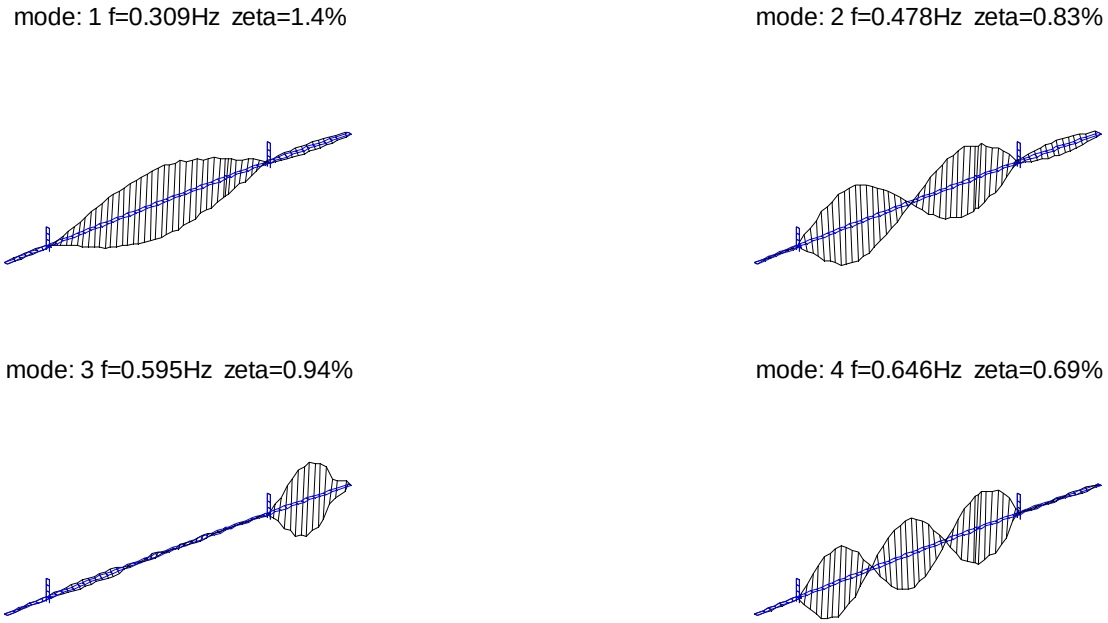

Figure 7 Torsional modes from difference of 68 DOF pairs

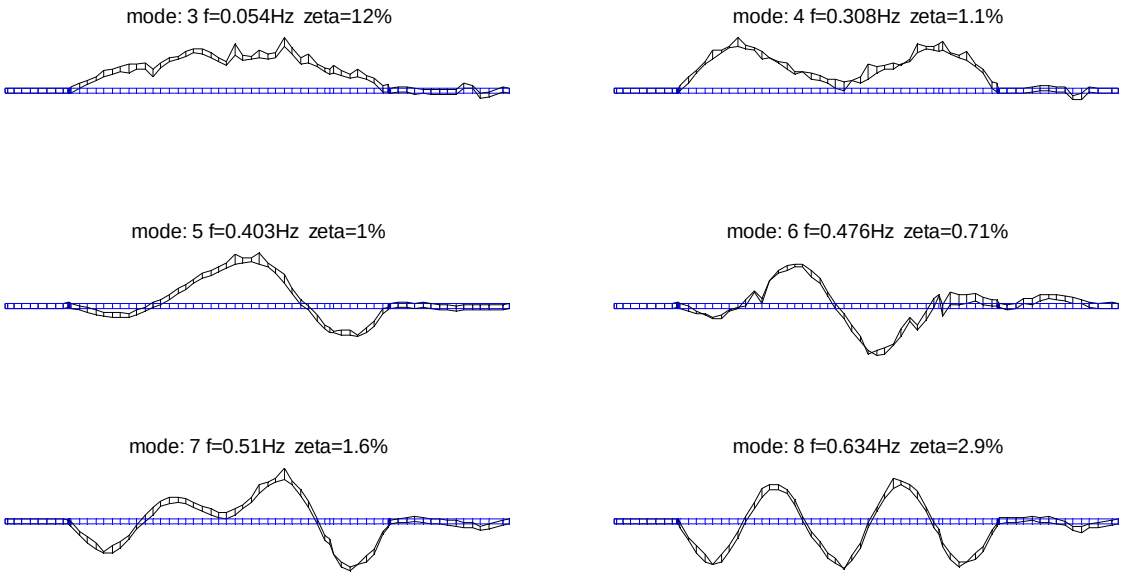

Figure 8 Relatively convincing lateral modes straight out of ERA

Finally, Figure 9 shows tower modes identified from pairs or sensors at the top of each tower and at h21. Modes 1 and 4 echo the main span/sidespan link of Figure 6, otherwise there is weak twisting of the tower in torsional modes. 1985 data showed many tower modes mimicking a propped cantilever i.e. with movement at portal levels that could not be measured in 2008.

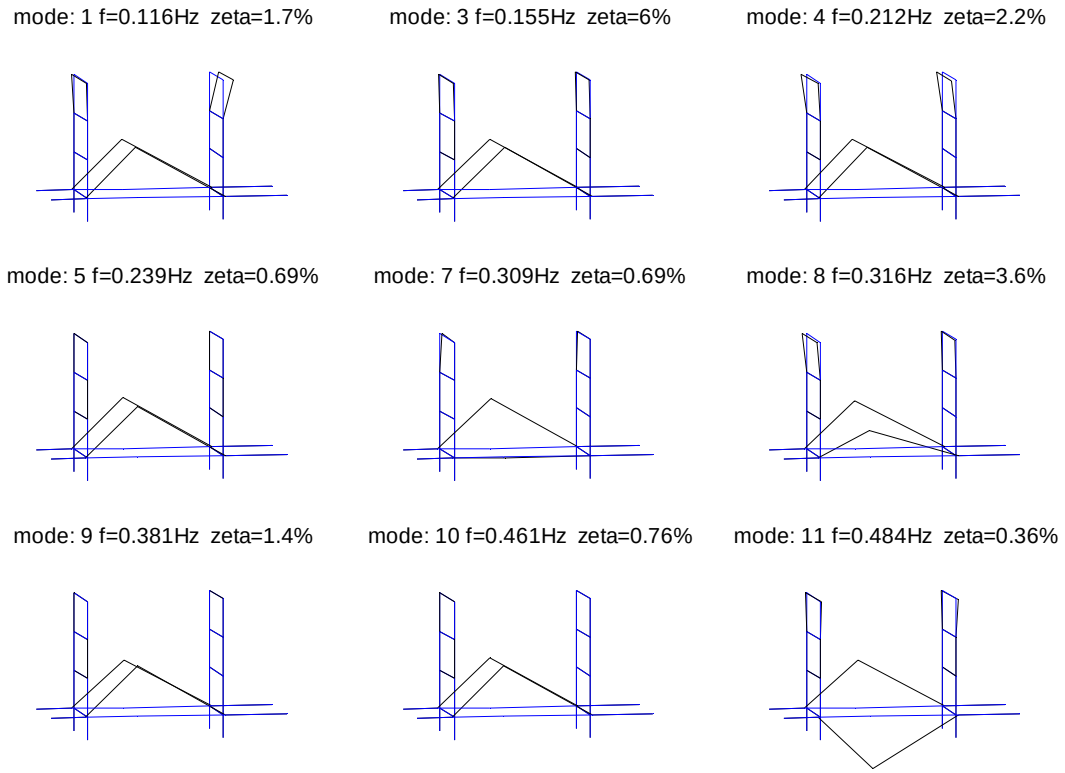

Figure 9 Tower modes from three accelerometer pairs.

6 DISCUSSION AND COMPARISON WITH 1985 DATA

Table 1 compares identification of vertical modes between the 1985 University of Bristol (UoB) exercise and the 2008 exercise by University of Sheffield (UoS) and FEUP teams. 'V' and 'T' are vertical and torsional mode types, with 'A' and 'S' denoting symmetry. In fact because the bridge is not symmetric in the sidespans, modes that engage sidespans do not have simple symmetry so A/S mean having odd/even numbers of nodal points between the towers. Even this system breaks down for the first and fourth modes: the first mode is effectively the first symmetric mode. Frequencies have generally reduced, by less than 1% and the difference between mode 6 and 7 frequencies remains minute.

The long term monitoring exercise[1] showed that frequencies are affected by wind speed in the second, third and fourth significant figures for the first lateral, torsional and vertical modes respectively. The four days of monitoring in 2008 show diurnal variations approaching 3% for the mode at 0.31Hz, that could be due to a combination of wind and thermal effects.

In the 2008 exercise no serious attempt was made to identify damping ratios; clearly the identified values given in the mode shape plots here are quite large. Following the 1985 study there was a published discussion[12] about the reliability of the published values and a follow-up study [13] on accuracy of damping estimates for suspension bridges. The identification method used in 1985 was inherently biased (positively) and long records –with stationarity assumed so that the exercise could be carried out – showed more reasonable values. Reliable identification of damping will be a key to the long term monitoring as damping is known to be affected by aero-elastic effects. Maximum average wind speed recorded was 9m/sec: from the damping study this would cause noticeable damping changes increase in first vertical mode and smaller effects on the first torsional mode.

Table 1: Comparison of modes identified from 1985 and 2008 tests

#	UoB/Hz	type	symmetry	Hessle	Barton	UoS/Hz	FEUP/Hz
1	0.1165	V	S	y	y	0.1155	0.115
2	0.1536	V	A	y	y	0.153	0.156
3	0.1772	V	S	y	y	0.175	0.176
4	0.218	V	S	y	y	0.215	0.217
5	0.2396	V	A	n	n	0.2394	0.239
6	0.31	V	S	y	n	0.3084	0.308
7	**0.3113**	**T**	**S**	**n**	**y**	**0.3091**	**0.309**
8	0.3828	V	A	n	n	0.3809	0.383
9	0.4636	V	S	n	n	0.4617	0.461
10	**0.4817**	**T**	**A**	**n**	**y**	**0.479**	**0.479**
11	0.5402	V	A	n	y	0.5385	0.537
12	0.6273	V	S	n	n	0.625	0.625
13	**0.6497**	**T**	**S**	**n**	**y**	**0.6464**	**0.645**
14	0.7196	V	A	y	n	0.7166	0.715
15	0.8144	V	S	n	n	0.81	0.808
16	**0.858**	**T**	**A**	**n**	**n**	**0.8514**	**0.85**
17	0.9129	V	A	n	n	0.9095	0.908

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