Dynamic properties of an eleven-span motorway bridge at different levels of excitation

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ABSTRACT: Bridge dynamic properties measured under a given vibration intensity condition would give a true picture of the behaviour for that particular condition. However, the use of the model derived from such data may not be reliable when applied for the prediction of response under a different vibration intensity condition. Therefore, it is necessary to investigate the structural dynamic behaviour at different levels of excitation in detail. This paper focuses on the experimental investigation of modal property variability at different levels of excitation. Both weak ambient vibration tests (induced by nearby traffic, wind and possibly microtremors) and forced vibration tests with different applied input force induced by eccentric mass shakers were performed on the Nelson St off-ramp bridge (an 11-span post-tensioned concrete, box girder structure forming a part of the motorway network in Auckland's CBD). Three separate system identification methods, namely peak-picking (PP), the frequency domain decomposition (FDD) and the data-driven stochastic subspace identification (SSI) method, were applied for accurate structural modal parameter identification. It was found that the three output only identification techniques are able to extract natural frequencies of the structure reliably, while the time domain SSI method yields the best mode shape estimates and PP may not be able to give accurate mode shape estimates for some modes. The variability of the dynamic properties for the 1st vertical and lateral bending modes was examined. A general trend of decreasing natural frequencies and increasing damping ratios was observed with increased level of vibration intensity.

1 INTRODUCTION

Dynamic properties of bridge structures, i.e. natural frequencies, damping ratios and mode shapes, are of significant importance in accurately predicting the dynamic response of structures at the design and reassessment stage. A majority of dynamic analysis procedures are developed around the assumption that the structure systems studied are time-invariant, and linear-elastic. However, it has been repeatedly demonstrated that these assumptions are disputable for many applications, and even small amplitude excitations can bring out non-linear features of the system (Trifunac et al. 2001; Butt and Omenzetter 2014; Butt and Omenzetter 2014), leading to erroneous analytical results. Therefore, it is necessary to investigate the structures dynamic behaviour at different levels of excitation in detail. For bridge structures, Shepherd and Charleson (1971) investigated the relationship between the eccentric weight of mechanical large shaker and the natural frequency of the tested abutment of a multi-span continuous deck bridge at the construction stage. Farrar et al. (2000) noted that the modal frequencies and mode shapes extracted from different vibration intensity tests on the Alamosa Canyon bridge were almost consistent, but there were significant changes in the damping ratios which were correlated with excitation amplitude. Fujino et al. (2000) observed that the fundamental frequency of a suspension bridge reduced as the wind speed increased. Zhang (2002) found the natural frequencies of the studied cable-stayed bridge can exhibit as much as 1% variation within a day due to different vibration intensity under normal traffic operation conditions. The damping ratios however are sensitive to the vibration intensity, especially when the deck vibration exceeds a certain level. These researches help us gain insight into the vibration intensity related dynamic behaviour of bridge structures preliminary.

However, a comprehensive and systematic exploration for the magnitude-dependent dynamic property of a bridge system is still inadequate, due to the relative lack of adequate field testing data on their dynamic behavior at different levels of vibration intensity, especially for long, multi-span, short-span highway/motorway concrete bridges. Another aspect is that significant uncertainties involved in vibration testing such as frequently unavoidable measurement noise, and this may hinder a more routine adoption of the structure identification results in support of bridge operational and maintenance management decisions. Thus, different parameter identification techniques need to be implemented to verify the reliability of the identified results and provides a bridge owner more confidence in using the identified results for decision making.

This paper is intended to check the variability of dynamic properties of a typical eleven-span motorway bridge at different levels of excitation. A series of varying input force within a relative wide range was applied to the bridge, which was generated by using dual-arm contra-rotating eccentric mass shakers. Three separate structure identification methods, including peak-picking (PP) (Bendat and Piersol 1993), the frequency domain decomposition (FDD) (Brincker et al. 2000) and the data-driven stochastic subspace identification (SSI) (Van Overschee and De Moor 1996), were implemented. Correlation analysis among these methods was carried out to reveal their advantage and disadvantage of dealing with in situ testing data contaminated by noise. The variability of dynamic property for 1st vertical and lateral bending mode was addressed preliminarily. A general trend of decreasing natural frequencies and increasing damping ratios was observed with increased level of vibration intensity. The research result is expected to assist in evaluating current design and analytical assumptions.

2 DESCRIPTION OF THE BRIDGE

The in situ dynamic testing was undertaken on the Nelson St off-ramp bridge (Figure 1), which links the Northern Motorway to Auckland's port and North-Western Motorway. The construction of the structure includes a total of 137 pre-cast concrete beams. These were delivered to the site and placed in their final position. Once in place, construction of the superstructure work and capping beams began using the moveable scaffold system. The bridge is a horizontally curved, post-tensioned, continuous concrete structure with a hollow box section girder. An elevation sketch showing span lengths and pier heights is displayed in Figure 2.

Figure 1. Views of the Nelson St off-ramp bridge

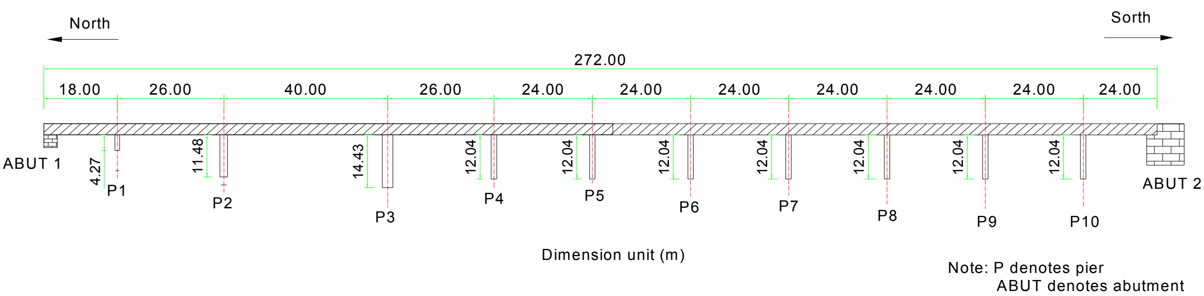


Figure 2. Elevation sketch of the bridge

3 INSITU DYNAMIC TESTING AND MODAL IDENTIFICATION

3.1 Testing at different vibration levels

Modal testing of a bridge on site provides an accurate and reliable prediction of its real dynamic characteristics, whilst the most comprehensive and accurate method remains forced vibration test (FVT) using a mechanical exciter to apply a harmonically varying force of a known frequency and amplitude to the structure. In the present study, two ANCO MK-140-10-50 eccentric mass exciters were deployed on the bridge deck to perform frequency sweeps in both the vertical and lateral direction. The amplitude and frequency of the applied force can be controlled by varying the rotational speed and the magnitude and eccentricity of the attached masses. The force output generated by each shaker, P, is expressed as:

$$P(t) = MR\omega^2 \sin \omega t \qquad (1)$$

where MR is total shaker mass eccentricity (kg-m), and ω is the circular frequency of rotation in radian/sec.

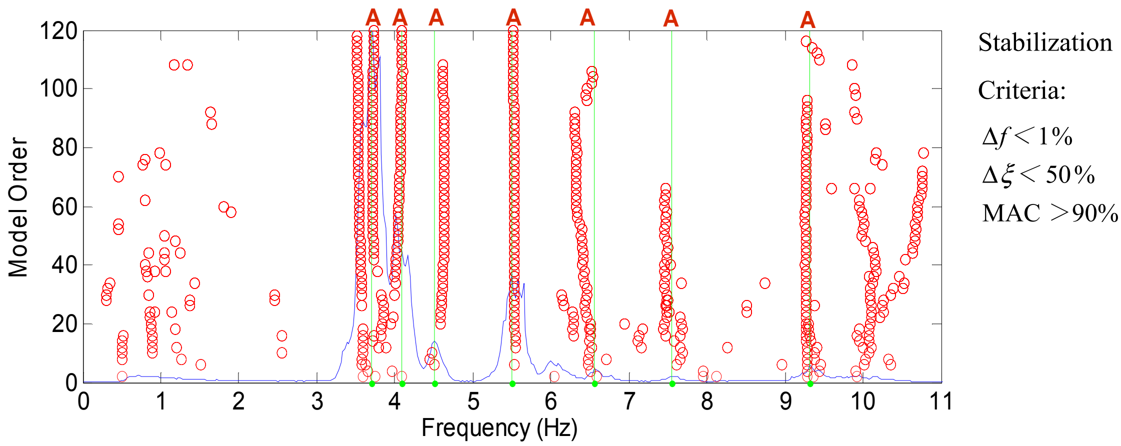
By changing the configuration and number of the steel masses attached to the flywheels of the shaker (Figure 3a), different amplitude levels of harmonic excitations were generated. Taking advantage of this, the modal property variability of the bridge structure at different levels of excitation was investigated. The basic procedure of the frequency sweep was as follows. Quick frequency sweep (0-10 Hz, 0.1 Hz increment, 15 seconds holding time at each frequency point, 1 small steel mass) were conducted to roughly determine the natural frequencies on site. Subsequently, a series of detailed frequency sweeps were performed in the vicinity of the natural frequencies identified from preliminary tests within around 0.5 Hz frequency varying zone, this time with a smaller increment (0.01 Hz) and using different numbers of steel masses. At every frequency point, the forcing was held constant for approximately 60 seconds to allow the bridge response to achieve a steady state and record this steady state response. Because of the torque limit of the vertical shaker, the maximum number of steel masses that could be installed was 1 big mass (15.5 kg) plus 1 small mass (3.6 kg). Up to eight big masses per shaft were gradually installed on the horizontal shaker during lateral sweeping tests. Based on a preliminary finite element analysis, the longest span between Pier 2 and 3 was selected to mount the shakers. The horizontal shaker was positioned at the mid-span along the midline of the bridge deck and the vertical shaker at 1/3 of the span length towards the West traffic lane so as to also excite possible torsional modes. During the tests, wireless 3-axial, stand-alone MEMS accelerometers (Beskhyroun and Ma 2012) were utilized to capture the vibration response of the structure at a user selectable rate of 80 Hz. The wireless USB type accelerometer/battery units were wrapped tightly onto small timber blocks. This measuring unit was attached to the selected measuring points on the bridge deck by using silicone adhesive to ensure good contact between the accelerometer and the bridge (Figure 3b). Prior to FVT, a weak ambient vibration test (AVT) was conducted, and the excitation sources mainly came from vehicles traveling on the motorway sections adjacent and underneath the bridge, wind. The accelerometers were the same as those used in the FVT and arranged along both bridge curbs with 2m to 4m distance between two measurement stations. 20mins vibration measurement was recorded with 160 Hz sampling rate at all accelerometers.

a)  b) 

Figure 3. Testing equipment: a) ANCO MK-140-10-50 eccentric mass exciter, and b) MEMS accelerometers

3.2 Operational modal analysis

The extraction of modal parameters from in situ dynamic testing was carried out by using three different techniques: PP, FDD and SSI methods. The PP is considered to be the simplest and the most popular method used in civil engineering to estimate the modal parameters of a structure. The method is based on the identification of the natural frequencies from the peaks of power spectrum estimates. Mode-shape components are then determined by the values of the transfer functions at the natural frequencies. The FDD method is based on the evaluation of the output spectral matrix. After this, the singular value decomposition of the spectral matrix is conducted. The obtained singular values are related with the natural frequencies, while singular vectors represent the corresponding mode shapes. Both PP and FDD techniques are implemented in the frequency domain, while SSI is a time domain method that directly works with time data, without the need to convert them to correlation functions or spectra. The data driven SSI algorithm extracts a system model in the state space using the measured output data directly. After the identification of the state space model, the modal parameters are obtained from the system matrices using eigenvalue analysis. The three methods referred to above are amongst the most widely used methods in civil engineering applications at the current state of the art and were applied to the Nelson St off-ramp bridge data for comparison of their performance. The data processing for modal identification was carried out by a modal parameter identification toolbox developed at the University of Auckland for civil engineering applications (Beskhyroun 2011). Figure 4 shows the stabilization diagram for AVT data showing identified stable frequencies.



Blue line: PP power spectrum; Green line: FDD identified frequencies; Red circles: SSI stable poles, A: identified frequencies

Figure 4. Stabilization diagram for AVT data.

4 COMPARISON OF PP, FDD AND SSI

Table 1 shows the identified natural frequencies and damping ratios from field testing data based on PP, FDD and SSI for AVT and FVT, respectively. The labels V and L stand for vertical bending and lateral bending mode, respectively. (Note not all modes were identified from AVT). The identified frequencies agree well among these three techniques. It is demonstrated that natural frequencies can be extracted from field dynamic testing reliably. A small difference of the identified natural frequencies between AVT and FVT can be observed, possibly due to a frequency-response amplitude relationship or testing environmental temperature variation etc. Damping ratios of between 0.4% and 2.6% were identified using SSI alone. These damping ratios are broadly in the range expected for concrete bridges. Differences between AVT and FVT damping results are clearly visible but are not larger than commonly encountered in experimental modal analysis. Figures 5 shows parts of identified mode shapes from AVT and FVT based on PP, FDD and SSI. Overall, a good agreement can be observed among the three methods, which means that the identified results have a relatively high reliability. However, the FDD and SSI yield more mutually similar mode shape estimates and PP cannot give good mode shape estimates for some modes, which can be clearly observed in Figures 5 c-l. In terms of the mode shape comparison between FDD and SSI, it can be observed from AVT that the estimates by SSI are more accurate than those by FDD, since the identified mode shape curves from SSI are typically much smoother than those from FDD. Especially for the 3L mode (Figure 5a, j), SSI gave much better identified results without the discontinuity seen in the FDD results. It can be concluded that SSI has stronger ability to capture the latent signal characteristics in a noisy environment. On the other hand, for FVT both algorithms performed well and gave consistent mode shape identification results for the majority of modes, since the FVT data had a much higher signal-to-noise ratio compared to the AVT data due to the greater excitation force level.

Table 1. Modal parameters obtained with PP, FDD and SSI.

Mode	Natural frequency (Hz)						Damping ratio (SSI) (%)	
	AVT			FVT (one small mass)			AVT	FVT (one small mass)
	PP	FDD	SSI	PP	FDD	SSI		
1V	3.17	3.17	3.22	3.20	3.17	3.18	1.8	1.1
2V	3.83	3.83	3.82	3.83	3.87	3.91	1.4	1.5
3V	—	—	—	4.14	4.18	4.19	—	0.5
4V	—	—	—	4.77	4.77	4.79	—	1.5
5V	—	—	—	5.63	5.66	5.66	—	2.1
6V	—	—	—	7.11	7.15	7.15	—	1.6
7V	—	—	—	7.89	7.93	7.92	—	1.8
1L	—	—	—	1.88	1.88	1.86	—	0.4
2L	—	—	—	2.58	2.54	2.56	—	0.5
3L	3.72	3.72	3.77	3.67	3.63	3.65	1.2	1.0
4L	4.48	4.50	4.46	4.45	4.53	4.54	1.3	1.1
5L	5.57	5.46	5.47	5.55	5.55	5.57	2.1	1.5
6L	6.59	6.64	6.63	6.64	6.64	6.61	1.2	1.9
7L	7.53	7.56	7.50	7.54	7.54	7.61	2.4	2.6
8L	9.32	9.37	9..38	9.38	9.38	9.32	2.5	1.3

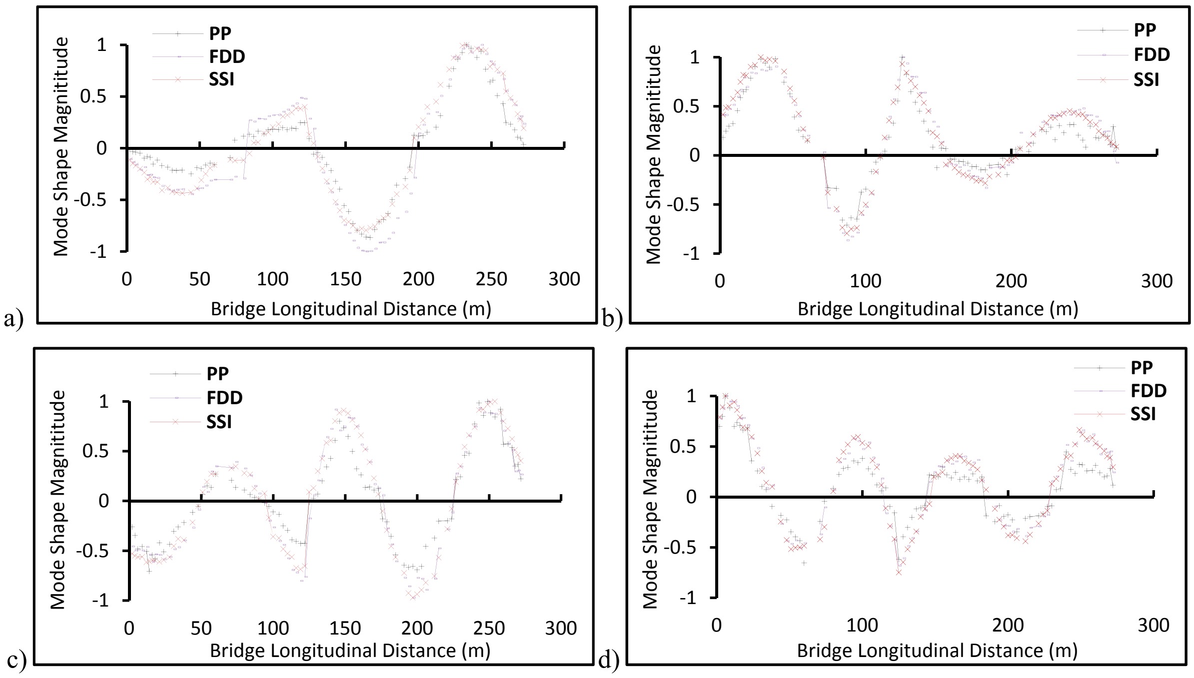


Figure 5. Mode shape comparison between PP, FDD and SSI: AVT a) 3L, b) 4L, c) 5L, d) 6L, e) 7L, and f) 8L; FVT g) 2V, h) 3V, i)7V, j) 3L, k) 5L, and l) 7L

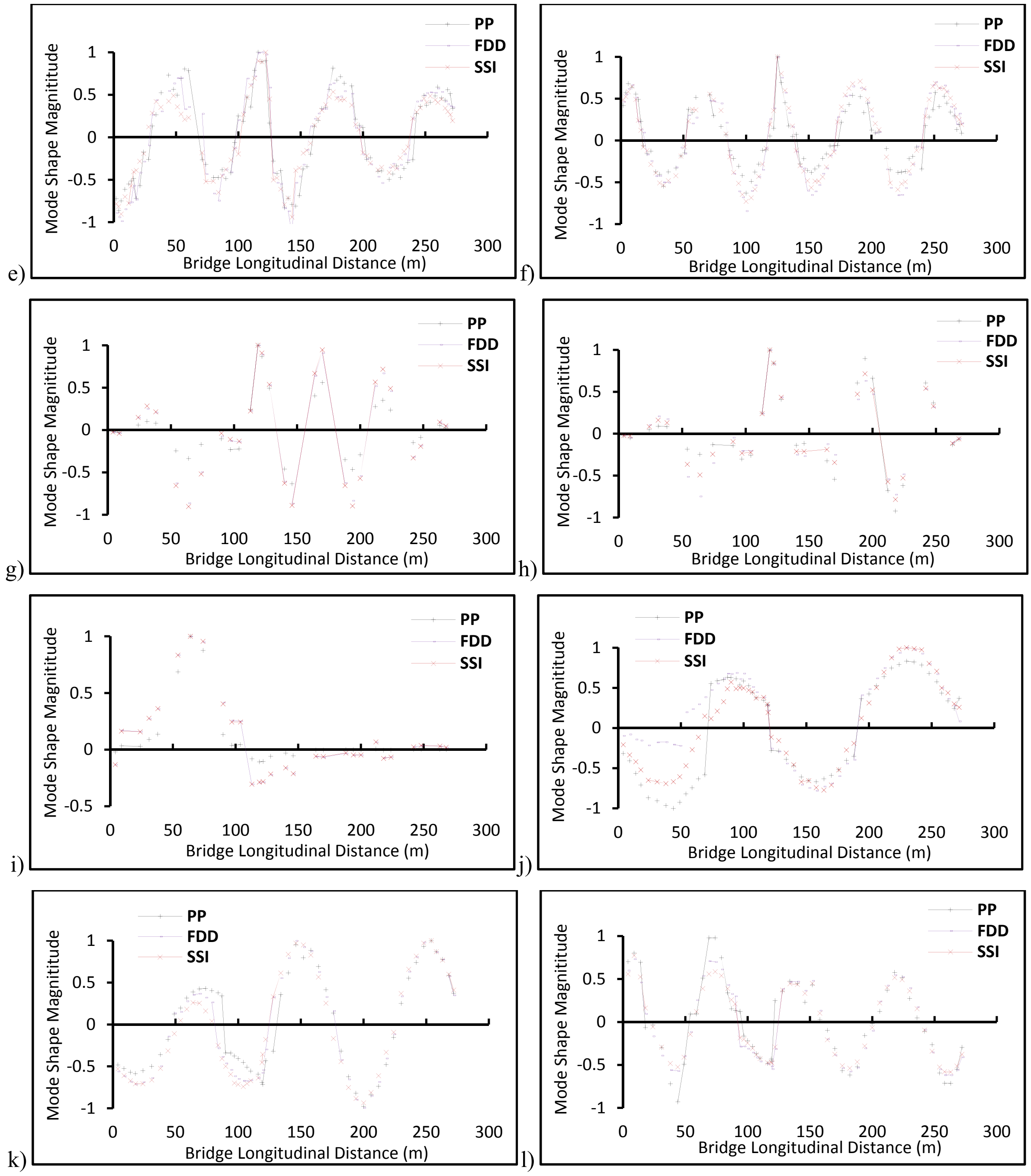


Figure 5. cont.

5 MODAL PARAMETER VARIABILITY OF THE BRIDGE STRUCTURE

In this section, modal property changes with the change of excitation level were investigated for the fundamental modes of the structure, i.e. the 1st vertical and lateral bending mode. The estimate of natural frequencies and damping ratios was based on PP, i.e. by examining the peaks of the normalized displacement response curves for resonant frequency and using the well-known half-power method (Chopra, 1995) for damping. Figure 6 shows the change in natural frequency and damping of the 1st vertical and lateral bending mode. Generally, it can be observed that the identified natural frequencies decrease with the increase of external excitation, while the identified damping ratios have an opposite tendency. Note that the natural frequencies and damping ratios have a much bigger difference with the loading change from one small mass loading to one big mass loading. Furthermore, from Figure 6c it can also be observed that the decreasing rate of the natural frequency for 1L become slower gradually with the increasing mass loading, which means the deterioration of structural stiffness slows down. The decline of the natural frequency with the rise of the shaking magnitude can be explained in terms of structural stiffness deterioration due to the effects of material and structural nonlinearities, cracking, yielding and influence of non-structural element etc. From Figure 6d it can be seen the damping ratio of 1L only rise at the beginning, while remaining almost constant for one big mass to eight big masses. It is speculated the initial increment of lateral mode damping mainly came from the friction of bearing joints mobilization. One big mass loading level has mobilized the bearing friction movements fully, and no additional energy dissipation is available despite the higher mass loading levels afterwards. The variability of mode shape has also been checked, but no obvious difference can be observed.

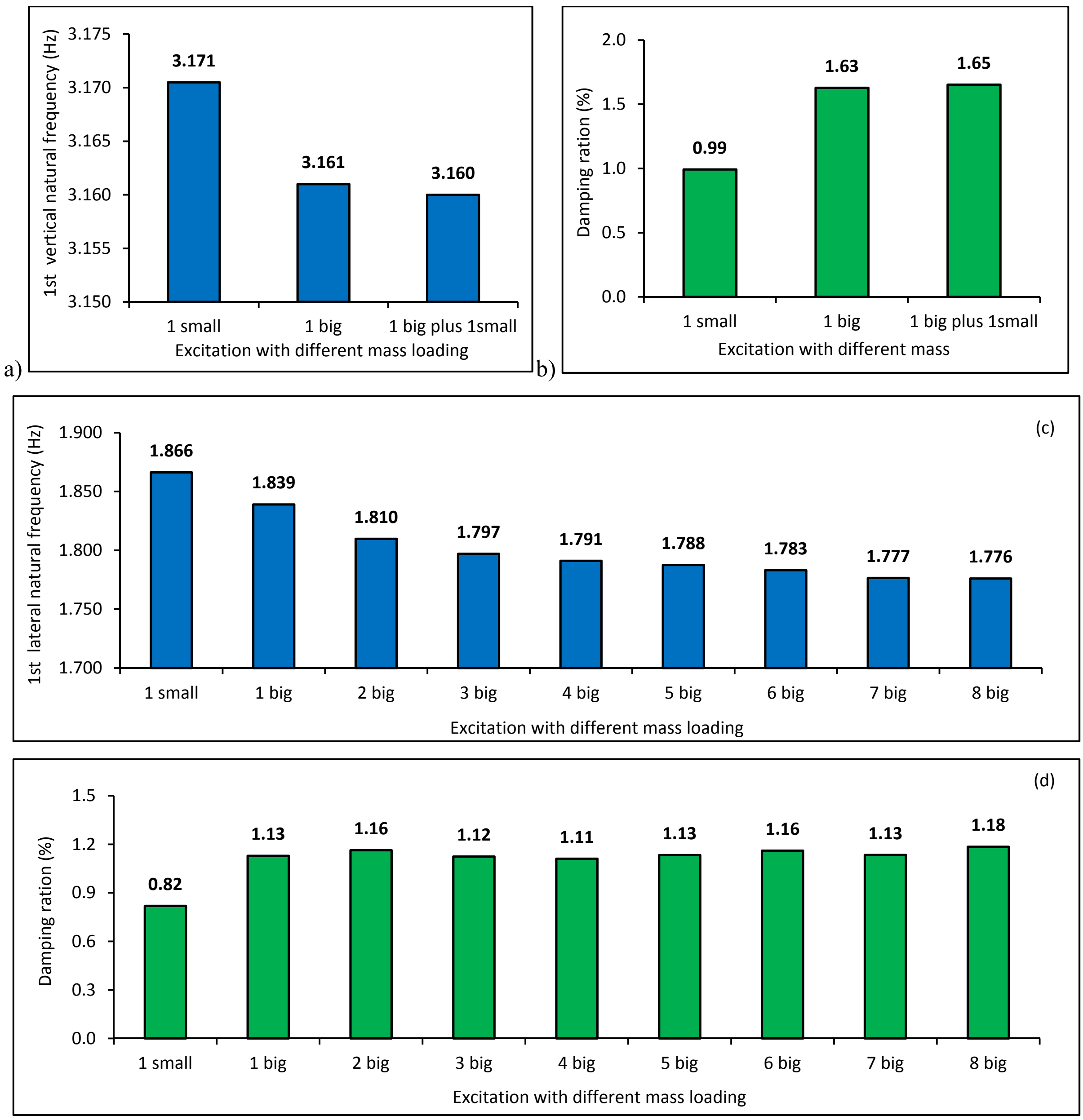


Figure 6. Modal paramter variability: a) 1V natural freuqecy, b) 1V damping, c) 1L natural freuqncy, and d) 1L damping

6 CONCLUSIONS AND FUTURE WORK

In this paper, both weak AVT and FVT with different applied input force induced by rotating eccentric mass shakers were performed on a 11-span contiguous concrete bridge, with a focus on checking the variability of dynamic properties of the structure. The following conclusions can be drawn based on the presented investigations:

1. Reliable natural frequency identification from field dynamic testing data can be achieved from either the frequency domain methods PP and FDD or the time domain method SSI. However, it is recommended to use PP on site to judge the overall dynamic characteristics of the structures quickly because of its fast natural frequency identification speed.
2. The time domain method SSI yields the best mode shape estimates among the threes methods and is more robust for dealing with in-situ dynamic testing data contaminated by noise. It is suggested that SSI be applied to carry out detail analysis to obtain mode shapes when one comes back to the office, since the computational effort of the SSI technique is significantly higher than PP or FDD.
3. The dynamic property variability was checked for 1st vertical and lateral bending mode, and the general trend found is that natural frequencies decrease with the increase of external excitation amplitude, while the damping ratios have the opposite tendency with the damping ratios initially increasing but later remaining constant. Both the frequency and damping have a much bigger change with the loading change from one small mass to one big mass.

Future investigations will involve checking the variability of dynamic properties of the bridge for higher modes and comparing their changes with the lower modes. In addition, a quantitative relationship between displacement/acceleration amplitude or excitation force magnitude and natural frequencies/damping will be explored.

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