Using Traffic Excitation to Establish Modal Properties of Bridges

N. Haritos, Reader in Civil Engineering, University of Melbourne
M. Aberle, Research Student (Civil Engineering), University of Melbourne

Abstract This paper describes a simplified technique of modal analysis performed on a reinforced concrete bridge in order to obtain its in-situ dynamic properties in the 0 to 50 Hz frequency range by using traffic as the source of excitation. The results obtained are compared with those from previous tests on the same structure that relied upon the use of controlled excitation by means of a shaker. The simplified technique, whilst incapable of providing any direct assessment of bridge stiffness, has nonetheless been found to provide reasonably accurate estimates of its natural frequencies and corresponding mode shapes.

1. INTRODUCTION

Due to the need by heavy road transport users to increase the load rating of road networks, there is a pressing requirement for bridge authorities to conduct estimates of the load carrying capacity of many of the bridges under their jurisdiction.

Hence, in-situ dynamic testing of bridges, based upon a hybrid implementation of experimental and theoretical modal analysis techniques, has emerged as a viable alternative to the classical "proof-load" static testing methods, (based upon simplified structural models), that can be of valuable assistance with performing a bridge load rating re-assessment.

Static testing (in which a known heavy load is applied to a bridge deck and displacements are monitored at a selected few points on the bridge superstructure) can only provide estimates of some of the bridge's stiffness characteristics. Experimental modal testing, on the other hand, can yield a great deal more information about the in-service condition of the bridge through identification of its modal properties (natural frequencies, mode shapes and damping levels). The information provided can be of immense value for the verification of finite element numerical models of the bridges concerned which can then be used with confidence by bridge engineers for predicting their load carrying capacities. In addition, (and largely because measurements associated with dynamic testing are taken over a much finer grid than used in static testing), it is possible that departures detected from the expected stiffness characteristics of the structure under test can be analysed to determine the location and severity of defects.

1.1 The University of Melbourne Experimental Modal Analysis Testing System

In recent years, the experimental modal testing technique, as implemented in the University of Melbourne dynamic testing package, has proven to be quite efficient and accurate in providing detailed information on the modal properties of bridge superstructures, (Chalko and Haritos (1993); Chalko et al, (1995a); Haritos et al (1993, 1995); Khalaf and Haritos, (1994); Khalaf (1994)).

The bridge over the flood plains of the La Trobe River on Princess Highway East near Rosedale, Victoria, has been the subject of the first field test of a collaborative research program by the Department of Civil and Environmental Engineering at The University of Melbourne and Vicroads, the State road and bridge authority of Victoria, that made use of controlled input excitation of the bridge decks under investigation.

Two adjacent 15.2 m simply supported spans of this 22 span over 60 year old reinforced concrete bridge were dynamically tested using The University of Melbourne experimental modal analysis testing package in March 1993.

This testing package consists of a 10 tonne actuator/shaker exciter powered by a 260 litre/min hydraulic power supply and controlled by hybrid data acquisition/control system which is also used to acquire data from transducers (usually a complement of 10 accelerometers) that are sequentially placed over a pre-defined grid of points on the bridge to measure its response whilst simultaneously recording details of the input forcing from the excitation. The force and response traces are used to evaluate Transfer Functions which are progressively averaged from repeat testing in the same configuration to reduce the levels of random "noise" on the data. These Transfer Functions are later operated on by special purpose software packages such as TMODE - that uses a curve-fitting procedure in order to identify the natural frequencies, damping ratios and mode shapes of the bridge deck under test, and more recently, NMODE - that uses a global simultaneous eigenvalue/eigenvector fit to obtain this same information (see Chalko et al (1995) ibid).

The first of the two spans was tested in both "force/actuator" (actuator reaction directly onto the ground) and "displacement/shaker" modes of actuator operation (3-tonne concrete mass suspended from the actuator which was itself clamped on the underside of a beam), whereas the second span was tested in "force/actuator" mode only. Results of the mode shape over a grid of 28 points (see Fig. 1) have been evaluated for the two adjacent decks as were those obtained from a detailed Finite Element Model using the package STRAND6 and both sets were made available to the present study, (Khalaf, (1994)).

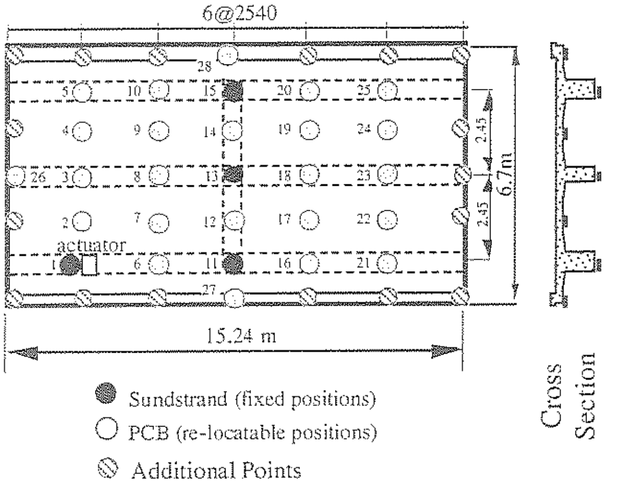

Figure 1: Grid on Bridge Deck Used in Original Test Series

1.2 Use of Traffic Excitation

Although these earlier field tests proved to be very efficient, the main disadvantage of using controlled forced excitation lies in the bulkiness of the major components necessary to achieve this (the shaker and hydraulic and electrical power supplies) that need to be transported and assembled at different testing locations on the bridge, coupled with the requirement that the bridge be closed during the performance of the actual testing, (Luz, (1992); Green (1995)).

Therefore, the objective of this most recent field test, which was carried out on the first span tested on the La Trobe river bridge, was to investigate the use of traffic generated vibration records as a means of performing a somewhat simplified "modal analysis". (The term "simplified" is used, since the technique concerned does not involve any measurement of the forcing and is based upon somewhat simplified assumptions).

The obvious major advantages of the technique are that the bridge does not have to be taken out of service during testing, and that no expensive equipment is required to excite the bridge. (These advantages are particularly useful for major, log-span bridges that are difficult to excite and too important to close to traffic). The obvious disadvantage is that because the input excitation cannot be measured, this poses problems on the technique for performing the analysis of the vibration response data collected.

Consequently, the purpose of the present study involved a re-visit of the first span of the La Trobe river bridge that was previously tested in April 1995 in order to perform a re-evaluation of the mode shapes and natural frequencies of the deck by making use of response measurements from excitation by traffic only on the bridge (ie in the absence of "controlled" excitation afforded by an exciter). The original 28 point grid on this bridge was extended to 45 in order to include further points along both edges of the span and to provide an extended regular mesh. This was done in an attempt to improve on the representation while at the same time providing the capability of a direct comparison with the results of the earlier study over the required 28 point sub-set of the data.

2. DESCRIPTION OF THE "SIMPLIFIED" EXPERIMENTAL MODAL ANALYSIS TESTING SYSTEM

2.1 Description of System Components

The three essential components necessary for any experimental modal testing technique to be capable of evaluating the vibration characteristics of a bridge super-structure are:

(i) an excitation mechanism of suitable size

(ii) a transduction system to measure the vibration response over a grid, and

(iii) analysis software capable of extracting the desired information from the measurements obtained.

All of these necessary components are identifiable parts of the University of Melbourne experimental modal dynamic testing system package used for the field testing of bridge super-structures. Item (i) would normally be the 10 tonne linear hydraulic actuator/shaker (in the case of controlled excitation) but here traffic excitation will be considered in its stead.

Transducers that can be used for vibration measurements (items (ii) in the above list) include those that measure displacement, velocity or acceleration or any other quantity that can be directly related to deck motion. Here, a complement of 10 accelerometers: 4 SUNDSTRAND servo-accelerometers and 6 PCB piezoelectric accelerometers have been used for this purpose. The sensitivities of the two accelerometer types are 5 Volts/g and 10 mV/g respectively, which make the SUNDSTRAND model more suited to the measurement of low amplitude response.

Software for performing the measurements and aspects of the experimental modal analysis include: TSPECTRA (1992), and TMODE (1992).

TSPECTRA can be described as a real time hybrid data acquisition/signal conditioning and experiment control hardware/software package. The hardware is based upon an IBM compatible PC 486 computer system and includes an ADDA card capable of up to a 200 kHz throughput on 16 simultaneously logged anti-alias filtered data channels. The hardware system is also able to operate from battery power if necessary in remote locations. Modules in TSPECTRA for evaluation of Fourier transforms and manipulation of the data series captured allow the package to operate as an equivalent 16 channel spectral analyser under software emulation on the same computer.

TMODE can be described as a general Experimental Modal Analysis (EMA) software package with modules for performing curve-fitting operations, eigenvalue evaluation, and eigenvector extraction (natural frequency and mode shape) as well as for wireframe animation display of mode shapes. A new version of the software (NMODE) with improved eigenvalue/eigenvector fitting capabilities (Chalko et al, (1995b)) was made available for the evaluation of modal characteristics soon after performance of the field tests for the present study.

The present study essentially made use of items (ii) and (iii) listed above, as excitation was only from the available traffic.

2.2 Basis of "Simplified" Approach

Although a small number of alternative techniques that can be used to perform a "simplified" EMA can be found in the literature (eg Luz (1992)) most of which are based upon the use of response cross-spectral density information and a variety of simplifying assumptions (such as "randomicity" of the excitation force), an alternative easily implemented approach will be described here.

Consider a grid of N measurement points on a structure that is dynamically responding to external forcing. The collection of second order ordinary differential equations describing the vibratory motion of such a structure over the grid of measurement points, (Craig, (1981)) becomes:

$$m\ddot{x} + c\dot{x} + kx = f \tag{1}$$

where m, c and k are the (NxN) mass, damping and stiffness matrices, and f and x are (Nx1) vectors of the force and response traces respectively.

Now, consider the response to be approximated to the sum of the first M (where M << N) modes of the structure with modal amplitudes, a, ie

$$x \approx \Phi a \tag{2}$$

where a is the required (Mx1 vector) of modal amplitudes and Φ is the (NxM) assemblage of the first M (Nx1) real eigenvectors obtained from Eq. (1) when both c and f are taken as zero-valued and their scaling satisfies: $\Phi^T m \Phi = I$, and $\Phi^T k \Phi = \Lambda$, where I is the (MxM) identity matrix and Λ is a (MxM) diagonal matrix containing the first M eigenvalues (corresponding to the square of the first M natural circular frequencies, ϖ_i^2).

Substituting Eq. (2) into Eq. (1), we obtain the uncoupled set of second order differential equations:

$$\ddot{a} + \varsigma\dot{a} + \Lambda a = r \tag{3}$$

where ς is a (MxM) diagonal matrix with diagonal elements given by $(2\varpi_i\zeta_i)$ where ζ_i is the critical damping ratio of mode i and r is an (Mx1) vector given by $\Phi^T f$.

Consider now the Fourier Transform of the i^{th} equation in the set described by Eq. (3), then

$$a_i(\omega) = \frac{r_i(\omega)}{\left(1 - \left(\frac{\omega}{\varpi_i}\right)^2 + 2j\zeta_i\left(\frac{\omega}{\varpi_i}\right)\right)} \tag{4}$$

where

$$r_i(\omega) = \sum_{k=1}^{N} \Phi_{ki}\, f_k(\omega) \tag{5}$$

Now, from Eq. (2), the Fourier transform of the q^{th} response location becomes:

$$x_q(\omega) = \sum_{p=1}^{M} \Phi_{qp}\, a_p(\omega)$$

$$= \sum_{p=1}^{M} \frac{\Phi_{qp} \sum_{k=1}^{N} \Phi_{ki}\, f_k(\omega)}{\left(1 - \left(\frac{\omega}{\varpi_i}\right)^2 + 2j\zeta_i\left(\frac{\omega}{\varpi_i}\right)\right)}$$

$$= \sum_{k=1}^{N} H_{qk}(\omega)\, f_k(\omega) \tag{6}$$

where $H(\omega)$ is the (NxN) Frequency Response Function matrix which relates response $x(\omega)$ to forcing $f(\omega)$ (ie $x(\omega) = H(\omega)f(\omega)$).

Now in the case of response at the natural frequency of one of the modes, ie $\omega = \varpi_i$, it is found that for structural systems exhibiting small damping (where typically, $\zeta_i < 0.1$), the contribution made by any other mode m, (where $m \neq i$) , can be considered to be very small, for ϖ_m sufficiently separated from ϖ_i, ie

$$\left(\frac{1}{\left(1 - \left(\frac{\omega_i}{\varpi_m}\right)^2 + 2j\zeta_m\left(\frac{\omega_i}{\varpi_m}\right)\right)}\right) << \left(\frac{1}{2j\zeta_i}\right) \tag{7}$$

Eq. (6) can then be approximated to:

$$x_q(\varpi_i) \approx \Phi_{qi}\, a_i(\omega)$$

$$= \Phi_{qi} \frac{\sum_{k=1}^{N} \Phi_{ki}\, f_k(\omega)}{2j\zeta_i\left(\frac{\omega}{\varpi_i}\right)} \tag{8}$$

Similarly, the Fourier transform of the response at location "o" considered to be a "reference" location, at this same natural modal frequency becomes:

$$x_o(\varpi_i) \approx \Phi_{oi} \frac{\sum_{k=1}^{N} \Phi_{ki}\, f_k(\omega)}{2j\zeta_i\left(\frac{\omega}{\varpi_i}\right)} \tag{9}$$

Considering the ratio of the Fourier Transforms of the response at the two locations q and o to be given by $R_{qo}(\omega)$, ie

$$R_{qo}(\omega) = \frac{x_q(\omega)}{x_o(\omega)} \tag{10}$$

Then, from Eq. (9), we simply have

$$\frac{\Phi_{qi}}{\Phi_{oi}} \approx R_{qo}(\varpi_i) \tag{11}$$

2.3 Practical Implementation of the Simplified Approach

Equation (11) provides the basis for the estimation of the modal amplitudes of mode i at any point p relative to the reference location o (modal amplitude has a unit value at the reference location) from knowledge of the modal frequency ϖ_i and function $R_{qo}(\omega)$.

The choice of the reference location "o" needs satisfy the requirement that it be sufficiently far removed from a nodal point of all mode shapes under consideration for reasonable accuracy to be obtained using this method in the presence of noise in the response data collected. Estimation of the values of natural frequencies ϖ_i can be performed by referring to the peaks in the autospectrum of the response at the reference location, $S_{oo}(\omega)$, (or any other suitable point for that matter). (The autospectrum can be simply obtained from the product of the Fourier transform of the response measurement and the complex conjugate of this Fourier transform, ie $x_o(\omega).x^*_o(\omega)$).

Practical implementation of the method, in the case of uncontrolled traffic excitation of a bridge superstructure, requires that $R_{qo}(\omega)$ be estimated from an ensemble average of several realisations in order that any errors associated with electrical noise in the measurement transducers be reduced to sufficiently small levels for the values at estimated natural frequency locations, $R_{qo}(\varpi_i)$, to themselves be able to be reliably estimated.

The amplitude of the mode shape at point q relative to the reference location can then be simply estimated by considering the magnitude of $R_{qo}(\omega)$ in the vicinity $\omega = \varpi_i$. The sign of the mode shape at point q relative to the reference location (whether +ve or -ve) can then be ascertained from the phase of $R_{qo}(\omega)$ (ie +ve if at or at least very close to 0° and -ve if close to 180°).

The evaluation of ensemble averaged $R_{qo}(\omega)$ and autospectra $S_{qq}(\omega)$ (where "q" is taken over all measurement points including the reference location "o") is easily facilitated within the package TSPECTRA by request. (The evaluation of $R_{qo}(\omega)$ for example is equivalent to the interpretation of $H_{qo}(\omega)$ when $f_o(\omega)$ is simply equated to $x_o(\omega)$ (ie $f_o(t)$ is taken to be $x_o(t)$). Also, acceleration records $\ddot{x}(t)$ can be used in lieu of $x(t)$ as these lead to an identical $R_{qo}(\omega)$.

3. RE-TESTING OF THE LA TROBE RIVER BRIDGE

Span#2 of the two spans of the La Trobe River bridge which had been previously dynamically tested using the University of Melbourne dynamic testing package (Khalaf, (1994)) was re-tested using the natural traffic over the bridge as the excitation source and the simplified approach detailed in § 2 above. Because no shaker or hydraulic power unit was required in this simplified testing procedure, all the equipment (transducers, associated power supplies and data acquisition system) was easily able to be transported to the site using a standard station wagon. (Figure 2 depicts a photograph of this 15.2m long 6.7 m wide span taken when the re-testing was in progress).

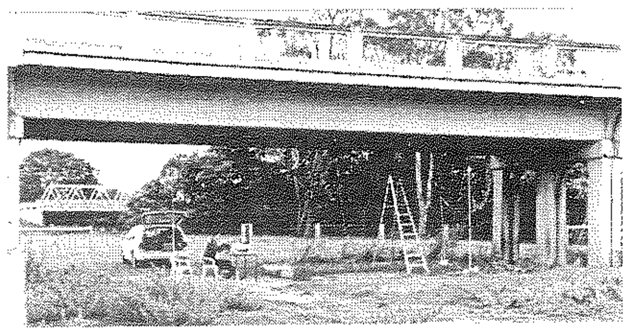

Figure 2: Photograph Depicting Experimental Data Acquisition of Span#2 in Progress

Principally because of the relative "insensitivity" of the 6 PCB model accelerometer transducers (when compared with the 4 SUNDSTRAND accelerometers available to this study), only records with "heavy traffic" ie traffic with an dynamic intensity sufficient to provide signals of "reasonable" strength from the PCB accelerometers, were selected for subsequent retention and analysis during data capture.

A basic grid of 45 accelerometer locations on the underside of the bridge deck was chosen for conducting measurements of the dynamic response of the bridge superstructure, (see Figure 3). For all accelerometer re-locations, three of the SUNDSTRAND accelerometers remained bolted onto pre-positioned metal plates at fixed locations throughout the tests whilst the other and remaining 6 PCB accelerometers were relocated on an additional 6 occasions to new measurement positions over the grid as required: (3 + 7*6 = 45). Since the PCB accelerometers were mounted onto magnetic bases, they could easily be relocated onto the required grid point locations identified by pre-positioned steel plates. The single "roving" SUNDSTRAND needed to be bolted onto fixed pre-positioned plates that had bolt holes suitably threaded for this purpose. One of the three fixed SUNDSTRAND accelerometers, (the one located over the grid point originally used for the forced excitation of the bridge and identified as S2) was chosen as the reference location for the simplified EMA procedure.

Suitable data records of 2048 data points per channel per record were sampled regularly over a 1/128th second sampling interval to produce time series of 16 seconds duration. This resulted in a peak represented frequency (allowing for "roll-

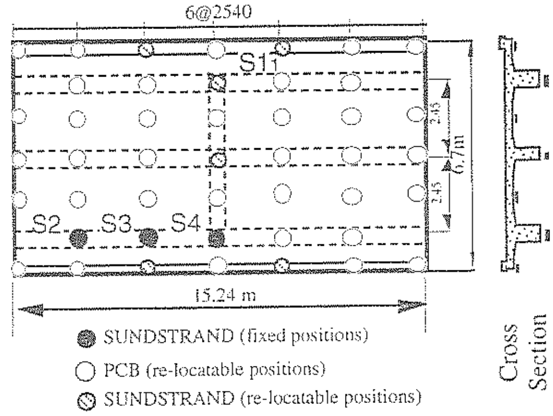

Figure 3: Grid of Measurement Locations

off" on the anti-aliasing filters) of 50 Hz, ie 800 frequency ordinates at 1/16 Hz increments, in the requested multi-record averaged $R_{qo}(\omega)$ and $S_{qq}(\omega)$ determinations. Some 64 repeat test records for every group of accelerometer setting locations were in fact used for this purpose. A total of 384 sets of time series of all 10 accelerometer measurements were in fact selectively recorded to satisfy the "signal strength suitability criterion" outlined above. This was able to be achieved over two days of testing with "normal" use of the bridge.

4. PRESENTATION OF RESULTS

A sample set of traces for all 10 accelerometers taken from one of the 64 realisations of the six accelerometer grouping configurations (considered to be representative) can be seen in Figure 4. The response due to the passage over the bridge of a "heavy" vehicle is evident in the first ~5 secs of record. Evidence of the passage of smaller vehicles, between 5 and 7.5 secs and again between 14 and 15.5 secs, is quite clear for the four SUNDSTRAND accelerometer records but has hardly been detected by the PCB "E" and "F" accelerometers. The characteristic low cycle "drift" observed in all of the PCB measurements is associated with the high amplification of the very small signals that result from their low sensitivity.

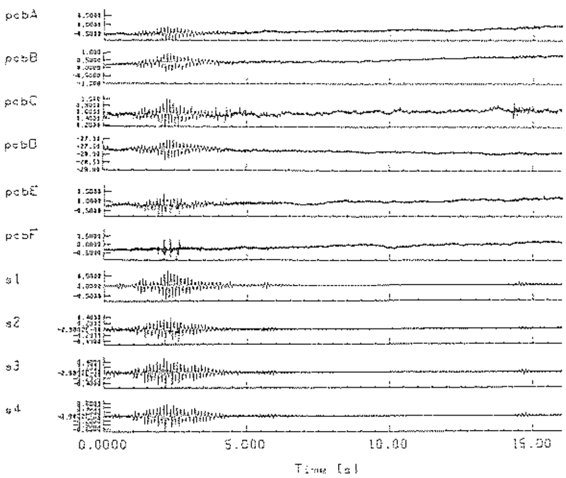

Figure 4: Sample Set of Data Records

4.1 Identification of Modal Properties

Autospectra ensemble averaged over the repeat 64 record set of accelerometer locations were used to estimate values of the natural frequencies from identification of local peaks in their variation. A representative sample of two of these variations (for SUNDSTRANDs S1 and S3) is depicted in Fig. 5.

A sample representation of $R_{qo}(f)$ for "o" taken at the reference SUNDSTRAND accelerometer S2 and "q" at another SUNDSTRAND accelerometer location, S4, (see grid illustrated in Fig. 3) is depicted over the frequency range 5 to 50 Hz in Fig. 6 and again over a more narrow frequency range of 6 to 12 Hz (encompassing the first two natural frequencies at ~8 Hz and ~10 Hz) in Fig. 6. (The latter representation was in fact used to "zoom" in on $R_{qo}(f)$ so as to estimate $R_{qo}(f_i)$ (= $R_{qo}(\varpi_i)$, for $\varpi_i = 2\pi f_i$ and $f_1 \approx 8$ Hz and $f_2 \approx 10$ Hz) conveniently "by eye".

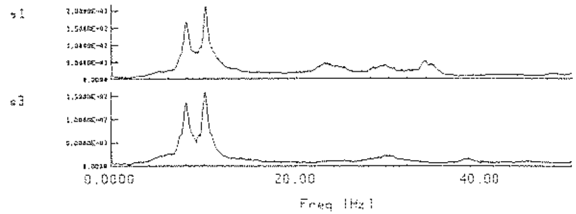

Figure 5: Autospectral Variations for S1 and S2

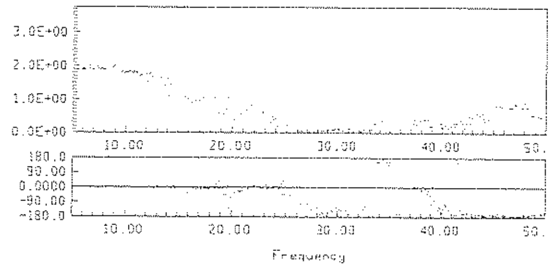

Figure 6: Sample $R_{qo}(f)$ Variation (5 to 50 Hz)

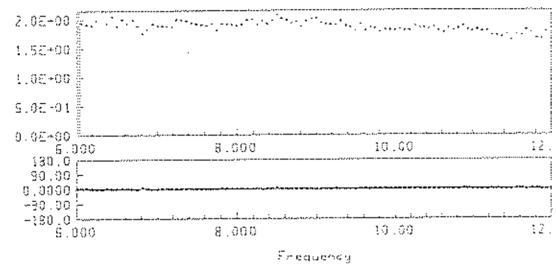

Figure 7: Sample $R_{qo}(f)$ Variation (6 to 12 Hz)

4.2 Comparisons with FEM model

Table I and Fig. 8 below compare the natural frequencies and bridge deck mode shapes obtained by the simplified EMA procedure above and the theoretical Finite Element Model (FEM) of this span produced by Khalaf (1994). While some differences are noticeable between these two independent estimates, the agreement is sufficiently close to be considered "quite good" for a bridge structure of this type.

4.3 Comparison with Forced Excitation Results

Figure 9 depicts a comparison of two representative modes from the simplified EMA approach with those obtained from earlier testing by Khalaf (1994) using forced excitation.

Table I: Comparison of Natural Frequencies

MODE	FEM Frequencies (Hz)	Experimental Frequencies (Hz)
1	8.05 Hz	7.98 Hz
2	10.83 Hz	10.1 Hz
3	15.26 Hz	15.3 Hz
4		23.2 Hz
5	30.71 Hz	30.1 Hz
6	34.41 Hz	34.5 Hz
7	34.95 Hz	
8	39.25 Hz	39.1 Hz
9	44.2 Hz	
10	44.97 Hz	45.1 Hz

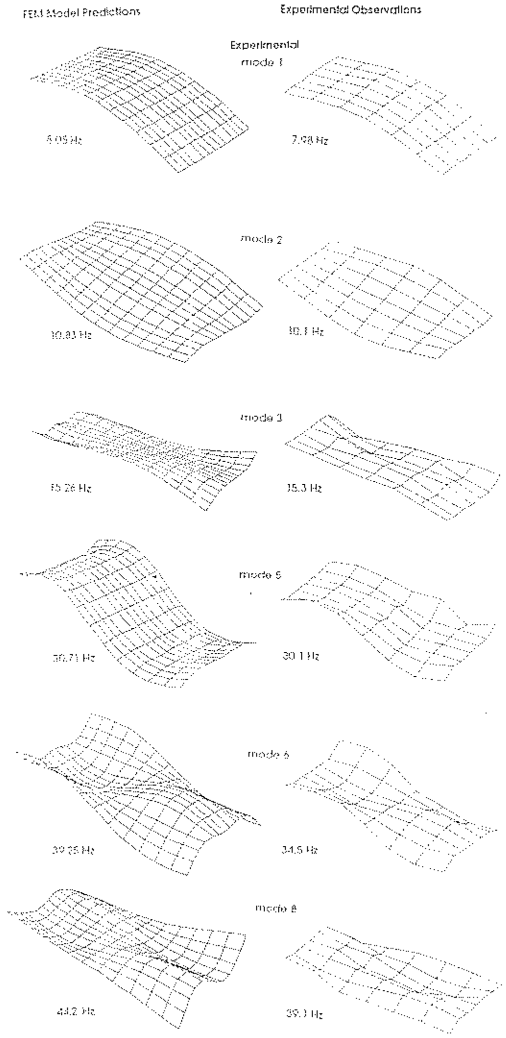

Figure 8: Comparison of Mode Shapes

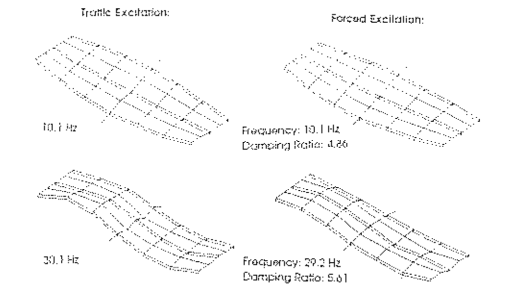

Figure 9: Comparison of Mode Shapes (Forced Excitation)

The representation is only over a 28 point grid sub-set that conforms to the earlier tests (see Fig. 1) and includes wireframe modelling of the beams on the superstructure.

Again the agreement in the mode shapes and natural frequencies between these two experimental approaches is considered to be quite good.

5. CONCLUSIONS

The use of the simplified EMA procedure described in this paper that makes use of natural traffic excitation for identification of the vibration modes of a bridge superstructure has proven to be quite successful.

Modal frequencies and mode shapes obtained by the method have been found to be in good agreement with both the findings of a previous study on the same bridge using controlled forced excitation and the results of an FEM model.

While the technique is incapable of providing direct stiffness information on the structure nor can it provide estimates of modal damping, (particular attributes of EMA with controlled input forcing) it is nonetheless considered useful as a dynamic testing tool for simple bridge structures.

6. ACKNOWLEDGMENTS

The authors wish to acknowledge the support by Vicroads and the Shire of Rosedale of this field testing program. The work also forms part of Mr. Aberle's M.Eng.Sci. degree program which is financially supported by a German DAAD scholarship.

7. REFERENCES

Chalko, T. and Haritos, N. On the Identification of the In-Service Condition of Bridge Structures using Modal Testing Techniques, *Proc. MODSIM 93*, pp 349-354, Perth, Dec., 1993.

Chalko.T, Gershkovich,V and Haritos,N. Optimal Design of Modal Experiment for Bridge Structures, *Proc. International Modal Analysis Conference IMAC-XIII*, pp 571-577, Nashville, 13-16 Feb, 1995a.

Chalko.T, Gershkovich,V and Haritos,N. Curve Fitting Algorithms for Modal Analysis, *Proc. MODSIM95*, Newcastle, pp 571-577, November, 1995b.

Craig, R.R., Structural Dynamics - an Introduction to Computer Methods, John Wiley and Sons, 1981.

Green, M., F., Modal Test Methods for Bridges, Proceedings of the 13th International Modal Analysis Conference, pp. 203-209, Nashville, 13-16 Feb., 1995.

Haritos, N., Chalko, T. and Khalaf, H., Modal Testing of Bridge Structures using a Linear Hydraulic Shaker, *Proc. ACMSM13*,, pp 349-356,Wollongong, July 1993.

Haritos, N., Khalaf, H. and Chalko, T, Modal Testing of a Skew Reinforced Concrete Bridge, *Proceedings of IMAC-XIII*, pp 703-709, Nashville, Feb., 1995.

Khalaf, H., and Haritos, N., Dynamic Testing of the La Trobe River Bridges, Proc. AUSTROADS'94 Conf., pp. 17.1-17.10, Melbourne, Feb., 1994.

Khalaf, H. Dynamic Testing of Bridges by the Method of Experimental Modal Analysis, M.Eng.Sci. Thesis, Civil & Environ. Eng., University of Melbourne, 1994.

Luz, E., Experimental Modal Analysis using Ambient Vibration, The International Journal of Analytical and Experimental Modal Analysis, pp. 29-39, January, 1992.

STRAND 6.1 - Finite Element Analysis Package, Reference Guide, G+D Computing P/L, Ultimo NSW.

TSPECTRA - 16 Channel Spectrum Analyser, User Manual, Scientific Engin. Res. P/L, Melbourne, 1992.

TMODE - Modal Analysis Program, User Manual, Scientific Engin. Res. P/L, Melbourne, 1992.