STRUCTURAL SYSTEM IDENTIFICATION FROM AMBIENT AND FORCED VIBRATION TESTING

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Abstract: For over two decades now the author and his team have accumulated wide-ranging experience in the dynamic testing of structural systems (such as flooring systems in buildings and stadia, pedestrian and road bridges and other structural forms such as gantry frames) for the purposes of their condition and performance assessment. Identification of the in-service state of a structure is important:

- for determining its basic articulation and the nature of the conditions at its supports;
- for determining its structural performance characteristics whilst in service;
- as a precursor to performing an assessment of its load carrying capacity;
- for gauging the effect of a retrofit that may have been introduced on it;
- for performing general condition monitoring and assessment to gauge the effects of any degradation over a period of time or following a specific possibly damaging event, (eg an earthquake, storm or accidental load). This paper considers several examples of application of dynamic testing drawn from this experience to illustrate the utility of the approaches adopted concentrating on (but not restricted to) bridge engineering applications

Keywords: dynamic testing; experimental modal analysis; vibration modes; structural health monitoring

1. Introduction

Vibration-based assessment techniques for estimating the structural “health” or in-situ condition or damage of bridges and other structures has been receiving increasing attention in the engineering scientific community in recent years, [1, 2, 3]. The basis behind the techniques adopted for doing this is the simultaneous recording of vibratory response, resultant from some sort of forcing stimulus, taken over a pre-designed grid of measurement points on the structure. When the forcing stimulus is able to also be contemporaneously measured and recorded with the vibratory response, then it is possible to perform traditional frequency-domain based Experimental Modal Analysis (EMA) on the data capture, [4], whereas if this is not possible, then alternative time-domain based methods or approximate frequency-domain based methods can be used, [5, 6] for performing the EMA. Ambient vibration is the term coined for where the excitation stems from the normal operating conditions of the structure of interest eg, wind action on a tall building; wave action on an offshore structure; traffic excitation on a road bridge and/or ground motion induced by traffic on roadways underneath a flyover bridge; pedestrian excitation of a footbridge or floor system, etc. Forced vibration is the term coined for where the excitation is purpose introduced through a shaker system (eg electromagnetic/ hydraulic shaker) or an impact hammer/device. In the case of forced excitation the input forcing function and its characteristics are user controlled/specified and able to be measured whereas for ambient excitation, it is not normally possible to measure the excitation forcing function and its characteristics.

The primary function of EMA is to essentially produce a set of mode shapes and their associated natural frequencies of vibration and damping levels from the original contemporaneously measured data capture after suitable conditioning and transformation. Information gleaned from the modes of vibration themselves or from the transformed original data can then be used to perform a “health assessment” of the structural system to which the data capture corresponds. This could be in the form of a direct comparison of the modes themselves with those predicted by a Finite Element Analysis (FEA) model in which differences between these can be used to perform FEA model updating of modelling parameters. Alternatively, a number of approaches that operate on the measured response characteristics to determine a Damage Index can also be exercised in an attempt to extend the condition assessment to the point of identifying the damage location(s) and degree of severity.

1.1 Overview of approaches for condition identification

Traditional Experimental Modal Analysis (EMA) is able to be performed when controlled forced excitation takes place simultaneously with response measurement over a sufficiently detailed grid of points over the structure to enable mode shapes, mode frequencies and associated damping levels to be determined using specialist software that operates on Frequency Response Functions (FRFs) in the frequency domain, [6]. It is also possible to estimate mode shapes, modal frequencies and associated damping levels from response only measurements, as would be the case with ambient vibration testing, where the modeling takes place in the time domain, eg ARMA models, [5], or a simplified modal analysis via Operational Deflection Shapes (ODS) in the frequency domain, [7], or through the use of wavelet analysis (combined/hybrid approach in the time and frequency domains), [8]. In any of these cases, the experimentally obtained structural dynamic response characteristics as exemplified by the identified modal properties (or various derivatives therefrom that distinguish the style of "damage detection algorithm") can then be compared with those obtained from previous past testing or from Finite Element Analysis (FEA) models of a "healthy" structure to ascertain whether degradation/damage has taken place and if so the location(s) and/or degree of severity, [9, 10, 11].

Figure 1 provides a diagrammatic representation of the so-called *model updating* approach of structural system identification that is representative of one of these classes of structural condition assessment, by way of illustration of such a technique. The model parameters that are "updated" would include the degree of fixity at the abutment supports and the effective EI and GJ value of the bridge deck for the bridge example depicted in the case of Fig. 1. Matching criteria would include frequency matching and "goodness of fit" of the mode shapes as described by the Modal Assurance Criterion (MAC_{ij}) between the modeled mode shape $\{\phi_i\}$ and the experimentally observed mode shape $\{\psi_j\}$ given by:

$$MAC_{ij} = \frac{\left|\{\phi_i\}^T \{\psi_j\}\right|^2}{\{\phi_i\}^T \{\phi_i\}\{\psi_j\}^T \{\psi_j\}} \tag{1}$$

MAC values greater than 0.9 reflect a high degree of correlation (with 1.0 being a "perfect" fit) and values less than 0.1 associated with uncorrelated (virtually orthogonally disposed) modes.

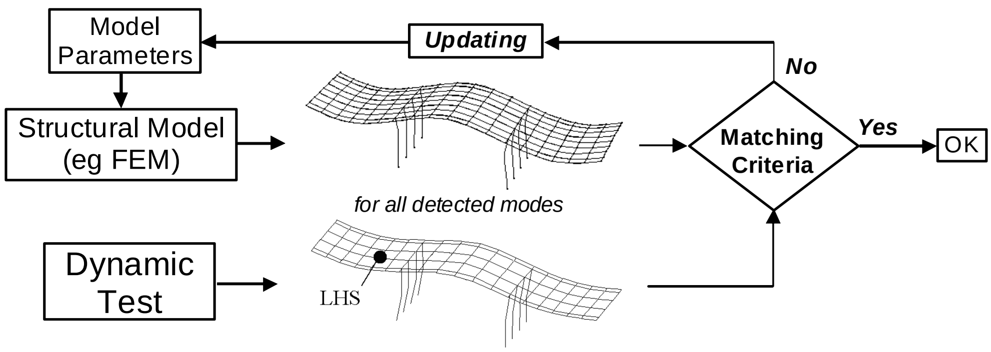

Figure 1: *Model updating approach for structural system identification*

1.2 Requirements for implementation

Ideally, the "hardware" necessary for performing structural system identification via vibration response measurements principally consists of:

i. A set of transducers suitable for measuring vibration response (eg accelerometers often used)
ii. An excitation source (impact device, shaker system (such as the Linear Hydraulic Shaker (LHS) or electromagnetic type) or healdrop excitation by a person (in the case of a floor system)

iii. A Data Acquisition System (DAS) with anti-aliasing filters, simultaneous sample and hold, modules for transducer signal-conditioning and excitation, and data storage/processing features

In addition to the above hardware set, suitable software to analyse the data capture for the key dynamic characteristics of the system under test, would also be necessary. The degree of sophistication of the hardware used for the data capture and the algorithms adopted in the analysis software is dependent upon the nature of the structural system under investigation and the degree to which the information content in the data capture is to be explored by the analyst.

Possible investigation scenarios would include:

i. Analysis of just the primary modal characteristics (natural frequency and associated damping) from response measurement records at a single point, eg from footfall excitation using simple curve-fitting of the response decay record itself (in the case of a floor system) or from performance of a Randec analysis [12], on multiple repeat record sets, (in situations where response only measurements have been performed). For this situation a simple inexpensive tri-axial accelerometer, such as a GCDC X6-2, [13], can be used, as it is compact and incorporates on-board storage of the data capture onto an SD card, which is easily accessible via USB connection directly onto the accelerometer unit itself.
ii. Performance of Experimental Modal Analysis (EMA) from ensemble-averaged FRF data determined from multiple repeat measurement of both the vibration response (over a grid of points) and the single point excitation force responsible for the vibration, [4, 6]. (This analysis determines the modal characteristics – mode shapes and corresponding natural frequencies and damping levels, of all participating modes in the test frequency range).
iii. Performance of a Simplified Experimental Modal Analysis (SEMA) from ensemble-averaged *Relative* Response Function (RRF) data evaluated from multiple repeat record sets of the vibration response (again taken over a grid of points) *relative* to a chosen reference response measurement point for when the excitation source is not measured, such as is the case with ambient vibration – eg pedestrian-induced vibration of footbridges and floor systems, traffic induced vibration of road bridges, [7], wind-induced vibration of building structures and trees, [14]. It is normally possible to determine the mode shapes and natural frequencies of only those modes that are sufficiently separated in frequency reasonably accurately using this simplified approach with estimation of associated damping being less reliable.
iv. Evaluation of modal characteristics via a Data Dependent Systems (DDS) approach using Auto-Regressive Moving Average Vectorized (ARMAV) modelling from time-domain records of response when excitation force measurement not possible, [5, 15]. Alternatively, commercial specialist software for dealing with data of this type that have built on and improved upon the DDS approach, such as ARTeMIS, [16], can be used.

In essence, the choice of analysis technique is dependent upon the particular conditions at hand. The experience gained with performing structural system identification via vibration response measurements of bridge, beam/frame and floor systems, and even trees, by The University of Melbourne, has embraced the full range of possibilities outlined above, [17, 18]. Some examples drawn from this experience, featuring key results, are presented in the sections that follow.

2. Structural system identification of road bridges from forced excitation

A 10-tonne Linear Hydraulic Shaker (LHS) system in combination with a 16-channel DAS involving 15 "roving" accelerometers has been used on a number of dynamic testing exercises on road bridges in country Victoria, Australia, for the purpose of performing structural system identification and gauging the in-service condition of the bridges so tested.

2.1 Application to typical simply supported span of multi-span RC deck on steel girder road bridge

A typical nominally simply supported span of McCoy's Bridge over the Goulburn River was dynamically tested so as to identify its in-service condition and verify the integrity of the composite action of the RSJ girders imbedded in the RC deck slab. Figure 2 depicts some of the features of this field test experience with a view of the hydraulic shaker mounted on the bridge deck, the 16 second long traces of vibration measurement taken contemporaneously with the force trace (a type of Swept Sine Wave forcing) and the associated ensemble averaged FRF function for this accelerometer (from 16 repeat test records) typical of an internal point from the 7 x 7 measurement grid adopted. Figure 3 depicts a photo of the bridge with its multiple simply supported spans, and the first three modes identified from DSMA which compare favourably with FEA predictions for a "healthy" bridge deck. Structural system identification here has verified the integrity of the composite action between the steel girders and the bridge deck, despite the age of the bridge at the time, being over 60 years.

2.2 Application to typical continuous span slab on beam bridge over Concongella Creek

Figure 4(a) depicts the Concongella Creek Bridge near Stawell, Victoria. This RC deck-on-beam bridge is over 60 years old and consists of three continuous spans and is meant to have been constructed en-castre with its abutment ends. Dynamic testing using the LHS and EMA via DSMA software enabled identification of several modes of vibration, the first three of which are depicted in Fig 4(b). It is noted that DSMA was capable of distinguishing Modes#2 and #3 despite these being so close in frequency. In addition, it is also clear from the observed modes that the fixity at the abutment ends has deteriorated to now act virtually as pins. The enhanced torsional stiffness due to aggregate interlock is reflected as a slightly higher modal frequency in Mode#2 than is predicted by FEA, whereas the observed and predicted modal frequencies of the flexural modes are in closer agreement.

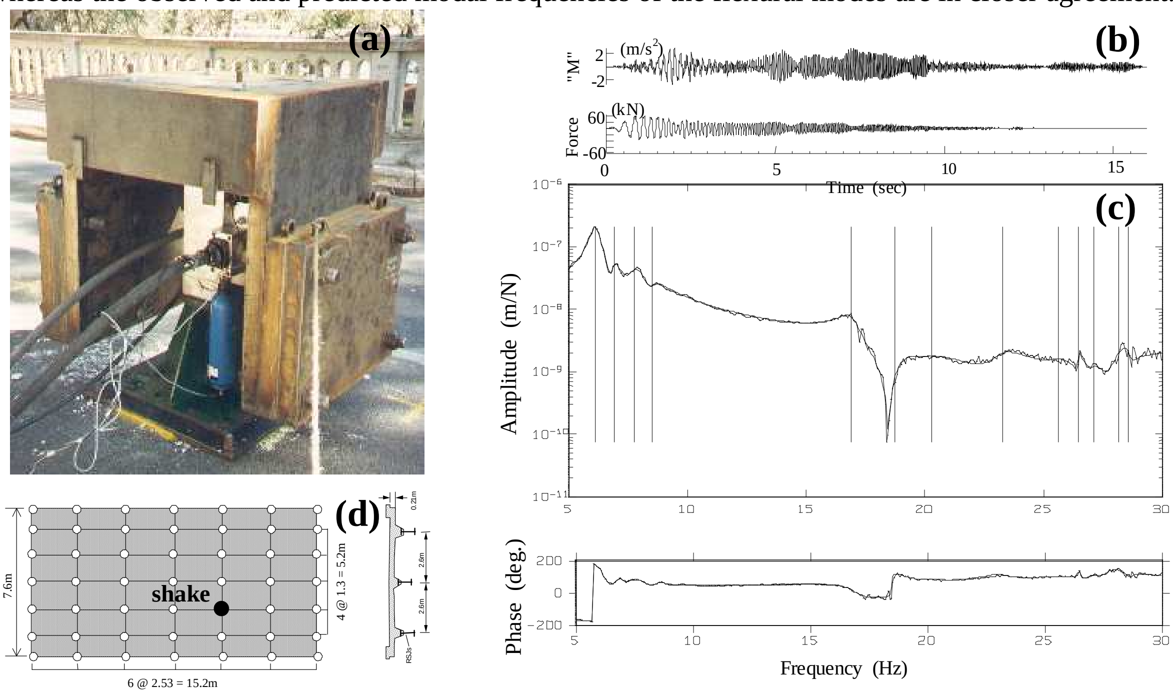

Figure 2: (a) *Hydraulic Shaker unit* (b) *Typical accelerometer and Forcing traces* (c) *FRF details for a selected accelerometer on a test span of McCoy's Bridge (d) Grid of accelerometer locations*

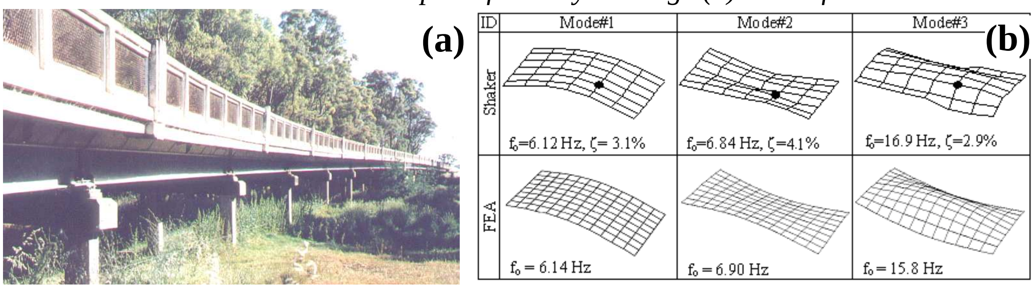

Figure 3: (a) *View of McCoy's Bridge* (b) *Comparison of EMA and FEA modal predictions*

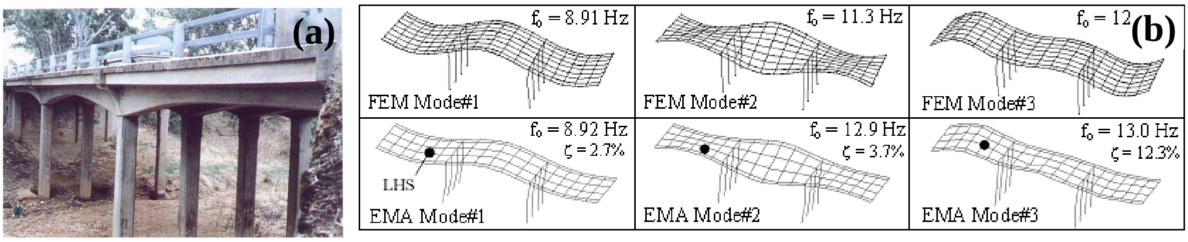

Figure 4: (a) *View of Concongella Creek Bridge* (b) *Comparison of EMA and FEA modal predictions*

3. Structural system identification from impulsive or ambient excitation

Simplified EMA, (SEMA), would suggest that the operational deflected shapes at or close to a natural frequency of vibration of a structure, can yield quite a good approximation to the corresponding mode shape of the structure at that frequency. Vibration response measurements taken contemporaneously over a grid of points on a structure, in the absence of an ability to measure the excitation responsible for the response, would be invaluable to such a modal identification exercise. Specialist software, such as ARTeMIS, can improve beyond the approximate capabilities of SEMA to provide better estimates of the modal parameters to include estimates of damping levels as well as modal frequencies and corresponding mode shapes. A couple of examples of SEMA drawn from our experience are provided in this section.

3.1 Application to heritage listed Swing Bridge at Sale, Victoria

The Swing Bridge at Sale, constructed in 1883 principally of wrought iron trusses with timber decking, and a balanced single swing span of 45.7m is located over the Latrobe River near Longford, close to the confluence of the Latrobe and Thomson Rivers, (see Fig. 5(a)). SEMA was performed on the bridge with excitation from a drop-weight device prior to and just after restoration works to verify the stability and integrity of the central pile group about which the bridge swings and the effect of replacement of the timber deck which was deemed to be in poor condition, [19, 20].

Dynamic response measurements were performed over a grid of 16 measurement points on one cantilever span before remedial work took place and repeated on the opposite balanced span when the bridge was part open with cantilever ends free, (first test series). These measurements were repeated at a later date, post installation of the refurbished timber deck, for the part open and fully closed conditions, (second test series). Model updating of an FEA model suggested that the effective soil stiffness determining the stiffness of the central pier and influencing the modal characteristics of the bridge as a whole was virtually mid range to the values inferred from soil tests at these piers, [19]. In addition, the effect of replacing the timber deck essentially improved the torsional stiffness of the cantilever sections leading to a higher torsional mode frequency compared to the original deck where timber planks were rather loosely fitting on the deck. Figure 5(b) provides insight into the modal results of the tuned/updated FEA model by providing a comparison with those observed from SEMA.

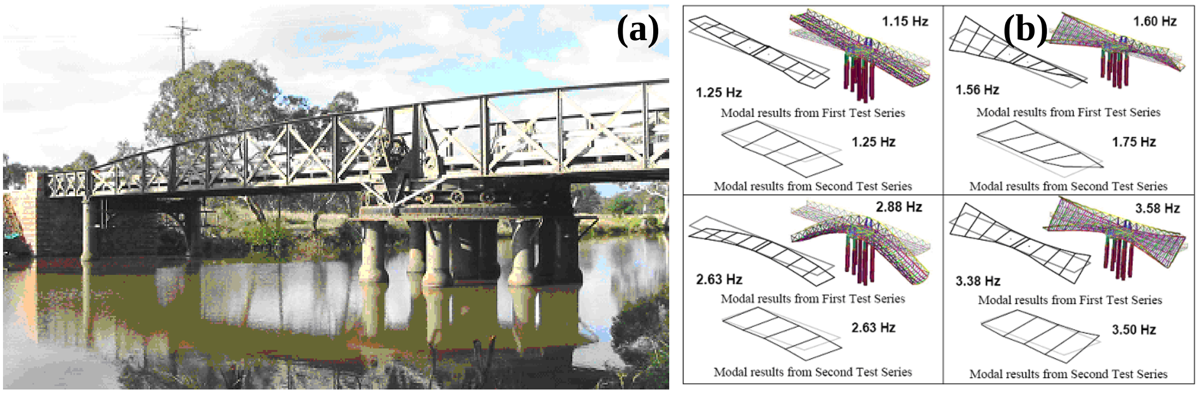

Figure 5: (a) *Sale Swing Bridge* (b) *Comparison of SEMA results pre and post restoration works*

3.2 Application to MCG Great Southern Stand

The design of the new Great Southern Stand at Melbourne Cricket Ground (MCG) involved the use of deep tapered steel cantilevers supporting pre-cast sectioned concrete units onto which seating could be bolted, once these were fixed in position. The steel cantilevers act as the primary structural support and as such were found to be significantly under-stressed under normal operating conditions (fully seated audience). There were deemed significant potential savings to be realised by reducing the plate thickness of these cantilevers, whilst satisfying strength requirements. However, these savings would only be possible if the primary mode frequency of the stand (and integer multiples thereof) remained clear of the frequency bands potentially able to be excited by crowd behaviour. Figure 6(a) & (b) depict a view of a portion (two adjacent sectors) of the stand being dynamically tested for its primary mode characteristics when construction was in progress for the remaining sectors, whilst Fig. 6(c) & (d) depict the response spectrum of a typical accelerometer and the results of SEMA taken over the relatively course grid of accelerometer measurements for excitation from an impact hammer. The spectrum depicts a cluster of closely spaced modes essentially of the same shape. The frequency conditions for the first mode cluster were deemed not to warrant re-design of the cantilever units to reduce plate thickness and overall material, fabrication and construction costs.

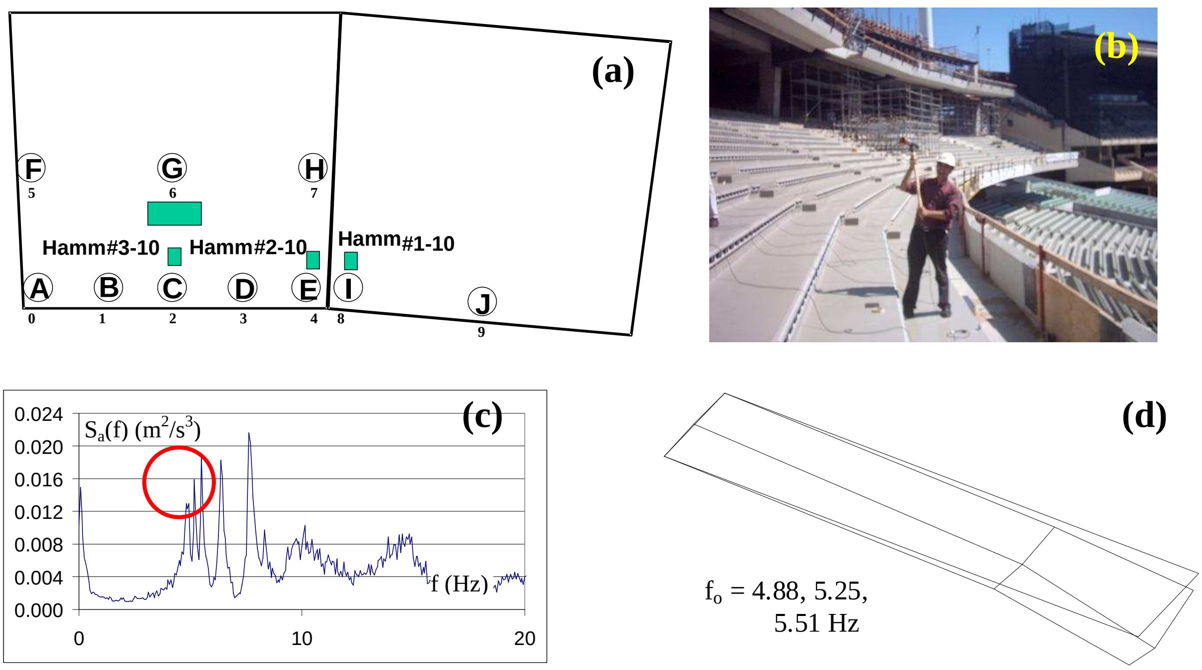

Figure 6: (a) *Accelerometer grid* (b) *Impact hammer testing in progress*
(c) *Primary mode "cluster" (accelerometer C response spectrum)* (d) *SEMA mode shape*

3.3 Application to gantry frames over Monash freeway

A much simpler approach towards ascertaining the primary mode frequencies of a selection of the fifteen newly constructed gantry frames over the Monash freeway was adopted recently, [21], with the advent on the market of the GCDC X6-1A and X6-2 model compact tri-axial accelerometer units with self-contained data logging and recording capabilities, [13]. The rather slender design (spans ranging between 40.3 and 53.6m with uniform box girder beam sections 1.2m wide and depth ranging from 0.8 to 1.2m, depending on the particular gantry frame considered) and the sharp features of the box edges of these gantry frames, prompted some concern over the possibility of these being susceptible to vortex induced excitation by wind and other associated aero-elastic instability phenomena such as galloping excitation. It was therefore decided to ascertain the in-service primary modal properties (frequencies and damping values) for the longitudinal and the horizontal and vertical transverse directions relative to the axis of the box girder beam section, of a selection of these gantries, using single point accelerometer response measurements to ambient excitation from traffic flowing on the freeway below the frames and from the surrounding wind.

Figure 7(a) and Figure 7(b) depict a photograph of a typical gantry frame and the acceleration response spectra for the X (longitudinal), Y (vertically transverse) and Z (horizontally transverse) directions for Gantry frame "G3", respectively. The primary mode frequencies in the three mutually orthogonal response directions X, Y and Z directions of 1.18 Hz, 1.47 Hz and 1.24 Hz are clearly distinguished as rather sharp peaks, reflecting the very low associated damping values of 0.4%, 0.3% and 0.8% critical, respectively. These conditions would suggest that a mean wind speed of 13.6 m/s incident at right angles to the plane of gantry frame G3 would correspond to a Strouhal number of 0.13 that would be associated with the possibility of vortex shedding with the potential of causing resonant vertical vibration on this gantry frame. The signage and other local attachments on the frame would likely disrupt the formation of any regular vortex street, but this contention remains to be further investigated.

4. Concluding remarks

This paper has considered a range of dynamic testing methods of varying degrees of sophistication aimed at performing some sort of structural system identification. Model updating techniques for tuning FEA model parameters to obtain high correlation of predicted modal characteristics to those observed from EMA or Simplified EMA (SEMA) have been overviewed and examples presented drawn from the author's experience of application to road bridges and the MCG grandstand. The versatility of newly developed compact tri-axial accelerometers with on board data-logging capabilities has been exemplified in this paper through reporting of their recent successful use for investigating the primary mode characteristics of slender gantry frames over the Monash freeway.

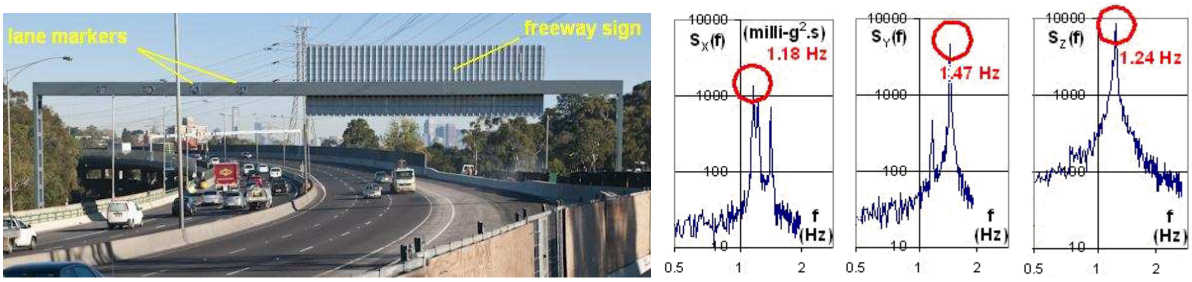

Figure 7: (a) *View of typical gantry frame* (b) *Ambient response spectra (X, Y, Z directions)*

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