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STRUCTURAL HEALTH MONITORING OF CIVIL ENGINEERING STRUCTURES USING OUTPUT ONLY DATA

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ABSTRACT

The use of Experimental Modal Analysis testing as a tool for structural system identification and damage detection of bridge superstructures is reported in this paper. This case study presents the results of an application of the Auto Regressive Moving Average Vector (ARMAV) technique for identification of bridges. The Department of Civil & Environmental Engineering at The University of Melbourne, Australia was commissioned together with Curtin University, School of Civil Engineering, to perform dynamic testing using traffic excitation of the MRWA bridge #617 located on the Great Eastern Highway near Meckering, Western Australia.

Dynamic testing techniques were used to ascertain and monitor the structural performance of MRWA bridge in its three repair stages. The modal parameters of the bridge are estimated from ambient responses. The dynamic testing was planned as an adjunct to the static testing that was performed by Curtin University, School of Civil Engineering on this bridge.

The time domain ARMAV technique was applied to measurements obtained from such a large civil engineering structure. For each repair stage, the ambient data was also analyzed using Fast Fourier Transform (FFT) based Simplified Experimental Modal Analysis (SEMA) and time domain based method RANDEC techniques. The results of the different approaches were discussed and compared with the results of ARMAV approach. The observed dynamic characteristics obtained from dynamic testing are then used to "tune" key parameters of Finite Element Analysis (FEA) based models of the bridge superstructure, by "matching" modeled results to these characteristics. The resultant "tuned" FEA model is considered to be able to more reliably be used by the Road/Bridge Authorities (than would be possible from design information alone) in such applications as the assessment of bridge load carrying capacity. Results obtained demonstrate the utility of the technique for identification of the in-service condition of bridge superstructures and for damage-detection applications in particular

KEYWORDS

Structural health monitoring, parameter identification, ambient vibration, bridge load carrying capacity, model updating, ARMAV Stochastic Subspace Identification.

INTRODUCTION

The purpose of this paper is to document the application of the time-domain ARMAV technique for ambient testing of in-situ bridge superstructures, where response of the structure is collected at many locations. Ambient testing of bridges refers to measurements of the vibrations of bridges due to ambient loads, such as traffic, wind, waves and micro-tremors, however, the main cause of vibrations in this chapter is traffic passing over the bridge.

This paper presents results obtained from identification of the MRWA Bridge #617 ─ located on the Great Eastern Highway near Meckering, Western Australia ─ from ambient vibrations. The Department of Civil & Environmental Engineering at The University of Melbourne was commissioned together with Curtin University, School of Civil Engineering, to perform dynamic testing using traffic excitation of MRWA Bridge #617. Dynamic testing of the bridge was carried out in three different test conditions as will be discussed later. The ARMAV technique performance was compared with well-known techniques such as the FFT-based SEMA technique and the time-domain based RANDEC technique (Abu-Aisheh et al. 1999; Haritos et al. 2002). Results

are then compared with the "tuned" Finite Element Method (FEM) model constructed on the bases of the bridge geometry and experimental results obtained after testing.

Bridge #617 is a four span flat slab reinforced concrete bridge about 130 km east of Perth on the Great Eastern Highway. The bridge was constructed in 1970. Analytical load rating of the bridge showed that its capacity was only 75 percent of the current requirement of T44. A relatively cheap and easy to implement method of strengthening this bridge was proposed and trialed in 1995. The method involved application of sprayed-on concrete across the underside of each of the bridge piers to form band beams 2200 mm wide and 250 or 300 mm thick. The concrete installed in 1995 was unsatisfactory with numerous voids and was removed and replaced with sound concrete in 1998. A series of static and dynamic load tests were conducted to evaluate the effectiveness of the repairs from measurements of strains, deflections and accelerations (see Chandler and Haritos 2000)

Bridge Geometry

MRWA Bridge #617 is a 29.6 meter long, consists of four continuous spans, simply supported on elastomeric bearings at its abutment ends, Figure 1. The reinforced concrete (RC) deck was originally cast directly onto three sets of piers, each comprising four pre-stressed concrete columns approx. 360 mm square and 2.6 m apart. Slab thickness in the deck is near uniform at 305 mm with successive spans being 7.1 m, 7.8 m, 7.5 m and 7.2 m in length. The bridge is located on the Great Eastern Highway near Meckering West Australia. Visual inspection indicates a large number of cracks on the soffit particularly in the middle section of spans 1 and 4. "Spraycrete" transverse bandbeams were introduced to this bridge at its pier locations in late 1995 in an attempt to improve the bridge's shear capacity and stiffness.

These "spraycrete" transverse bandbeams were since found to contain fissures and voids, particularly around the regions of added reinforcement, which raised concerns about their effectiveness to satisfy their original intent. A decision was therefore made to replace these transverse beams and this original work "made good". An opportunity arose for applying dynamic testing techniques to ascertain and monitor the structural performance of the bridge in its three stages of structural "condition," viz:

(1) Bridge with original "spraycrete" transverse bandbeams intact - measurements performed in March 1998.
(2) Bridge with original "spraycrete" transverse bandbeams removed (i.e. to the state before these beams were introduced in the first place) – measurements performed in April 1998.
(3) Bridge with original "spraycrete" transverse bandbeams replaced with new versions of the same

DATA ACQUISITION AND SIGNAL PROCESSING

Many aspects had to be considered for an effective data analysis that would permit proper identification of the dynamic characteristics of a structure. The inherent uncertainty in determining the mode shapes of a very complex structure can be minimized by properly locating the sensing instruments and ensuring good quality of the data. During testing of a major civil engineering structure, many decisions need to be made in-situ to obtain the highest possible data quality. This can be facilitated by analyzing and processing some of the data immediately after it has been collected.

The basic grid chosen for performing acceleration response measurements for the purpose of conducting modal analysis that is suitable for ARMAV and SEMA techniques is depicted in Figure 2. A significant number of locations (118 positions) were selected for measurements of vertical vibrations. Data of acceleration response (16 second long records of 2,048 points sampled at 128 Hz) were distinguished into two sets ("car" and "truck" record sets) for the resulting analysis. Manual triggering for data capture of vehicular excursions over the bridge was performed using the TSPECTRA data acquisition software (TSPECTRA 1992) resident on the data acquisition system (DAS). Figure 3 depicts a typical view of a truck and a car passing over the bridge during dynamic testing.

From the experience in the modal testing of short span (<20m span) bridges gained by the University of Melbourne Dynamic Testing Group, a frequency range of 0 to 50 Hz had been found to capture the most significant modes participating in the response of such bridges to excitation by ambient traffic.

FINITE ELEMENT MODEL

A preliminary Finite Element Method (FEM) based model of Bridge #617 was set up using the FEM analysis package STRAND version 6.15 (G & D Computing 1993). Information on the basic geometry of this bridge was

used in this model. This preliminary FEM model was used to estimate the lowest dynamic response modes and their characteristics (mode shapes and natural frequencies) for the bridge in all three test stages prior to actual testing. Knowledge of the modal property estimates from the preliminary FEM analysis assisted with the planning of the dynamic test program such as the choice of "reference" locations in the measurement grid and details of the measurement grid itself.

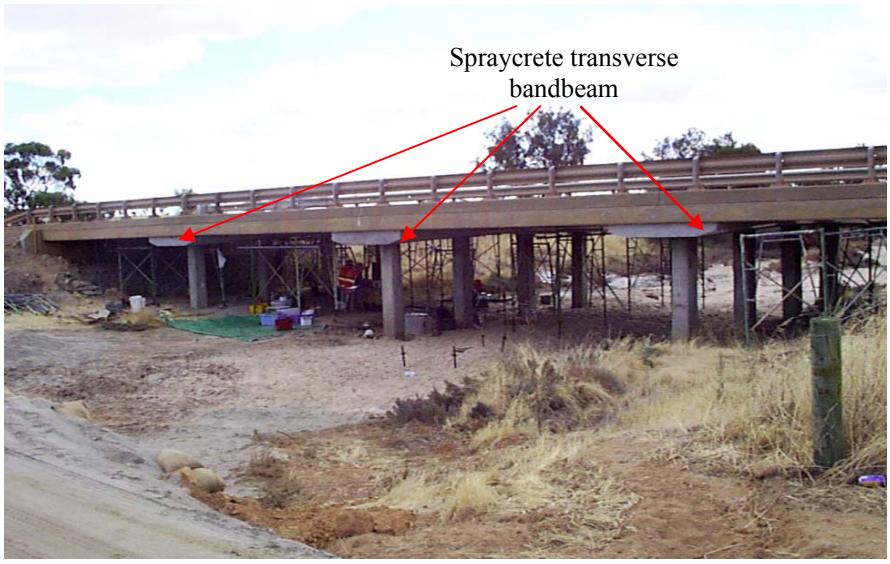


Figure 1. View of MRWA Bridge#617 prior to performance of Test #1 (Original bandbeams in place)

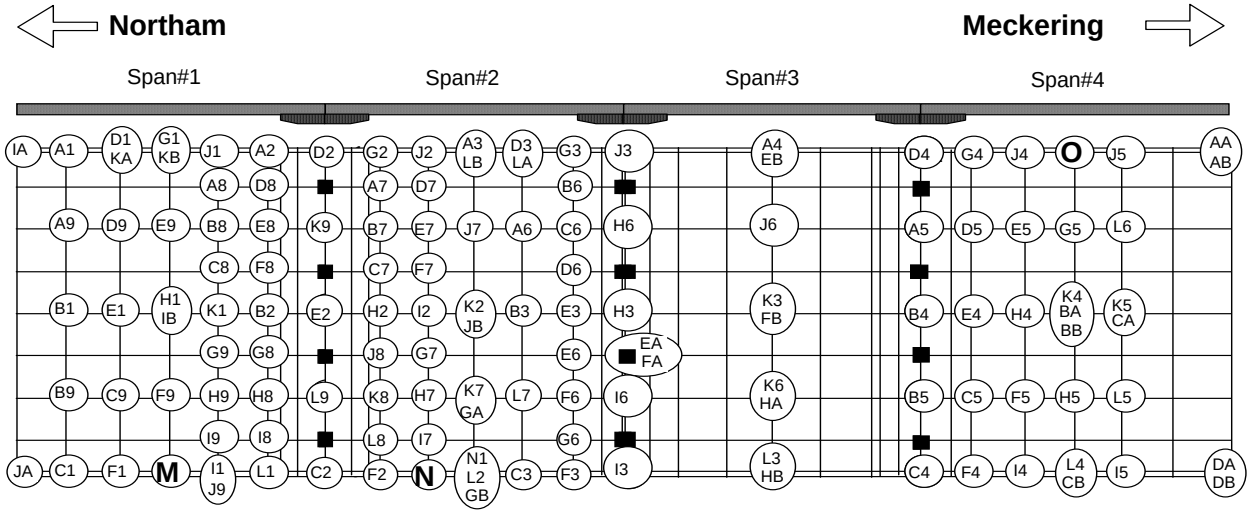


Figure 2. Accelerometer response measurement grid depicting accelerometer identification and re-location (setup) sequence for test #1. Bold identification means reference accelerometer position

MODAL PARAMETER IDENTIFICATION

Frequency-Based Methods

Simplified experimental modal analysis

The SEMA testing approach is a method developed by The University of Melbourne that attempts to obtain the modal characteristics of bridge superstructures from response measurements solely due to natural traffic excitation, (Haritos 1995; Aberle 1996). In this method, the simplifying assumption is made that responses are dominated by "resonant" modes at their corresponding natural frequencies. This assumption is reasonable for those modes that are sufficiently "separated in frequency", but is violated by the condition of modes that are "closely-spaced"; hence it poses a limitation in its general use. In addition, the technique is limited to response measurement points that either run alongside the edges of the bridge (to avoid traffic) or are located on the underside of the bridge, thus requiring appropriate access.

Figure 3. View of a typical truck (left) and a typical single car (right) crossing the bridge during dynamic testing

The application of the technique requires definition of the relative response functions, (RRFs), performed over a grid of measurement points. RRF, $\widetilde{R}_{qo}(\omega)$, is given by:

$$\widetilde{R}_{qo}(\omega) = \frac{X_q(\omega)}{X_o(\omega)} \tag{1}$$

where $X_q(\omega)$ and $X_o(\omega)$ are the Fourier transforms of response records, obtained from traffic-excitation, at locations "q" and "o" respectively, and "o" is treated as the "reference" point.

Now
$$X_q(\omega) = \sum_{k=1}^{N} h_{qk}(\omega) F_k(\omega) \tag{2}$$

where $F_k(\omega)$ corresponds to the Fourier transform of the force trace component at point "k" on the grid due to traffic-excitation and the frequency response function (FRF) for point "q" due to excitation at point "k", $h_{qk}(\omega)$, can be obtained from Eqn. (2).

Now, when circular frequency ω corresponds to one of the natural frequencies ϖ_i, the RRF value for mode "i", $Rqo(\varpi_i)$, after substitution of Eqn. (2) into Eqn. (1), approximates to:

$$R_{qo}(\varpi_i) = \frac{X_q(\varpi_i)}{X_o(\varpi_i)} \approx \frac{\varphi_{qi}}{\varphi_{oi}} \tag{3}$$

provided the adjacent modal frequencies to this mode are sufficiently "well separated". Hence $Rqo(\varpi i)$, reduces to the ratio of the modal amplitudes (active point "q" to reference point "o") of mode "i". After considering all points "q" on the grid, this result enables a mode-shape estimate to be made for the entire structure.

The method for establishing the modal frequencies, ϖi, for performing SEMA is to identify the frequency location of well-defined peaks in the auto-spectrum of response at the reference location, $So(\omega)$. The SEMA technique is thus capable of determining modal frequencies to within the frequency resolution (= 1/time-length of record) of successive ordinates in the auto-spectrum and mode shapes to an accuracy dependent upon the number of multi-record averages used to obtain ensemble-averaged versions of $Rqo(\omega)$, and the actual contributions made by adjacent modes at this modal frequency. The level of damping in these modes is not, however, able to be directly established using SEMA (Haritos and Abu-Aisheh 1998).

The Random Decrement (RANDEC) method (Cole 1973) can be applied to time domain traces of response measurements to estimate both the natural frequency (as an alternative to SEMA) and damping value associated with a chosen level of response (Haritos and Dao 1997).

ARMAV Stochastic Subspace Identification (SSI)

It is beyond the scope of this paper to explain all detail about the stochastic subspace identification (SSI) method. The interested reader may refer to literature: (Van Overschee and De Moor 1996; Abu-Aisheh 2003). In this paper only the main topics are explained.

It is well-known that a structural model can be described by a set of linear second-order differential equations with constant coefficient:

$$\mathbf{m}\ddot{\mathbf{z}}(t)+\mathbf{c}\dot{\mathbf{z}}(t)+\mathbf{k}\mathbf{z}(t)=\mathbf{f}(t) \tag{4}$$

Equation (4) expresses force equilibrium between external and internal forces. The bold face will be used to highlight a vector or matrix notation. $\boldsymbol{z}(\boldsymbol{t})$and $\boldsymbol{f}(\boldsymbol{t})$ are the $\boldsymbol{p}\,\boldsymbol{x}\,\mathbf{1}$ displacement and force vectors at the mass point, respectively. All other matrices have the dimension $\boldsymbol{p}\,\boldsymbol{x}\,\boldsymbol{p}$. **m** is a diagonal and positive definite mass matrix. The dissipation of energy is modeled by the symmetric and positive semi-definite damping matrix **c**. The stiffness matrix **k**, is symmetric and positive definite. The system is assumed time-invariant, since **m**, **c** and **k** do not depend on time. The acceleration, velocity and displacement responses are denoted as $\ddot{\boldsymbol{z}}$, $\dot{\boldsymbol{z}}$ and $\boldsymbol{z}$, respectively. One commonly used reformulation in a state-space representation is:

$$\dot{\mathbf{x}}(\mathrm{t})=\mathbf{A}\mathbf{x}(\mathrm{t})+\mathbf{B}\mathbf{u}(\mathrm{t}) \tag{5}$$

The matrix **A** is the state matrix, whereas the matrix **B** is the input matrix. The system equations are given by the following:

$$\mathbf{A}=\begin{bmatrix}\mathbf{0} & \mathbf{I}\\ -\mathbf{m}^{-1}\mathbf{k} & -\mathbf{m}^{-1}\mathbf{c}\end{bmatrix},\qquad \mathbf{B}=\begin{bmatrix}\mathbf{0}\\ \mathbf{m}^{-1}\end{bmatrix},\qquad \mathbf{u}(t)=\begin{bmatrix}\mathbf{f}(t)\\ \mathbf{0}\end{bmatrix} \tag{6}$$

The system response of the state space system can be observed through the observation vector $\boldsymbol{y}(\boldsymbol{t})$. This response depends on the type of measurements and the process to be measured. In the case of civil engineering structures, the response will typically be displacements, velocities or accelerations.

Furthermore, the output vector of interest $\boldsymbol{y}(\boldsymbol{t})$, can be a part of, or a linear combination of system states, such as

$$\mathbf{y}(t)=\mathbf{C}\mathbf{x}(t)+\mathbf{D}\mathbf{u}(t) \tag{7}$$

Here **C** is a real output, observation or measurement matrix and **D** is a direct term matrix. Equations (5) and (7) constitute a continuous-time state-space model of a dynamic system. An ARMAV (n, n) model can be represented in a state space realization as:

$$\begin{aligned}&\mathbf{x}(t_{k+1})=\mathbf{A}\mathbf{x}(t_k)+\mathbf{B}\mathbf{u}(t_k),\qquad \mathbf{u}(t_k)\in\ NID(\mathbf{0},\boldsymbol{\Delta})\\ &\mathbf{y}(t_k)=\mathbf{C}\mathbf{x}(t_k)+\mathbf{D}\mathbf{u}(t_k)\end{aligned} \tag{8}$$

where NID(0, Δ) denotes normally independently distributed with zero mean and variance Δ .

In general, if noise is present, both state and observation equation will be affected. It will be assumed that both the process noise and the measurement noise can be described by a zero-mean Gaussian white noise processes denoted $\mathrm{w}(t_k)$ and $\mathrm{v}(t_k)$ respectively. The most common approach for analyzing the effect of such noise is based on probabilistic or statistical models, where random elements are added to the input and output components. The resulting stochastic system has the form:

$$\mathrm{x}(t_{k+1})=\mathbf{A}\mathrm{x}(t_k)+\mathbf{B}\mathrm{u}(t_k)+\mathrm{w}(t_k),\qquad \mathrm{x}(0)=\mathrm{x}_0\ :\ \textit{Process} \tag{9a}$$

$$\mathbf{y}(t_k)=\mathbf{C}\mathbf{x}(t_k)+\mathbf{D}\mathbf{u}(t_k)+\mathbf{v}(t_k)\qquad :\ \textit{Measurement} \tag{9b}$$

It is difficult to determine accurately the individual process and measurement noise characteristics and therefore some assumptions are required. Here the process noise $w(t_k)$and measurement noise $v(t_k)$ are assumed to be of zero-mean, white and with covariance matrices.

In the practical cases of civil engineering structures, the reality is that only the responses of a structure are measured, while the input sequence u_k remains unmeasured. In the case of ambient vibration testing, it is impossible to distinguish the input term $u(t_k)$ from the noise terms $w(t_k)$, $v(t_k)$ in Equation (9). Modeling the input term $u(t_k)$ by the noise terms $w(t_k)$, $v(t_k)$ results in a purely stochastic system:

$$\mathrm{x}(t_{k+1})=\mathbf{A}\mathrm{x}(t_k)+\mathrm{w}(t_k), \tag{10a}$$

$$\mathrm{y}(t_k)=\mathrm{C}\mathrm{x}(t_k)+\mathrm{v}(t_k) \tag{10b}$$

Equation (10) constitutes the basis for the time-domain system identification through ambient vibration measurements.

There have been several techniques to realize system identification algorithms based on Equation (10). The SSI algorithm is probably the most advanced method known up to date for ambient vibration measurement system identification. The subspace method identifies the state space matrices based on the measurements and by using

robust numerical techniques such as QR-factorization, Prediction Error Method (PEM), singular value decomposition (SVD) and least squares. Once the mathematical description of the structure (the state space model) is found, it is straightforward to determine the modal parameters (natural frequencies, damping ratios and mode shapes) by eigenvalue decomposition (see Ren and Zong 2004).

RESULTS AND DISCUSSION

Average Spectral Densities

The Averaged Spectral Densities (ASD) will give a rough idea about the number of modes present in the data and the level of noise it may contain. Consequently, the ASD for the car data were averaged separately from the truck data for bridge #617 and then all car and truck ASD were averaged together to investigate the effect of heavy vehicles on modal parameters for later study not included in this paper. To avoid the effect of heavy vehicles on modal parameters, a decision was made to use only "car" data records in the system identification. A typical comparison of ASD for the entire set of "car" data (all setup sequences) of all three test stages is depicted in Figure 4.

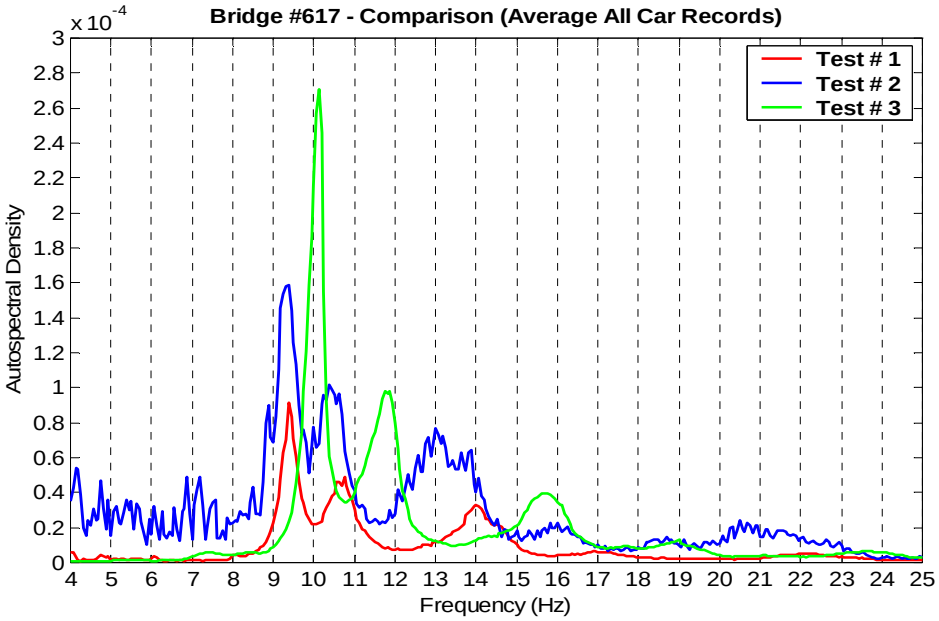


Figure 4. Comparison of Auto spectral densities of - Test #1, Test #2 and Test #3 (using FFT)

Experimental Results

Almost all modes predicted by the FEM model were definitely identified with their associated estimates of uncertainties using the ARMAV technique with the PEM as a tool for parameter estimates. However, in using the FFT-SEMA technique, modes were identified on the basis of the autospectral peaks of Figure 4 and mode shapes were chosen based upon those corresponding to the least "noisy" of the operational deflected shapes in the peak frequency region. Also, damping estimates associated with each mode together with their uncertainties were identified using the ARMAV technique, whereas, damping estimates were not possible to produce using the frequency domain SEMA method.

The preliminary FEM models were "tuned" in the light of the results obtained from the dynamic testing program to produce a more realistic representation of Bridge #617 and its in-service performance. Results for the dominant modes are summarized in Table 1 and mode shapes are depicted in Figure 6.

The studies performed clearly demonstrated that the new "spraycrete" bandbeams were effective in substantially increasing the general stiffness of the deck (by approximately 250%) whereas the original bandbeams were much less effective in doing so (less than approximately 10% stiffness increase).

Model Validation

The adequacy of the models is illustrated by plotting the spectral densities obtained directly from measurements using the FFT together with the spectral densities obtained from the ARMAV models for individual accelerometer positions. Figure 7 shows a comparison of autospectral densities resulting from FFT based analysis and ARMAV models for one of the fixed accelerometers M for all three-test configurations.

Table 1. Comparison of the revised FEM Models with Experimental Estimates of Modal Frequencies using SEMA and ARMAV methods for the three test configurations of Bridge #617. (modes shown bolded are most dominant in the response measurements). (Key: F: Flexural; T: Torsional; P: Plate-like)

Mode #	TEST #1 Freq. (Hz)			Mode #	TEST #2 Freq. (Hz)			Mode #	TEST #3 Freq. (Hz)		
	FEM	SEMA	ARMAV		FEM	SEMA	ARMAV		FEM	SEMA	ARMAV
1 - F	**9.45**	**9.38**	**9.44**	**1 - F**	**9.50**	**9.38**	**9.42**	**1 - F**	**9.88**	**9.88**	**10.08**
2 - F	**11.23**	**10.63**	**10.77**	**2 - F**	**10.79**	**10.63**	**10.61**	**2 - F**	**12.13**	**11.69**	**11.77**
3 - T	**13.34**	**13.81**	**13.94**	**3 - T**	**12.49**	**12.94**	**12.79**	**3 - T**	**15.69**	**15.63**	**15.73**
4 - F	14.16	-	14.08	4 - F	12.98	-	13.05	4 - F	16.12	-	16.16
5 - T	14.38	14.81	14.47	5 - T	13.61	14.25	14.07	5 - T	16.30	16.63	16.47
6 - T	**16.93**	**17.63**	**17.29**	**7 - F**	**15.59**	**16.38**	**16.24**	**6 - F**	**20.05**	**19.0**	**19.11**
7 - F	17.45	-	-	6 - F	15.11	-	-	6 - F	20.78	-	20.92
8 - T	19.10	18.75	18.47	8 - T	17.14	-	-	8 - T	22.77	-	22.37
9-sway	19.63	-	20.20	9 - T	17.89	20.06	18.57	-	-	-	-
10 - P	**21.49**	**22.06**	**22.19**	**11 - P**	**20.36**	**21.69**	**21.71**	**10 - P**	**24.09**	**23.63**	**23.75**

Modal Assurance Criteria (MAC) values were obtained to compare mode shapes obtained using ARMAV technique among themselves and also among SEMA technique. Generally, it is found that a MAC value in access of 0.9 (or 90%) should be attained for well-correlated modes and a value of less than 0.1 (or 10%) for uncorrelated modes. In some situations, depending on the application and test circumstances, the boundaries for "acceptably correlated" and "non-correlated" are quoted as above 80% and less than 20% (Ewins 2000). The MAC values calculated for the five most dominant modes of Test #1 obtained using ARMAV technique to quantify the correlation between them and those obtained using SEMA technique are presented graphically in Figure 5. It is clear from Figure 5 that the modes obtained using the ARMAV technique are an "excellent" match to those obtained using SEMA method.

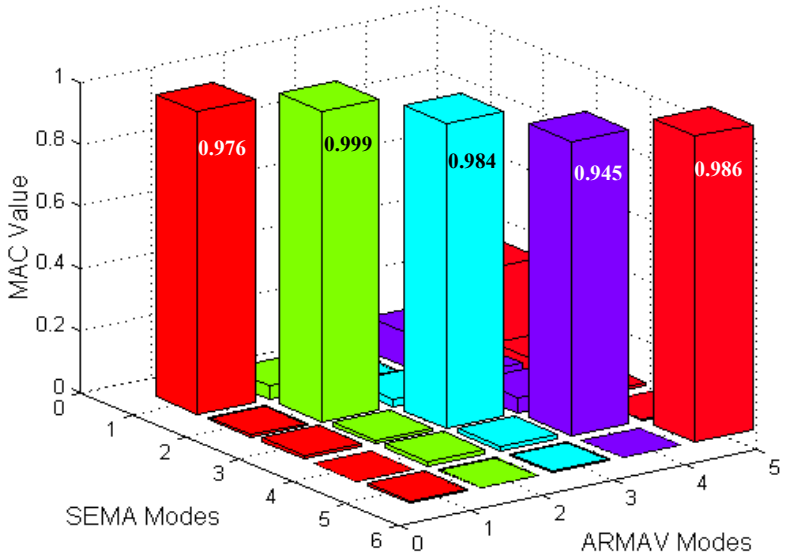


Figure 5. MAC Stability Diagram - ARMAV model with those obtained using SEMA technique - Test #3

In Figure 8, the MAC stability diagram for the mode shapes of Test 3 is presented. Modes with a damping ratio of less than 8% and a MAC value of at least 95% are marked with a diamond (◊) in the stability diagram; otherwise the mode is not depicted. The location of the diamond will be at the coordinate indicating the number of the current model and the natural frequency corresponding to the mode shape.

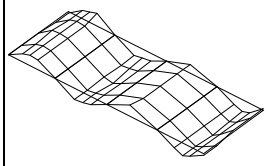
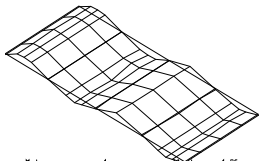
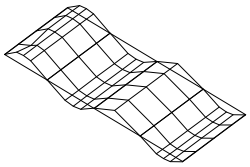
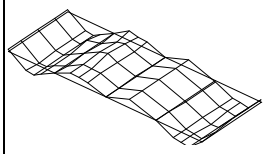
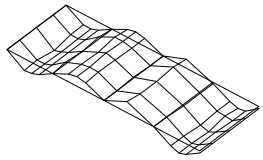
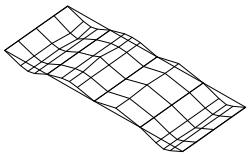
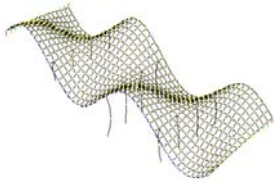
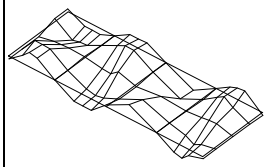
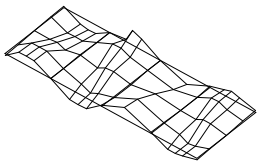
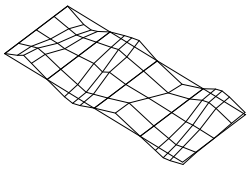
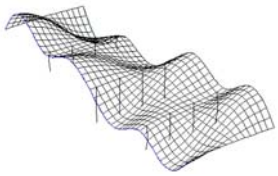
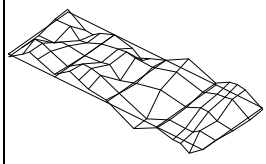
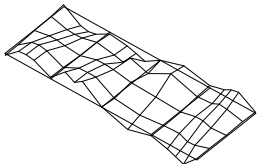
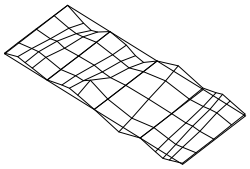
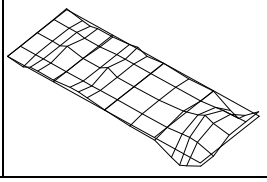
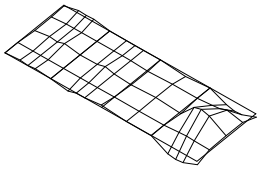
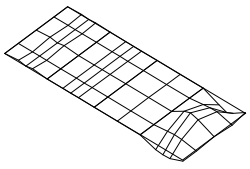
	FEA Models	ARMAV – Test 1	ARMAV – Test 2	ARMAV – Test 3
	f1,2,3= 9.38, 9.50, 9.88Hz	*f = 9.44Hz, ζ = 2.99*	*f = 9.42Hz, ζ = 1.73*	*f = 10.08Hz, ζ = 2.62*
Mode #1				
	$f_{1,2,3}$=11.23, 10.79, 12.13Hz	*f = 10.77Hz, ζ* = 3.73	*f = 10.61Hz, ζ* = 3.59	*f = 11.77Hz, ζ* = 4.99
Mode #2				
	$f_{1,2,3}$=13.34, 12.49, 15.69Hz	*f = 13.94Hz, ζ* = 2.91	*f = 12.79Hz, ζ* = 4.48	*f = 15.73Hz, ζ* = 3.54
Mode #3				
	$f_{1,2,3}$=16.93, 15.59, 20.05Hz	*f = 17.29Hz, ζ* = 3.92	*f = 16.24Hz, ζ* = 4.98	*f = 19.11Hz, ζ* = 2.79
Mode #4				
	$f_{1,2,3}$=21.49, 20.36, 24.09Hz	*f = 22.19Hz, ζ* = 2.43	*f = 21.71Hz, ζ* = 2.01	*f = 23.75Hz, ζ* = 5.52
Mode #5				

Figure 6. Comparison of results for most dominant modes obtained using the "tuned" FEM against those obtained using the ARMAV model. (MRWA Bridge #617 (Test 1 → 3), $f_{1,2,3} \rightarrow f_{Test\ 1}$, $f_{Test\ 2}$, $f_{Test\ 3}$)

CONCLUSIONS

A number of conclusions can be drawn from this experimental study, as summarized below:

(1) Dynamic testing, in combination with FEM model updating, is a powerful technique for performing SSI.

(2) ARMAV and SEMA algorithms, were both able to be applied to frequency domain data, both algorithms can exhibit comparable accuracy for determining dynamic modal characteristics of structures from acceleration response measurements.

(3) The most accurate results were obtained with the SSI method based on ARMAV modeling ─ that makes use of output-only vibration response data in the time domain. With this method also closely spaced modes can be separated.

(4) Reasonably fine measurement grids of vibratory response would be necessary for localizing damage from algorithms applied to experimentally obtained modeshapes

The role of Experimental Modal Analysis as a useful investigative tool for evaluating structural system performance, whether the structure is damaged or undamaged, has been exemplified in this study. Experimental Modal Analysis and its application to civil structures, such as bridges, remains a fruitful area for research.

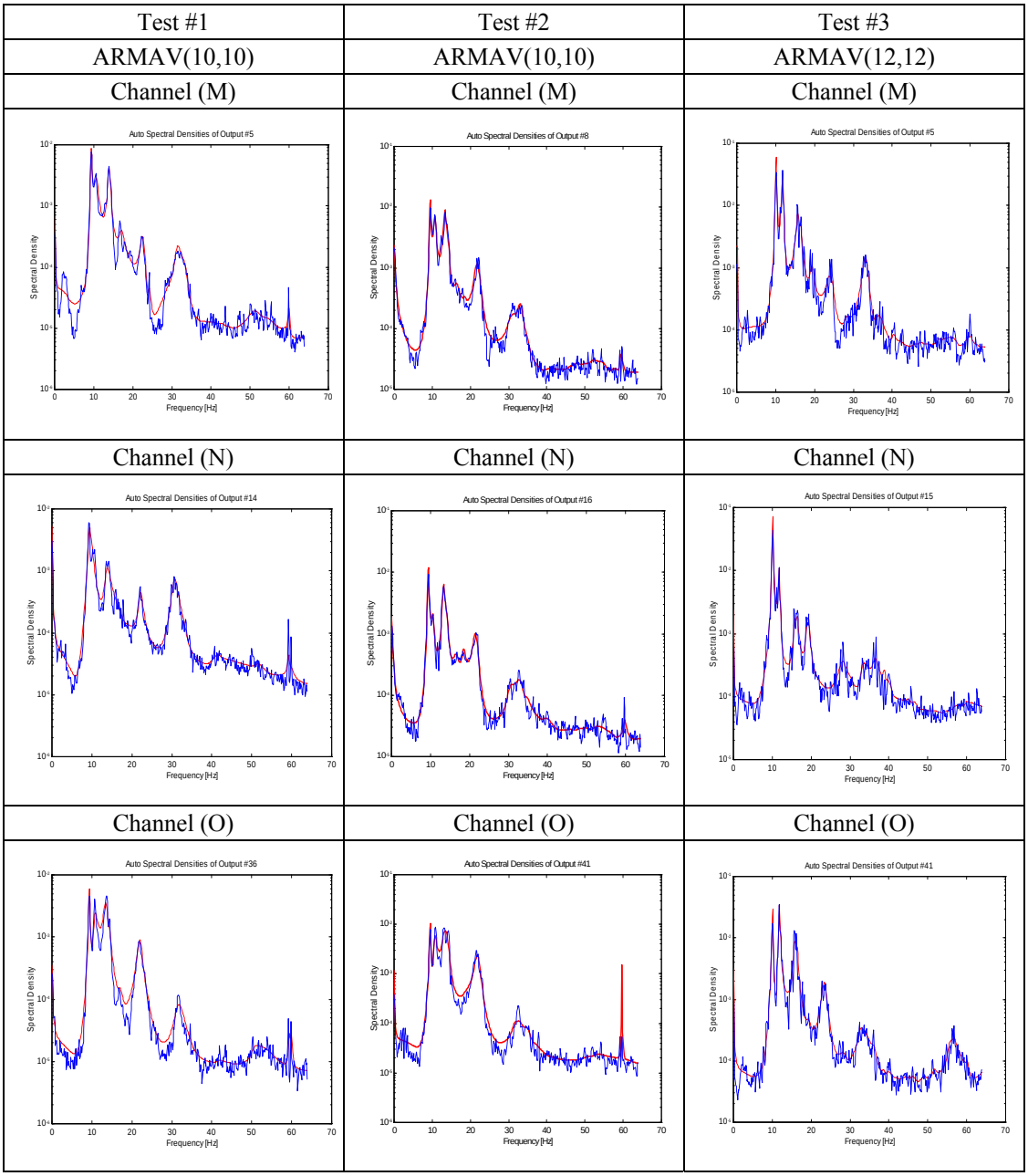


Figure 7. Comparison of typical autospectral densities obtained using FFT (blue) and the estimated ARMAV model (red) for the three fixed accelerometer positions (M, N and O) and one roving accelerometer position (J) - All Test conditions

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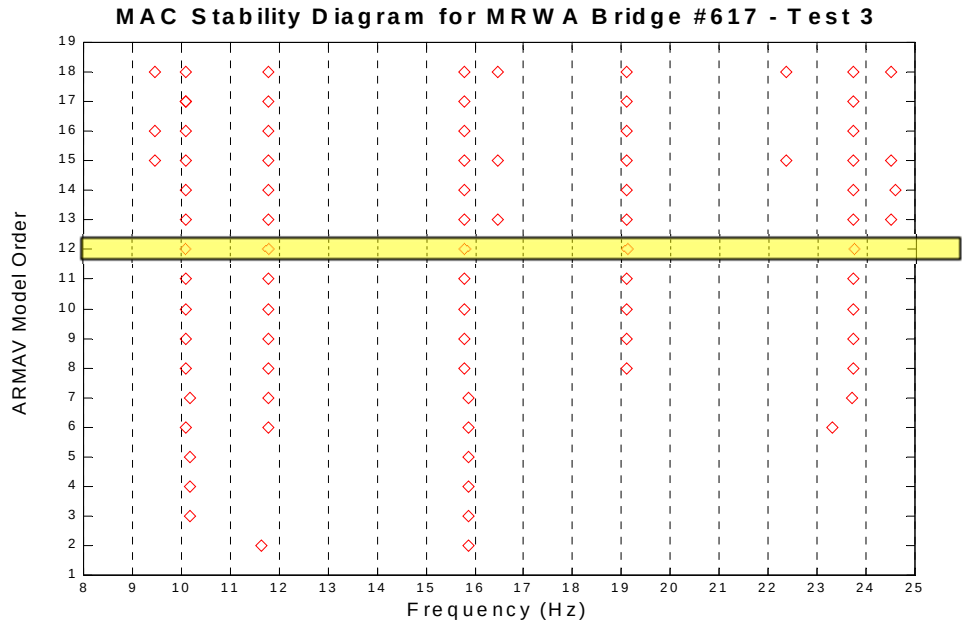

Figure 8. MAC Stability Diagram – Test 3. (Highlighted model order is the correct model)

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