

# Dynamic testing and characterisation of in-service bridges

L.S. Hogan, L.M. Wotherspoon, S. Beskhyroun &  
J.M. Ingham

*Dept. of Civil & Environmental Engineering University of Auckland,  
Auckland, New Zealand.*



2013 NZSEE  
Conference

**ABSTRACT:** The following is an overview of a field testing program carried out to investigate the dynamic properties of in-service bridges. Two bridges in the Auckland region, Caitcheon's Bridge and Glasgow's Bridge, were subjected to forced vibration testing and a system identification process. The aim of this testing was to determine the in situ dynamic response of each bridge system accounting for both the structural and foundation components. Each bridge was excited in both the transverse and longitudinal direction using an eccentric mass shaker and the response measured with a dense sensor array of up to 160 channels. Testing was performed over two nights at each bridge, with the extensive sensor array and eccentric mass shaker installed and removed each night in order to minimise the impact on normal bridge operations. A description of the test bridges and review of the testing procedures used are first summarised. The characteristics of the overall response of each bridge are then presented with a particular focus on the relative stiffness contributions from the abutments and pier foundations.

## 1 INTRODUCTION

Bridge foundations and abutments provide large interfaces between a bridge superstructure and the surrounding soil, contributing significantly to the overall stiffness and damping of the bridge system when loaded seismically (Kotsoglou and Pantazopoulou 2009). Due to the complicated nature of the bridge-foundation-soil interaction, one of the inherent difficulties when modelling this effect is verifying the validity of the model used as different modelling approaches can lead to wide variations in stiffness distribution, modal properties and damping (Aviram et al. 2008). While laboratory studies have provided insight as to how well the model describes the physical behaviour (Johnson et al. 2008; Anastasopoulos et al. 2010), ideally testing would be carried out on full scale specimens during in-service conditions. Forced vibration testing of in-situ structures allows for this type of verification.

Forced vibration testing has been used for many decades to determine dynamic characteristics of bridges, (Moss et al. 1982; Bolton et al. 2005), but most studies have investigated vertical excitation of in-service bridges or lateral excitation of bridge components (Elgamal et al. 1996; Halling et al. 2001). There still exists a paucity of work investigating the dynamic characteristics of in-service bridges subjected to lateral forced vibration loading.

In response to the lack of research on lateral forced vibration of bridges, a large field testing program was undertaken at the University of Auckland (UoA) to investigate the in situ dynamic characteristics of bridge-foundation systems when subjected to horizontal loading. The program investigated both bridge components and in-service bridges in order to isolate the contribution of stiffness and damping that specific components have on the bridge-foundation system. All bridges were tested in both the main transverse and longitudinal axes using a horizontal eccentric mass shaker. Acceleration data was captured using a dense array of accelerometers and analysed for translational modes using a MATLAB based GUI modal property identification toolbox (MPIT) developed at the UoA (Beskhyroun 2011). The testing procedure, modal property identification methodologies, and preliminary dynamic characteristics for two in-service bridge tests are described here, whilst testing and analysis details of the first three component tests are detailed in Hogan et al. (2011; 2012a, 2012b).

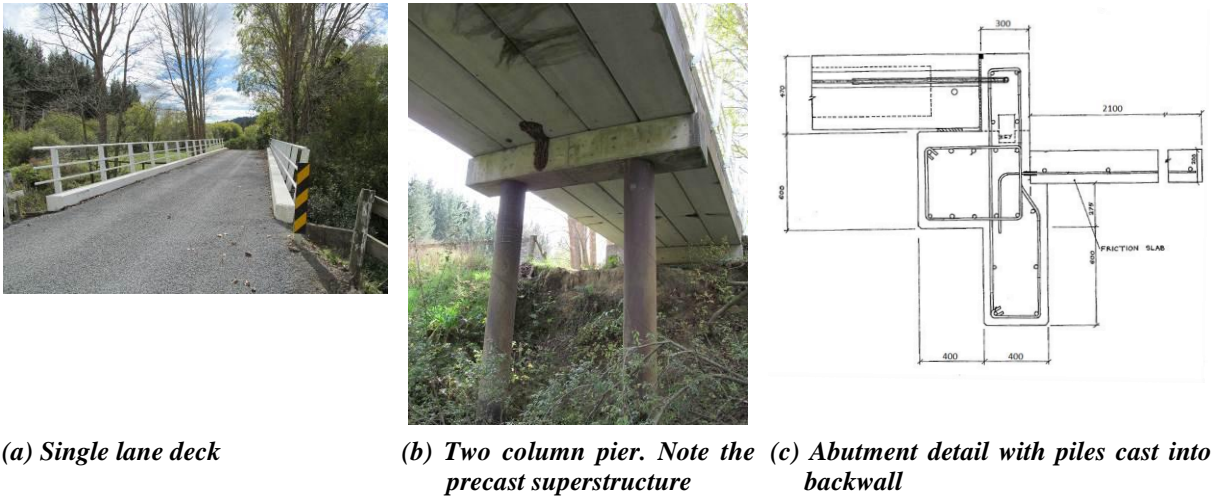
**2 TEST BRIDGE DESCRIPTIONS**

The two in-service bridges tested as part of the program to characterise in situ dynamic properties of bridges were Caitcheon’s Bridge, and Glasgow’s Bridge. These bridges were chosen for testing because they represented the two most prevalent bridge types in New Zealand: 1) precast concrete superstructure on flexible piers and 2) a reinforced concrete superstructure cast monolithically with stiff wall type piers. A description of bridge location, construction details and access that needed to be considered while testing is provided in the following.

**2.1 Caitcheon’s Bridge**

Caitcheon’s Bridge is located 3 km south of Hunua and services traffic on Caitcheon Road. The three span, single lane bridge was constructed in 1982. Each 9.13 m span consists of five precast concrete double hollow core units, 914 mm wide and 458 mm deep. Precast units are supported on 12 mm thick rubber bearings. Concrete kerbs 300 x 300 mm are cast continuously over the length of the bridge. Piers and abutments are each founded on two concrete filled steel piles 450 mm in diameter which extend to the pier and abutment caps. Due to scour from the creek that the bridge crosses, the northern pier is 6.22 m and the southern pier is 5.02 m tall. At the seat-type abutments, the piles are cast into a 6.6 m wide wall. A 2.1 m long friction slab at both abutments extends into the approach fill 595 mm below the deck surface. The backwall at the northern abutment is 1.15 m tall while the southern backwall is 1.77 m tall.

Testing was performed over two nights on Caitcheon’s Bridge. Because Caitcheon Road is a no exit road servicing three farms, access to Hunua Road was closed during testing. To allow emergency traffic to cross the bridge during testing, the eccentric mass shaker was mounted as close to the western kerb as possible providing enough space to accommodate an ambulance.



*Figure 1: Caitcheon’s Bridge superstructure and substructure configuration.*

**2.2 Glasgow’s Bridge**

Glasgow’s Bridge is located on Runciman Road, 9 km northeast of Pukekohe. The three span, cast-in-situ concrete bridge was constructed in 1947 with overall dimensions of 8.015 m wide and 23.12 m long. The two approach spans are each 7.0 m long while the centre span is 9.12 m. Piers and abutment backwalls are 8 m wide reinforced concrete walls respectively founded on six and four 400 mm diameter octagonal piles. Pier walls are 3.125 m tall but due to scour the supporting piles are exposed up to 1.0 m below the pile cap. This type of scour damage is common for many bridges throughout New Zealand. The reinforced concrete superstructure consists of a 170 mm thick deck and

four T-beams which are 440 x 760 mm at the abutments and mid-span and haunch linearly starting 1.83 m from the pier centre lines to a depth of 914 mm. The superstructure is cast monolithically with both the piers and the abutment. A 255 mm wide by 1.07 m tall perforated concrete guardrail is installed over the continuous 400 x 235 mm concrete kerb. The guardrail is not continuous over the piers.

Similarly to Caitcheon’s Bridge, testing was performed on Glasgow’s Bridge over two nights. Again, the shaker was positioned such that one lane of traffic could remain open for emergency traffic.



(a) Deck with concrete barriers (b) Wall pier with scour exposed (c) Abutment detail piles.

Figure 2: Glasgow’s Bridge superstructure and substructure configuration.

3 TESTING METHODOLOGY

3.1 Excitation Source

Forced vibration testing was performed on the bridge using an eccentric mass shaker anchored to the superstructure at the mid-length of each bridge. The shaker consisted of a series of 15.5 kg steel weights bolted onto two counter-rotating flywheels controlled by a variable speed three phase induction motor. 440 V power was supplied by a 40 kVA diesel generator located approximately 20 m away from the abutment. The force output of the shaker was dependent upon mass and driver frequency up to a maximum output of 98 kN. The frequency dependant force output for different sets of weights per flywheel at various driver frequencies is shown in Figure 3.

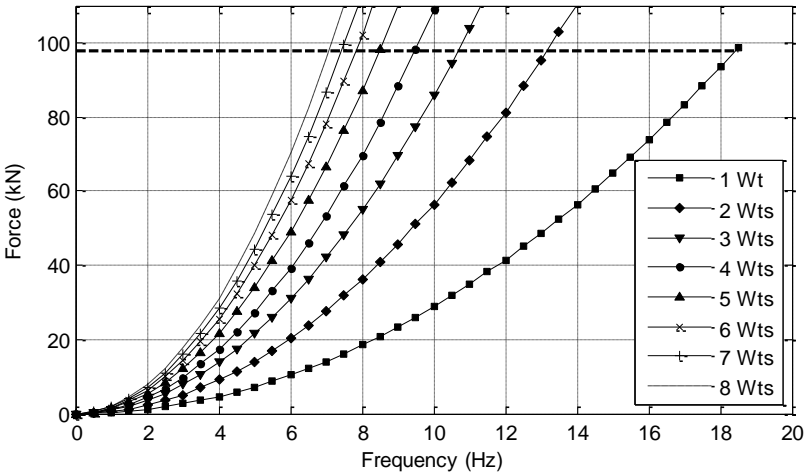


Figure 3: Eccentric mass shaker force output with varying driver frequency.

Each bridge was excited in both principal directions by sweeping through a range of frequencies with the eccentric mass shaker. Because access to each bridge was available for a short time frame, the number of tests that could be performed was limited. Table 1 details the number of tests, frequency range, and maximum force tested in the transverse and longitudinal directions at each bridge. For each sweep the excitation frequency was increased in 0.2 Hz increments, and each frequency increment was held for ten seconds with a five second ramp up time from the previous excitation frequency. This excitation protocol allowed the bridge to achieve steady state response for each excitation frequency increment while reducing the overall time needed to perform each test.

**Table 1. Forced vibration testing parameters**

<b>Bridge</b>	<b>Trans. Tests (#)</b>	<b>Trans. Freq. Range (Hz)</b>	<b>Trans. Max Force (kN)</b>	<b>Long. Tests (#)</b>	<b>Long. Freq. Range (Hz)</b>	<b>Long. Max Force (kN)</b>
Caitcheon's	6	0-8	18.4	6	0-16	73.8
Glasgow's	6	0-22	45.9	6	0-22	45.9

### 3.2 Instrumentation

Because access to each bridge was limited to two nights, the sensor array that was used needed to be dense enough to adequately capture the dynamic behaviour of the bridge yet be capable of being deployed and removed rapidly. Each bridge was instrumented with two types of accelerometers. The first set of accelerometers was uniaxial and wired into a mobile data acquisition system to provide real time acceleration data during testing. These accelerometers were oriented in the direction of shaking and installed along the longitudinal axis of each bridge deck and at the top of the piers and abutments. The second set of accelerometers used were inexpensive, wireless, triaxial MEMS accelerometers that wrote to an internal microSD card requiring data to be downloaded post-test using a USB port. The wireless USB accelerometers were located along both kerbs of each bridge and equally spaced between the ground and top of the piers and abutments. Additionally, geophones were placed on the approach fill during longitudinal excitation to characterise the damping of the abutment-embankment system. By using this combination of sensors, installation of up to 160 sensors was achieved in less than four hours.

## 4 ANALYSIS

### 4.1 Signal Processing

Due to shaker excitation force increasing exponentially with increasing frequency, the acceleration records needed to be force normalized before performing analysis in order to avoid spurious modal identifications caused by larger input forces at higher frequencies. Because driver speed on the eccentric mass shaker was directly measured during testing, this parameter was used to compute the force output at each time step and force normalise the acceleration data.

### 4.2 Analysis Methods

The forced vibration data for each test was analysed using the same methodology implemented with a MATLAB based modal property identification toolbox (MPIT) developed at the University of Auckland (Beskhyroun 2011). Five system identification algorithms were used to extract modal properties from recorded acceleration data. Three of the algorithms were frequency domain based and included peak picking (PP), frequency domain decomposition (FDD) (Brincker et al. 2001), and enhanced frequency domain decomposition (EFDD) (Jacobsen et al. 2007). The remaining two algorithms were two variations of the time domain based stochastic subspace identification (SSI) method (Katayama 2005). Analysis of modal properties was performed in two phases. First, plausible

modes were identified using the entire force-normalized acceleration records from each test. Then the analysis was repeated using the non-force-normalized acceleration data trimmed to only include excitations in a narrow frequency band centred around the mode identified in the previous step. For both phases of modal identification, mode shapes were selected using a rigorous acceptance methodology to avoid biased modal identification, detailed in Hogan et al. (2012a). System identification parameters used at each step of the analysis are shown in Table 2.

**Table 2. System Identification Parameters**

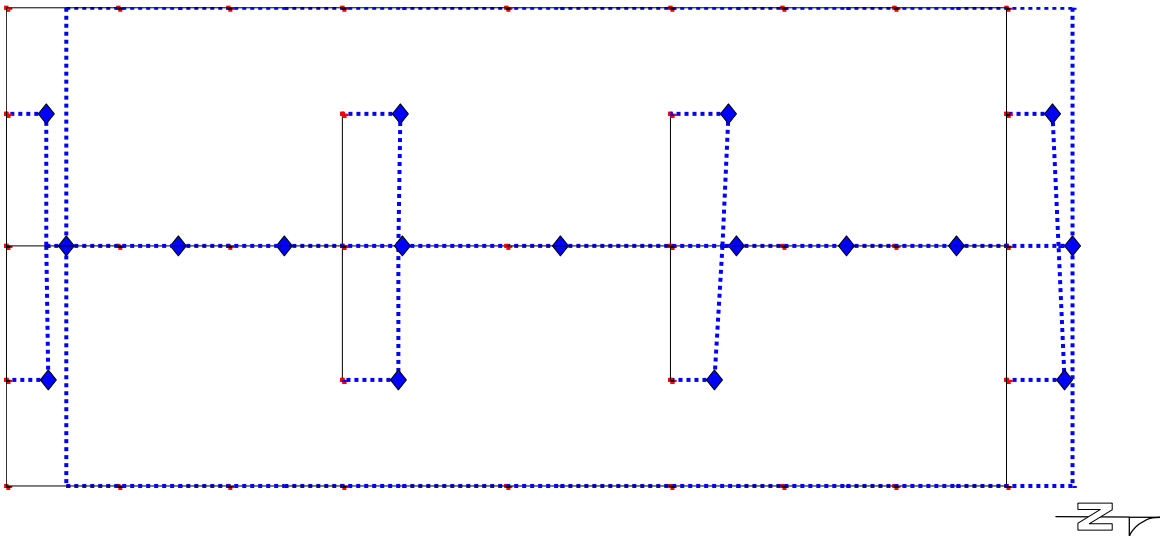
Bridge	Window Size	Hankel Matrix Size	System Order	SSI Freq. Band (Hz)	Stable Pole Freq. Difference (%)	Stable Pole MAC Value
Caitcheon’s	4096	30	100	0.5	5	0.95
Glasgow’s	4096	30	100	0.5	5	0.95

**5 IDENTIFIED MODES**

After acceleration records were analysed and the false modes discarded, modes were identified in each direction for a given bridge. This modal data was used to provide an insight into the influence of the different substructure components (abutments and approach soil mass, settlement slab, and pile foundations) on the overall dynamic response of each bridge. Due to space restrictions, only modes identified using the wired accelerometer data (i.e. accelerations parallel to the direction of shaking) are discussed here. Frequency content, amplitude, and phase difference of acceleration records from the wireless accelerometers were used to confirm preliminary conclusions made from the wired channels, but no modal identification based upon those sensors is discussed.

**5.1 Caitcheon’s Bridge Modes**

In the longitudinal direction Caitcheon’s Bridge had a natural period of 0.0743 s and a mode shape dominated by translational motion. A plan view of the mode shape is shown in Figure 4 with the modal amplitudes of the pier and abutment caps represented as the blue diamonds on either side of the longitudinal centreline. The longitudinal mode is approximately symmetrical about the longitudinal axis of the bridge. The uniformity of motion arises most likely due to the approach soil behind the abutment dominating the response in the longitudinal direction rather than the pile foundation system. This soil mass provides passive resistance along the abutment backwall, while the soil overburden on the settlement slab develops a frictional stiffness component at the interface between the soil and the slab.



**Figure 4: Longitudinal mode shape of Caitcheon’s Bridge. Blue diamonds represent sensor locations displaced by modal amplitude.  $T = 0.0743$  s.**

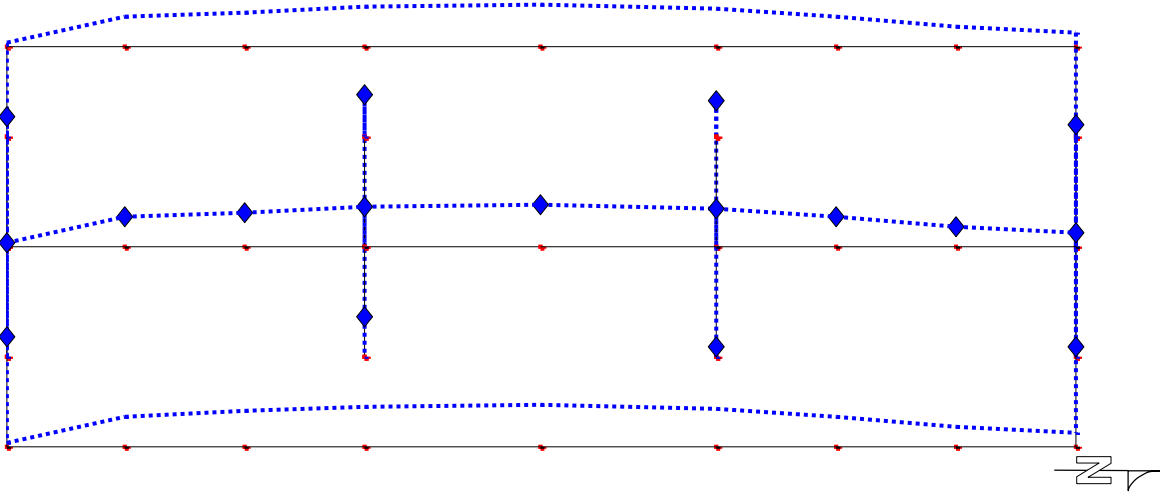
The mid-span wired accelerometers of each span recorded lower accelerations than those over the piers and abutments. Because only motion in the longitudinal direction was measured, the likely cause of the lower amplitude measured was vertical movement of the deck due to a coupled elastic buckling mode. Amplitudes and phase comparisons between the vertical acceleration records from the wireless MEMS accelerometers confirm that at the longitudinal natural period there is significant vertical movement at mid-span of the outer spans. This movement is out-of-phase between the two spans with the mid-span movement of the centre span about a third of the amplitude of the outer spans despite the excitation source being located at this span.

In the transverse direction, Caitcheon’s Bridge had a natural period of 0.1766 s. This mode shape was primarily translational with a small degree of asymmetry caused by a torsional component centred around the northern abutment. This torsional component was much more pronounced as the distance from the transverse centreline increased as is shown in Table 3.

**Table 3. Modal Amplitude Increase between the North and South Sensors**

<b>Dist. from trans. centreline</b>	14.25 m	11.06 m	7.88 m	4.69 m	0 m
	(Abut.)			(Piers)	(Centreline)
<b>% Diff. in modal amplitude</b>	80%	50%	19%	12%	0%

The torsional response is of particular interest at the substructure. Modal amplitudes at the southern abutment were 80% higher than those at the northern abutment, while the difference between pier modal amplitudes is less significant with the southern pier amplitude only 12% higher than the northern. The larger modal amplitude at the southern pier appears counter-intuitive, as the northern pier is 1.2 m longer than the southern pier and has approximately half the elastic stiffness. If the abutments did not contribute to the transverse stiffness of the bridge, the modal amplitude would indeed be higher at the northern pier. However, because the opposite response was captured, it is apparent that the abutments have a significant effect on the overall transverse response of the bridge. Because the northern abutment backwall is 600 mm shorter than the southern abutment backwall, it is stiffer, and the southern end of the bridge responds with larger modal amplitudes. This difference in modal amplitude between the two ends of the bridge decreases as the distance to the transverse centreline reduces because the inertia from the superstructure begins to dominate the transverse response.



*Figure 5: Transverse mode shape of Caitcheon's Bridge. Blue diamonds represent sensor locations displaced by modal amplitude  $T = 0.1766$  s.*

## 5.2 Glasgow's Bridge Modes

During the testing and subsequent analysis of Glasgow's Bridge no distinct modes were found in either the transverse or longitudinal directions. In the longitudinal direction, the likely dynamic response would be characterised by the longitudinal stiffness at the abutments, producing a longitudinal natural frequency outside the excitation range of 0-22 Hz used during testing. This response would be similar to the NZTA Bridge Manual member design criteria and foundation design for bridges "locked in" to the surrounding soil, i.e. the bridge will move in phase with the approach fill during an earthquake (Transit New Zealand 2003).

While the stiff response in the longitudinal direction was anticipated, the inability to detect a mode in the transverse direction was unexpected. It was originally hypothesised prior to testing that the stiff wall type piers would act like lumped masses, and the transverse deformation would take place in the exposed piles. However, it appears that similar to Caitcheon's Bridge, the abutment stiffness had a significant effect on the dynamic response in the transverse direction as well as in the longitudinal direction. Because the height to width aspect ratio of the abutments was approximately 1:8, the abutments were the stiffest elements resisting load in the transverse direction and therefore dominate the transverse response.

As no distinct modes were identified in either the transverse or longitudinal, a separate dynamic characterisation needs to be performed. This will be comprised of characterising the superstructure stiffness from the modal response of the vertical deck accelerometers, and then matching acceleration records to steady state excitations performed at various frequencies and force inputs.

## 6 CONCLUSIONS & FUTURE WORK

Forced vibration testing was performed on Caitcheon's Bridge and Glasgow's Bridge to determine the in situ dynamic characteristics of in-service bridges and the stiffness contributions of the different components of the substructure. Testing was performed within two nights at each bridge through the use of an innovative sensor array employing both uniaxial wired accelerometers, and wireless triaxial MEMS accelerometers. Modal properties were extracted through a two phase identification process using MPIT and the suite of system identification algorithms it utilizes. This process provided robust identification of mode shapes that resulted in an overview of the stiffness contributions of various substructure components on the dynamic response of each bridge.

Two modes were identified in both principal lateral directions of Caitcheon's Bridge. The longitudinal mode was found to be coupled with elastic buckling of the bridge superstructure, while the transverse mode was found to be more influenced by the abutment stiffnesses than the piers. No distinct horizontal modes were identified in Glasgow's Bridge over the excitation frequency range tested, but it was clear that in both the longitudinal and transverse direction the abutment stiffness controlled the dynamic response of the bridge. This testing suggests that for short bridges, the abutments dominate the dynamic response in both the longitudinal and transverse directions.

This testing was performed as part of a larger suite of forced vibration tests on full scale bridge structures to develop a better understanding of the in situ response of bridges and the interaction between the various structural and foundation components. CPT and shear wave velocity profiles will be conducted at four places at each bridge: next to each abutment and next to each pier. These profiles will be used to classify the soil conditions at each pier/abutment foundation and long with test results from both the bridge components and the in-service bridges computational models will be constructed to further quantify the contribution of bridge components to the overall dynamic response of the bridge. Models will be calibrated for stiffness using mode shape data and natural periods and for damping using acceleration records of the bridge and geophone records of the approach soil velocity.

## 7 ACKNOWLEDGEMENTS

The authors would like to thank Auckland Transport, and Kimberley Twigden, Luke Storie, Yasmine Zaiem, Rhys Rogers, and Chase Helgenberger for providing access to the bridge, the donation of equipment and personnel time, and logistical support. Funding was made possible through the University of Auckland Faculty Research and Development Fund, Natural Hazards Research Platform, and EQC.

## 8 REFERENCES

- Anastasopoulos, I., T. Georgarakos, V. Georgiannou, V. Drosos and R. Kourkoulis (2010). "Seismic Performance of Bar-Mat Reinforced-Soil Retaining Wall: Shaking Table Testing Versus Numerical Analysis with Modified Kinematic Hardening Constitutive Model." *Soil Dynamics and Earthquake Engineering* 30(10): 1089-1105.
- Aviram, A., K. R. Mackie and B. Stojadinovic (2008). "Effect of Abutment Modeling on the Seismic Response of Bridge Structures." *Earthquake Engineering and Engineering Vibration* 7(4): 395-402.
- Beskhyroun, S. (2011). *Graphical Interface Toolbox for Modal Analysis*. Ninth Pacific Conference on Earthquake Engineering, Auckland, New Zealand.
- Bolton, R., C. Sikorsky, S. Park, S. Choi and N. Stubbs (2005). "Modal Property Changes of a Seismically Damaged Concrete Bridge." *Journal of Bridge Engineering* 10(4): 415-428.
- Brincker, R., L. Zhang and P. Andersen (2001). "Modal Identification of Output-Only Systems Using Frequency Domain Decomposition." *Smart Materials and Structures* 10(3): 441-445.
- Elgamal, A. W., S. Alampalli and P. Van Laak (1996). "Forced Vibration of Full-Scale Wall-Backfill System." *Journal of Geotechnical and Geoenvironmental Engineering* 122(10): 849-858.
- Halling, M. W., I. Muhammad and K. C. Womack (2001). "Dynamic Field Testing for Condition Assessment of Bridge Bents." *Journal of structural engineering* New York, N.Y. 127(2): 161-167.
- Hogan, L. S., L. M. Wotherspoon, S. Beskhyroun and J. M. Ingham (2011). *Forced Vibration Testing of in Situ Bridge Span*. Ninth Pacific Conference on Earthquake Engineering, Auckland, New Zealand.
- Hogan, L. S., L. M. Wotherspoon, S. Beskhyroun and J. M. Ingham (2012a). "Modal Identification of a Bridge-Abutment System Using Forced Vibration Testing." *Bulletin of the New Zealand National Society for Earthquake Engineering* 45(4): 161-170.
- Hogan, L. S., L. M. Wotherspoon, S. Beskhyroun and J. M. Ingham (2012b). *Vibration Testing of an in Situ Bridge Pier to Determine Soil-Structure Interaction Effects*. 15th World Conference of Earthquake Engineering Lisbon, Portugal.
- Jacobsen, N. J., P. Andersen and R. Brincker (2007). *Using Efd as a Robust Technique to Deterministic Excitation in Operational Modal Analysis*. The 2nd International Operational Modal Analysis Conference (IOMAC). Copenhagen, Denmark.
- Johnson, N., R. T. Ranf, M. S. Saiidi, D. Sanders and M. Eberhard (2008). "Seismic Testing of a Two-Span Reinforced Concrete Bridge." *Journal of Bridge Engineering* 13(2): 173-182.
- Katayama (2005). *Subspace Methods for System Identification*. Berlin; London, Springer.
- Kotsoglou, A. and S. Pantazopoulou (2009). "Assessment and Modeling of Embankment Participation in the Seismic Response of Integral Abutment Bridges." *Bulletin of Earthquake Engineering* 7(2): 343-361.
- Moss, P. J., A. J. Carr and G. C. Pardoen (1982). "The Vibrational Behaviour of Three Composite Beam-Slab Bridges." *Engineering Structures* 4(4): 277-288.
- Transit New Zealand (2003). *Transit New Zealand Bridge Manual*. S. Stewart, Z. Transit New and C. Opus International. Wellington, Transit New Zealand, c2003.