The Missing Link in Bridge Design

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Abstract

We measured the response of the Sydney Harbour Bridge (SHB) to ambient vibration, and determined its natural frequencies and damping, the first time this has been done to our knowledge. We then compared our measurements of the natural frequency and several higher modes with those computed recently using a Finite Element model of the Bridge, and there is unexpected disagreement. We recommend that engineers working on the design of important structures take the extra step of measuring the response of the completed structure and comparing that with their model, then changing the model until it mirrors the measurement. The SHB deck was replaced between our first and second SHB monitoring sessions, and this may have contributed to a small but significant change to the measured modal frequencies. We recommend that iconic structures like the SHB be treated like other important structures, dams and nuclear reactors, and be monitored in real time to compare any short-term or long-term changes in the modal frequency or damping characteristics of the structure.

Keywords: seismic, hazard, bridge, modal vibration, frequency, finite element model

Introduction

Bridge design developed rapidly with the industrial revolution as new materials (steel in particular) became available to replace timber and masonry, and structural forms evolved, such as the box girder, to increase strength without increasing weight. Failures in the 19th and 20th

centuries drove improvements in design and construction. The computer became a significant factor in design during the 1960s, allowing modeling techniques such as finite element modeling to be utilised.

Maintenance over the lifetime of a bridge can be more expensive than the original construction costs, but rarely does it involve dynamic testing of the bridge. Long term degradation of the bridge due to fatigue, or component rusting, or foundation failure, may be very difficult to identify in a quick visual inspection. Unexpected loads such as earthquakes and wind have caused modern bridge failures, such as wind causing the 1940 Tacoma Narrows bridge failure, and earthquakes in the 1971 and 1989 Californian highway bridge collapses, and toppling of the 1990 Kobe elevated railway in Japan. At least three major bridge failures have occurred in Australia since 1970[1] although none as yet attributed to earthquakes.

The Sydney Harbour Bridge is a nationally and internationally renowned iconic structure that opened in 1932 and is a busy and important portal for the city of Sydney. A picture of the bridge imbues a sense of Australia just as much as a picture of a Koala or Kangaroo. It is not just a traffic bridge. As such it would be expected to be able to accommodate large, infrequent loads, be they wind or earthquake, or 200,000 people walking across the bridge. Earthquake design was never considered in the 1920s in Australia or the UK when the bridge was being designed and constructed but since inauguration it has been shaken by several moderate, distant earthquakes; in 1961 (Robertson-Bowral), 1973 (Picton), 1989 (Newcastle) and 1994 (Ellalong), the peak intensity at the bridge being about MM5 on the modified Mercalli scale. No measurements were made of the bridge response to these earthquakes, nor are there anecdotal stories about the shaking. There are a few free-field records of the 1994 Ellalong earthquake obtained at various distances. The question has to be asked just how the bridge would have responded if one of these earthquakes had been larger, and/or closer, one magnitude larger say, and 10 km distant – not too imaginative. Factors that have to be considered in any analysis of the response and robustness of the structure are: what is its projected lifetime? what are its resonant frequencies? and what is the damping factor of its riveted steel construction?

A team of UTS and Melbourne University researchers led by the third author has undertaken dynamic modeling of the SHB structure for the NSW RMS[2] and computed a number of the vibration modes and frequencies that can then be used with simulated earthquakes to answer the performance question. Many assumptions have to be made in such an analysis and some testing of these results is warranted since the computed performance of the bridge is dependent on them.

Recent technology developments have made it possible to measure the vibration characteristics of structures under wind and traffic loading, so the first two authors have made recent

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[1] http://en.wikipedia.org/wiki/List_of_bridge_failures

[2] Roads & Maritime Services, formerly the Road Transport Authority.

measurements at the centre-point on the eastern side of the SHB decking. The comparison of computed and measured natural frequencies is discussed in the paper and suggestions made as to why theory and measurement do not exactly agree. We discuss the need for further measurements and permanent monitoring.

Method

With the recent enormous advances in personal computing power, low cost digital seismographs are now increasingly available to private individuals. These seismographs range from basic vibration monitoring via the 3-axis accelerometer sensor incorporated within most Smart Phones, to sophisticated sensitive multi-channel seismograph systems that only a few years ago would have been exclusively the domain of major consulting companies. As with personal computers, this seismograph equipment has become relatively cheap and compact, and a briefcase sized battery operated seismograph station, incorporating precision GPS timing, can be assembled for a few hundred dollars.

Such a briefcase system was assembled by the first author, and it has been used to measure the deck motion of large road bridges, and other structures. This system may be conveniently carried to a place of interest and set collecting data within minutes. Seismic sensors used for detecting and localising distant earthquakes are normally very sensitive, whereas the natural and induced motion of man-made structures is normally very large by the standards of seismology. Hence for structural motion monitoring, one may utilise small inexpensive ‘MEMS’ solid-state accelerometers[3], of the same kind that exist in most Smart Phones. Costing only a few dollars apiece, such sensors are mechanically rugged and have a suitable operating range for measuring structural motion. Their biggest performance shortfall is high internal noise, for inexpensive ‘consumer grade’ MEMS sensors at least. Such sensors are frequently used for sensing the orientation of objects in the Earth’s gravitational field, such as the STMicroelectronics LIS331DLH[4] sensor used in the popular iPhone 4. The Analog Devices ADXL335[5] sensor has very similar performance but with a slightly lower internal noise floor.

The level of this noise floor is frequency dependent (e.g. 218μg/√Hz for the LIS331DLH versus 150μg/√Hz for the ADXL335, where 100μg≈1mm/s^2), and on first glance this may seem rather too high for measuring accelerations likely to exist within an undisturbed large structure, such as

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[3] Reviewed by Nicholas Haritos in ‘Low Cost Accelerometer Sensors – Applications and Challenges’, published in the proceedings of the 2009 AEES Conference.

[4] http://www.st.com/internet/com/TECHNICAL_RESOURCES/TECHNICAL_LITERATURE/DATASHEET/CD00213470.pdf

[5] http://www.sparkfun.com/datasheets/Components/SMD/adxl335.pdf

a tower or road bridge. But the important point to note is that these sensors' internal noise is random and incoherent, whereas actual structural accelerations, such as modal vibrations, are normally coherent and narrow-band. Hence if a long seismograph record is spectrally analysed using a Fast Fourier Transform (FFT), coherent signal amplitudes can be measured at levels far below the sensor's noise floor. Thus a MEMS sensor may not be able to detect slow semi-random accelerations, such as near-static structural deflection from steady-wind loading, but it can be very sensitive to the modal vibration that such loading induces, provided the data record length is long enough.

When one is measuring road bridge structural motion with the sensor placed on the road deck of a bridge, the environmental [incoherent] noise recorded may be considerable, and often significantly exceeds the sensor's internal noise. Using an ADXL335 sensor, and by trial and error, the authors determined that a data record length of 60 minutes generally delivered an acceptable FFT spectral noise floor, one that was low enough to show the [often] weak fundamental modes of vibration. Although the level of a noise floor presented within a FFT spectrum will be somewhat dependant on the environmental noise at the measuring location, we found that a spectral noise floor of 1-2mm/s^2 (at ≈1Hz) was typically achievable even when measuring the roadway of a heavily trafficked road bridge.

The ADXL335 has a nominal sensitivity of 300mV/g, which is presumed to hold true even well below the sensor's internal noise floor. To check this, and the sensitivity of the sensor across a range of frequencies, a test rig was constructed to gently sinusoidally tilt the sensor by 1-2 degrees of angle, across the frequency range of 0.1-2Hz. This test rig incorporated a stepper motor, whose speed could be set precisely from a voltage signal generator. Thus by knowing the amplitude of the induced acceleration of the sensor, the amplitude of the resultant FFT spectral peaks could be calibrated, and the sensor's sensitivity verified to be satisfactorily linear across 0.1-2Hz.

The portable seismograph system used by the authors incorporates a 16-bit 3-channel digitiser[6], sampling at 200sps, connected to a small fully solid-state laptop computer. The logger is capable of sampling at 500sps by swapping-in a suitable anti-aliasing filter/amplifier. This system has been used extensively to measure the motion of bridges and structures around Sydney, and it also saw brief service measuring earthquake aftershocks in Christchurch, New Zealand, and mine blasts in the Hunter Valley.

[6] http://psn.quake.net/serialatod.html

Results

The portable seismograph system was taken to Sydney and operated at the centrepoint of the Sydney Harbour Bridge (SHB), on the pedestrian walkway that runs along the eastern side of the bridge. Two separate observations were made, the first was conducted on the 28th October 2011 (30 minutes with 500sps sampling) when there was very light wind, and the second on 2nd July 2012 (60 minutes with 200sps sampling) with moderate wind. The sensor's three axes were aligned with the SHB's longitudinal (L), transverse (T) and vertical (V) axes. The centre of the SHB is a noisy place, and vibration from nearby traffic may be easily felt through one's feet. The SHB traffic speed limit during observations was 70km/h, which for normal sized automobiles implies a wheel rotation speed of around 10Hz, and indeed there was a peak spectral component of road deck vibration at around this frequency, very probably induced by car traffic. Truck tyres are typically 20-40% larger in diameter than car tyres and hence truck tyres rotate at around 7-8Hz at 70km/hr, which is consistent with increased spectral noise observed in this band.

Figure 1 shows an example of acceleration spectra recorded at the SHB, in this case a comparison between the recording runs on different days, showing the SHB transverse motion spectra between 0.1-10Hz. The two plots are clearly very similar and clearly show the lowest frequency transverse modal vibration. A simple approximate formula for the fundamental (first mode) frequency of a simply supported bridge is f_0(Hz) = 100/L where span length L is in metres. Given the SHB span length of around 500 metres, one would anticipate the lowest modal frequency at around 0.2Hz, and indeed the lowest spectral peak shown in Figure 1 occurs at about 0.28Hz. The amplitudes shown in Figure 1 are normalised to the peak amplitude, which occurs at around 10Hz. The actual peak amplitude for f_0(transverse) is ≈0.8mm/s^2. Similar plots may be presented for all the L, V & T axes of the SHB, as shown in Figure 2, but this time with a frequency range of 0.1-100Hz. These plots again demonstrate that most of the spectral energy is concentrated around 10Hz, and that the f_0 modes of vibration for longitudinal, transverse and vertical motion are around 1.4, 0.28 and 0.91Hz respectively.

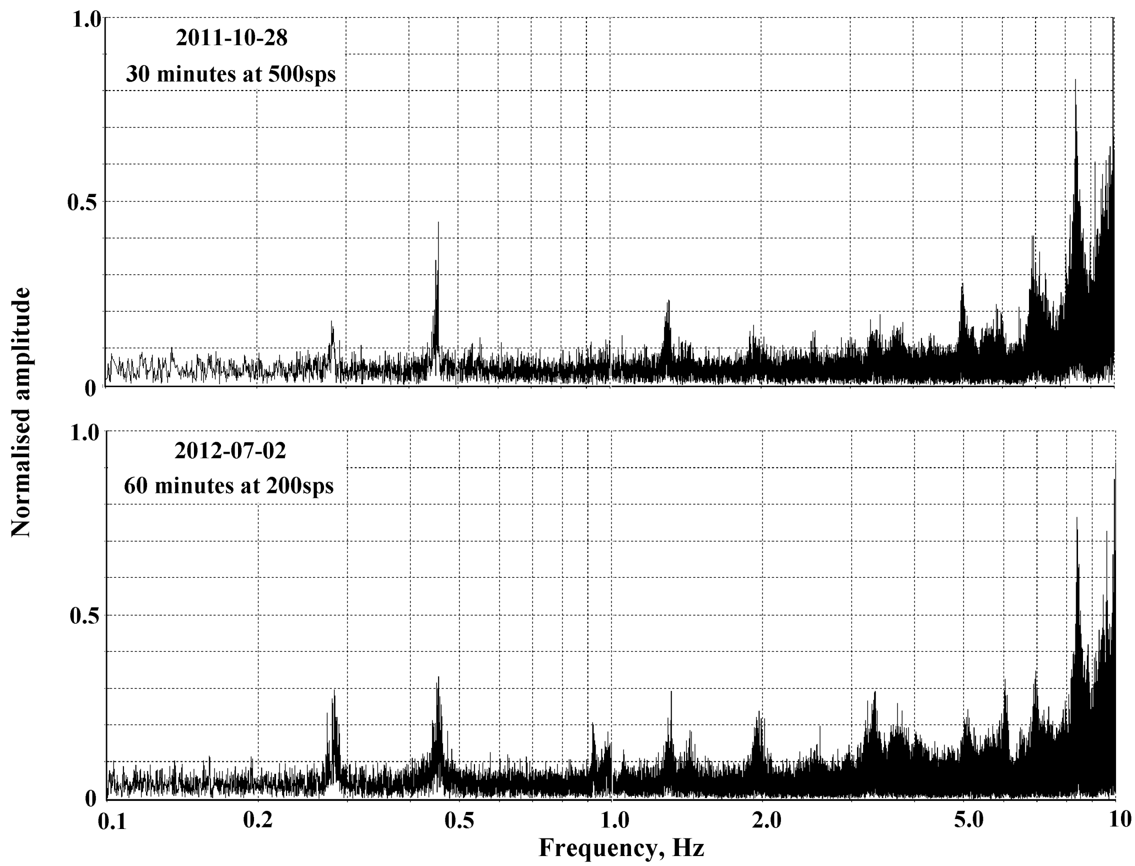

Figure 1 - Amplitude spectra of SHB transverse acceleration, from data recorded on different days, with different wind conditions, for different durations and with different sampling rates. Amplitudes are normalised to the peak amplitude, which occurs at around 10Hz.

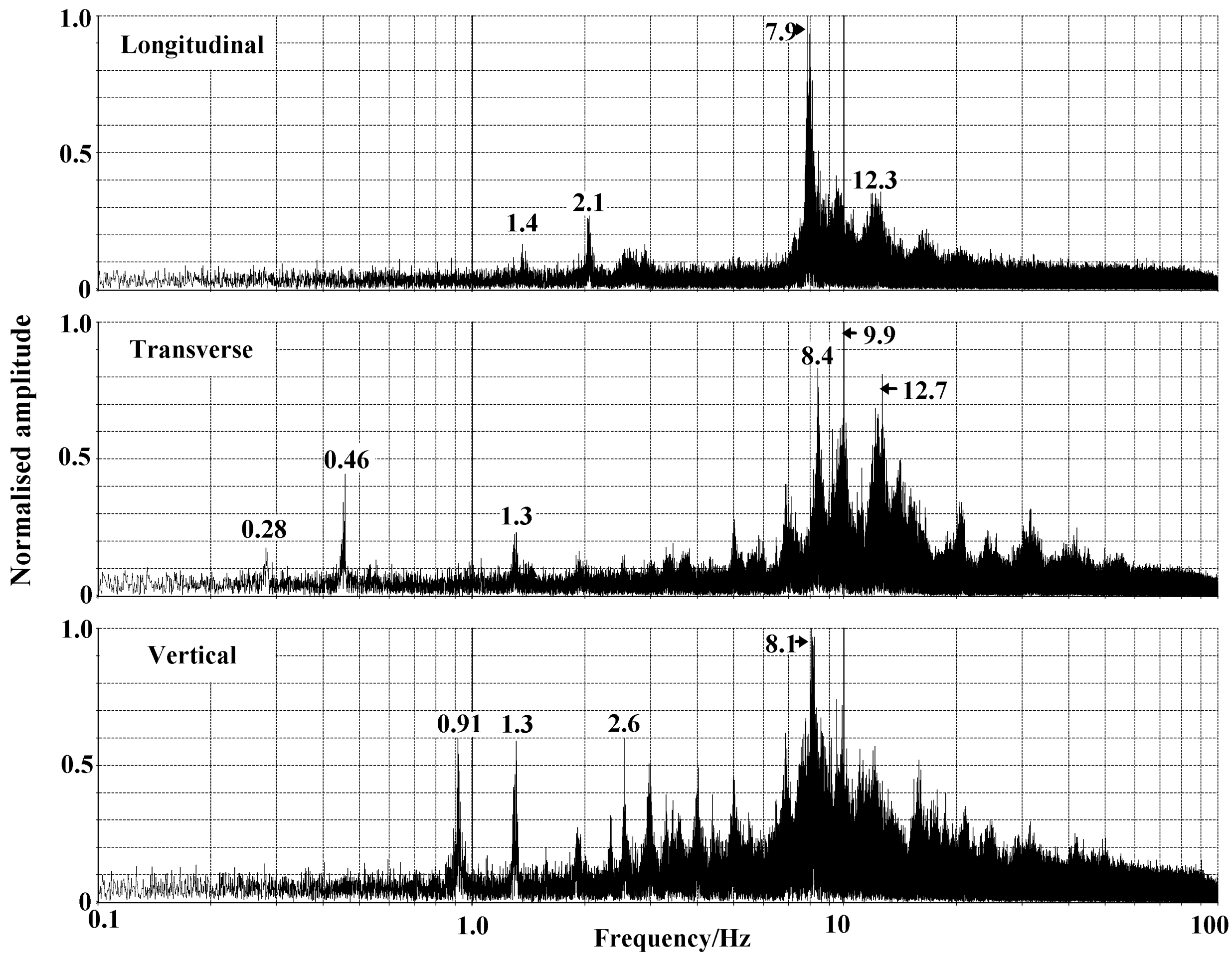

Figure 2 - Amplitude spectra of SHB acceleration, for longitudinal, transverse and vertical motions. Amplitudes are normalised to the peak amplitude, which occurs between 8-10Hz. Noteworthy spectral lines are annotated with their centre frequency.

One of this paper's authors (Samali) oversaw the development of a comprehensive Finite Element Model (FEM) for the SHB, and he supplied the results of the first few transverse modes. Two FEM computer programs were employed for these calculations, namely MicroStran[7] and LS-DYNA[8]. The predictions are presented in Table 1.

The predictions from the two FEM computer programs agree with each other to better than 1%, and these figures were overplotted onto a section of the SHB transverse motion spectra, presented in Figure 4. If the lowest peaks appearing in the spectrum at 0.28Hz and 0.46Hz represent the two lowest modes, then this indicates the model frequency predictions are around 10% and 24% too low. If the spectral peak at 0.46Hz is in fact Mode 3 of the model, then this

[7] www.microstran.com.au
[8] www.lstc.com/products/ls-dyna

prediction is about 9% too high. Two of the predicted modes are absent in the recorded spectral plot, but they may be relatively weakly excited and thus beneath the noise floor of the observational data, or the centre point of the bridge is at a node for these modes.

Unfortunately, predictions of the vertical and longitudinal motion were not obtained for comparison in this paper.

	Frequency, Hz		
	MicroStran	LS-DYNA	Measured
Mode 1	0.2611	0.2598	0.282 (±0.005)
Mode 2	0.3530	0.3512	0.458 (Mode 3, 4?)
Mode 3	0.5027	0.5001	
Mode 4	0.5420	0.5405	

Table 1 - Comparison of the computed modal natural frequencies, SHB transverse motion.

We have also estimated the damping of the bridge as a percentage of critical damping, a parameter that is important for estimating elastic spectral response amplitudes under earthquake excitation. This could be done by giving the bridge an impulsive displacement and measuring the relative amplitude of the restoration swing. We used the logistically simpler method of analysing FFT spectra, and measuring spectral-line width.

Figure 3 presents a frequency-expanded plot about the fundamental natural frequency of the transverse component of the bridge response, sampled on the 2nd July, 2012. From this we can estimate the damping factor 'Q' which is defined as $f_n/\Delta f_n$ where Δf_n is the spectral-line width at 0.71× the spectral-line amplitude. The more familiar damping ratio (damping/critical damping) is defined as 1/2Q.

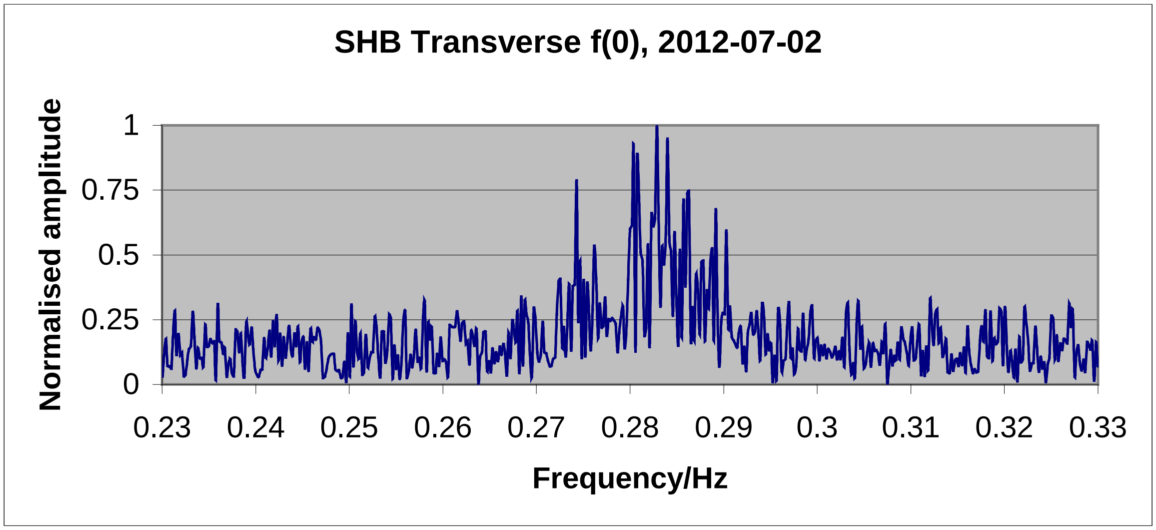

Figure 3 - Spectrum of the Sydney Harbour Bridge transverse acceleration, showing the most fundamental mode at a frequency of around 0.283Hz (period = 3.54 seconds).

The spectral peak is reasonably bell-shaped and the measurement yields Q ≈ 0.28/0.007 = 40, corresponding to a damping ratio of 0.01 (i.e. between 1% and 2% critical).

Although the SHB acceleration spectra from our 2011 & 2012 observing runs are extremely similar, we did notice one significant difference between them. Figure 5 shows the spectrum for the vertical f_0 mode with occurs at a frequency of ≈0.92Hz. It was noticed that the 2012 spectral peak was slightly higher in frequency (by a factor of ≈0.1%), a pattern that also occurred with higher order modes. Possible causes include changes in the thermal material properties of steel, traffic/train mass on the road deck, water absorption by the road deck concrete, variable stiffness of asphalt, etc. But during March 2012 there was a complete replacement of the SHB road deck surface, the first time this had occurred within the Bridge's 80 year lifetime, and there is a possibility that the Bridge's modal behavior changed slightly as a result. Had continuous monitoring of SHB motion been implemented during the 2012 road deck resurfacing, this change could have been confirmed and quantified. Moreover these data could have been included in a longitudinal record of the Bridge's modal behavior that could potentially become an extremely sensitive indicator that the Bridge is undergoing structural changes. Such changes may indicate causes that are not obvious to visual inspections, and thus potential problems may be detected earlier.

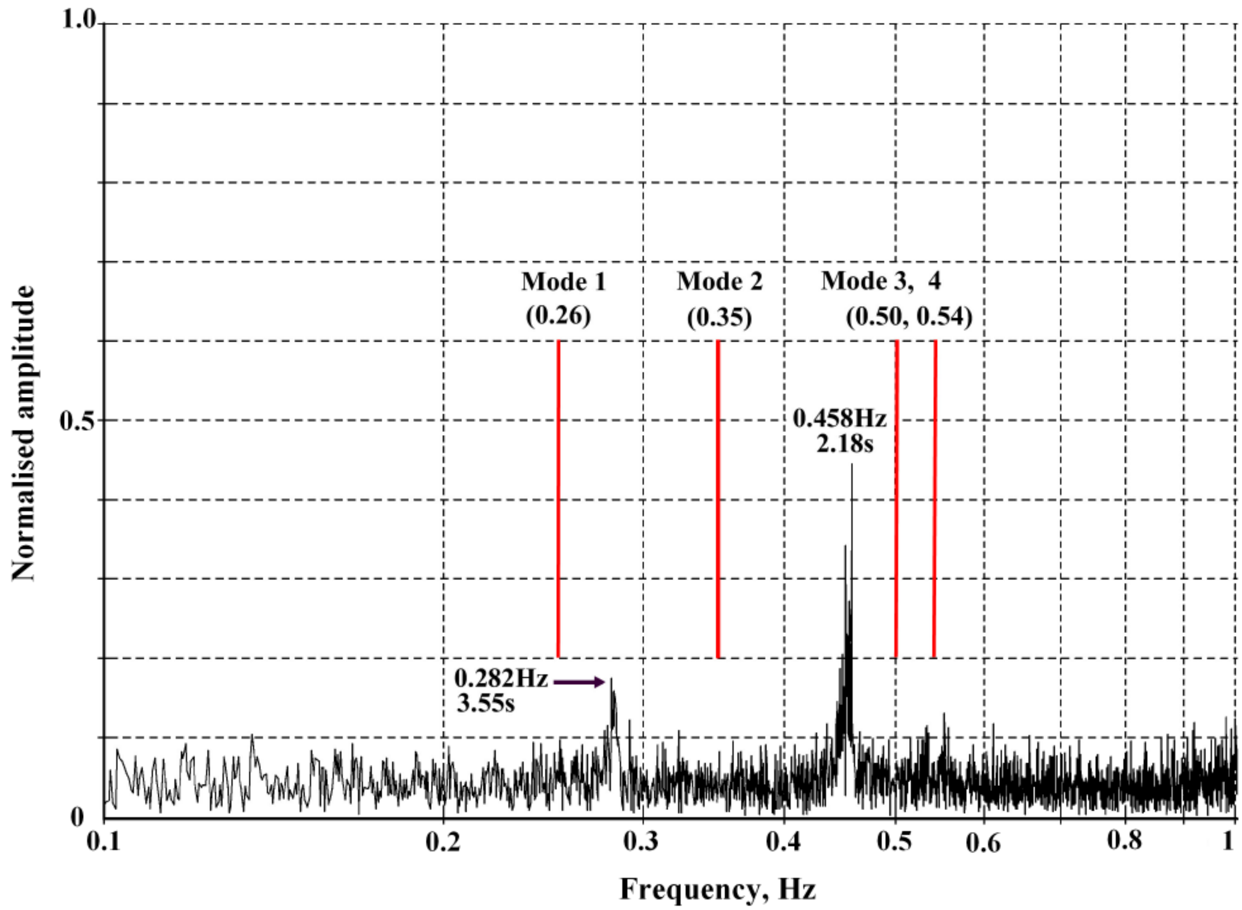

Figure 4 - Comparison of the measured and computed modal frequencies for SHB transverse acceleration. The vertical red bars indicate the first four modes computed from two FEM models (figures from which are presented in Table 1). Spectral amplitudes are normalised to the peak amplitude, which occurred at around 10Hz.

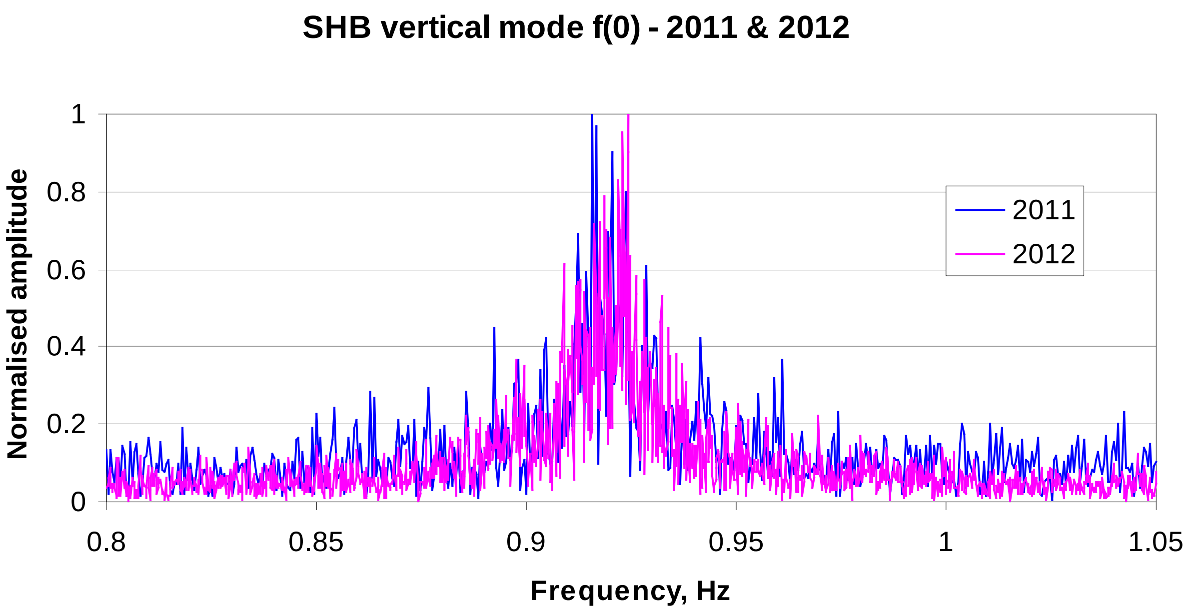

Figure 5 - Normalised spectrum of Sydney Harbour Bridge vertical acceleration showing the fundamental natural period (≈0.92Hz), measured on the 28th October 2011, and the 2nd July 2012. Higher order modes show similar frequency shifting.

Discussion

Figure 4 above demonstrates that in this limited case, the FEM model for the SHB motion needs fine tuning to bring its modal vibration predictions into line with observational data. The authors appreciate the difficulty in producing an accurate FEM based upon numerous assumptions that would need to be made concerning bridge construction materials and techniques, where no precise data may exist (e.g. the precise steel material properties and dimensions, and the behaviour of riveted plate joints under dynamic load). Thus it may be contended that a 10% error in frequency estimate of a fundamental mode of vibration, is actually impressively accurate. However, in view of the substantial effort in constructing the FEM, it is unfortunate that the final step of cross-checking the model against observational data was not undertaken by the owner. Had this been done, then the model's value in predicting its response to earthquake ground motion, would have been considerably improved.

Only months after the first European settlers arrived at Port Jackson in January 1788, they felt and reported on an earthquake, its epicentre and magnitude unknown. We do know that in the last 100 years, 17 earthquakes of at least magnitude 5 have occurred within NSW. Of these, four were in the Sydney Basin, including one of the largest at Newcastle, in December 1989, its magnitude 5.6. Simple earthquake recurrence statistics with a 'b' value of 1.0, show that a large magnitude 6 earthquake is overdue in NSW. There is no reason to suppose that a magnitude 6.3

intra-plate earthquake similar to that which devastated Christchurch, New Zealand, on the 22nd February, 2011, could not occur within the Sydney Basin, and within the immediate area of the SHB. This is the worst fear of re-insurance companies concerned with catastrophe modeling. In Australia earthquake insurance for domestic buildings is virtually free as it is included in the fire policy.

Prior to the 2010 and 2011 quakes, Christchurch was considered a relatively low earthquake hazard site, as indicated in the current New Zealand Loading Code (the seismic coefficient is similar to Meckering WA and about double that for Sydney NSW). Since there is now ample high quality earthquake strong motion data recorded from around the world, including that collected around the city of Christchurch, it is now possible to enter real-world ground motion data into FEM models to assess structural performance. One of the outcomes in Christchurch was that modern buildings survived the major September 2010 earthquake without collapse. The buildings satisfied the design criteria. However, no dynamic testing of the buildings was done to test their capacity to absorb the shaking from a subsequent earthquake, as building codes take no account of the fact that a second large earthquake may occur, in this case a smaller but closer aftershock. No one could have predicted that this combination of earthquakes would occur. It was the combination of the two earthquakes that led to so many buildings having to be demolished. The problem was assessing their capacity after the first large earthquake just by a quick visual inspection.

In reality, many of the buildings suffered such serious damage in the first large but distant earthquake in September 2010 that they were 'munted[9]' by the smaller but closer large earthquake in February 2011. There was apparently no simple way to assess this initial damage without causing quite serious additional damage when inspecting columns and beams hidden behind cladding.

The best solution is to measure the response of the structure before, during and after the earthquake, that is to instrument the structure and continuously monitor its response. A less desirable method would be to measure critical parameters such as natural period and damping before and after the earthquake.

The spectral peaks appearing in our SHB measurements tend to maximise around 10Hz. Since these peaks appear to be constant at different times, with different weather and traffic conditions, it is probable that they do represent higher order vibrational modes responding to the automobile tyre rotation speed of around 10Hz. It would be most interesting to repeat this observation during one of the occasional closures of the SHB to road traffic, to see if the 10Hz vibration amplitude decreased. Likewise it would be most interesting to log data continuously until a

[9] Kiwi slang for 'severely damaged'.

suitable event occurred with a period (measured at Sydney) in the 0.1-1Hz range, which would excite the lowest order vibrational modes of the bridge.

Conclusions

In view of the relative ease with which structural motion data may now be collected quickly and economically, and the enormous potential value of these data to significantly improve design finite element models, collection of such post-construction data should be mandatory on large construction projects within Australia. Measurements should be repeated after strong shaking events, earthquakes, tornadoes or cyclones to ensure that no permanent damage has been caused, as indicated by changes in the frequency and damping characteristics. Ideally, nationally important structures such as the SHB should be monitored in real time so that actual responses to high impact events such as earthquakes can be measured and short-term and long-term structural degradation could be documented rather than guessed at by visual inspection. This is not the expensive option that it used be, following the development of relatively cheap MEMS type accelerometers, GPS and internet connectivity.

Running real earthquake ground motion data through updated FEM models may identify potential risks that were not apparent at the design stage, and permit remedial action to be undertaken early in the life of a major structure.

Engineers and major infrastructure owners need to develop a better understanding of the nature and effects of dynamic loads on their structures and appreciate the benefits and relative simplicity of real time monitoring to achieve that end.

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