Vibration Testing of an In Situ Bridge Pier to Determine Soil-Structure Interaction Effects

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SUMMARY:
The lateral dynamic response characteristics of a single span from the decommissioned Puhinui Stream Bridge in Manukau, New Zealand were determined through a series of forced vibration tests performed along the longitudinal axis of the bridge using an eccentric mass shaker. Following forced vibration testing, the dynamic characteristics of a three column pier group from the span were determined using snapback testing. Responses of the bridge span and pier group measured during the vibration testing were used to construct finite element models accounting for soil-structure interaction using a Winkler spring idealisation of the soil. Because of the simplified nature of the pier group, it was modelled first, and used to perform sensitivity analyses to obtain realistic bounds for soil and material properties based upon CPT data and concrete specifications. The pier group model will be extended to capture the response measured by the forced vibration testing of the bridge span but is not discussed here.

Keywords: Field testing, bridge, soil-structure interaction, dynamic response

1. INTRODUCTION

In recent years the importance of properly modelling soil-structure interaction has been widely investigated (Kappos and Sextos 2009; Kotsoglou and Pantazopoulou 2009; Ülker-Kaustell et al. 2010), but due to the complicated nature of soil-structure interaction, one of the inherent difficulties when modelling this effect is verifying the validity of the model. While laboratory studies can provide some insight as to how well the model describes the physical behaviour (Anastasopoulos et al. 2010), ideally testing would be carried out on full scale specimens in the conditions they are most likely to experience while in service. Forced vibration and snapback testing of in situ structures allows for this type of verification and both methods have been used for several decades to determine eigenfrequencies and mode shapes of bridges (Moss et al. 1982; Samman and Biswas 1994; Halling et al. 2004; Bolton et al. 2005).

Forced vibration and snapback testing methods were used to determine, respectively, the in situ dynamic characteristics of a simple concrete bridge span and a three column pier group in Manukau, New Zealand prior to their demolition. Results from in situ testing were analysed using a MATLAB based system identification toolbox (SIT) developed at the University of Auckland, and the captured responses were subjected to a suite of system identification algorithms to determine modal properties of the two structures. Dynamic properties identified by SIT were used to calibrate finite element models of the tested structures. The development of the pier group model and the investigation of soil-structure interaction effects using a beam on Winkler springs idealisation for the soil is discussed, along with a parametric study to determine the sensitivity of the model to various input parameters. Finally, a brief plan for future work on the pier group model and the extension of this model to account for the response of the bridge span is presented.

2. BACKGROUND

To investigate the response of a bridge span on pile foundations, the Puhinui Stream Bridge in Manukau, New Zealand was selected for vibration testing and system identification. The bridge formerly carried four lanes of traffic on SH20 located approximately 5km west of the Manukau City Centre and was scheduled for demolition as a result of traffic being diverted to a new bypass. Constructed in 1986, the four span superstructure consisted of eighteen 10 m precast single hollow core concrete beams per span. Beams were seated on elastomeric pads and diaphragm action between them was provided by a 100 mm thick concrete slab. The superstructure was founded on piers consisting of seven 450 mm wide octagonal concrete piles driven into sandy deposits. Piers were skewed at 30° and had a cross fall of 6%.

The original bridge as it existed in 2007 is shown by the outlined region in Figure 1a. To facilitate construction of the new bypass, the northernmost span of the original bridge needed to be demolished. A temporary bridge was built adjacent to the original that allowed for redirection of traffic over both bridges and the demolition of the span (Figure 1b). Upon completion of the bypass, but prior to the demolition of the Puhinui Stream Bridge, Puhinui Stream was diverted to a culvert southeast of the original bridge (Figure 1c). A significant amount of approach material was removed around the southeastern abutment as a result of this diversion.

Because of the various modifications performed on the site to accommodate the new bypass, the bridge system was too complicated to test unmodified and could no longer be considered representative of New Zealand bridge stock. Therefore during the demolition of the bridge two simplified structures were tested: a single span of the original bridge with no abutments, and a three column pier group.

(a) (b) (c)

Figure 1: Changes to Puhinui Stream Bridge site during SH20 bypass is constructed (a) September 2007, (b) April 2009, (temporary bridge shown dashed) (c) April, 2011 (bridge has been demolished)

3. BRIDGE SPAN TESTING & SYSTEM IDENTIFICATION

In order to fit within the demolition schedule, the bridge span (Figure 2a) was only available for testing during one night. Therefore, in an effort to gain as much useful information as possible in such a short time span, the bridge was only excited in the longitudinal axis of the original bridge, as this was more flexible than the transverse direction and easier to excite with a low frequency source. The bridge span was subjected to a series of frequency sweeps from an eccentric mass shaker (Figure 2b) with a maximum horizontal output of 30 kN. Longitudinal accelerations of the span were measured with twenty-two uniaxial accelerometers: four on the deck and eighteen on the columns of the southern pier. Three sweeps ranging from 0-4.5 Hz were performed to identify the first translational and torsional modes. Once the dynamic properties of the span were determined, soil was removed from around the two westernmost piles to a depth of 450 mm. Shaking was repeated and the difference in response between the original foundation condition and the altered condition was determined. For a detailed account of instrumentation, excitation, analysis, and identified modal properties of the bridge span testing refer to Hogan et al. (2011).

(a) (b)

Figure 2: Isolated span (a) western elevation and (b) shaker location on deck.

4. PIER GROUP TESTING & SYSTEM IDENTIFICATION

4.1. Experimental Set Up

Once the forced vibration system identification had been completed, the bridge was demolished except for a column group from the southern pier. The pier group was comprised of three columns and a pier cap that cantilevered past the easternmost column (Figure 3). To prepare the pier group for snapback testing, five 10 g piezoelectric accelerometers with a 1024 Hz sampling rate were installed. Two accelerometers were placed above the tops of two columns on the pier cap and the three were placed at approximately mid-height of each column. Column heights and sensor locations are shown in Table 1. Accelerometers were attached to timber blocks previously installed on the bridge (Figure 4a).

Snapback testing of the pier group was performed using a 60 ton excavator to displace the pier group and then rapidly releasing it (Figure 4b). Testing was repeated five times and accelerations in the out-of-plane direction were recorded. Due to limited access to the bridge site during demolition there was not ample time between the demolition of the bridge span and the snapback tests to build a reference frame to measure deflections of the pier group, thus only accelerations were measured.

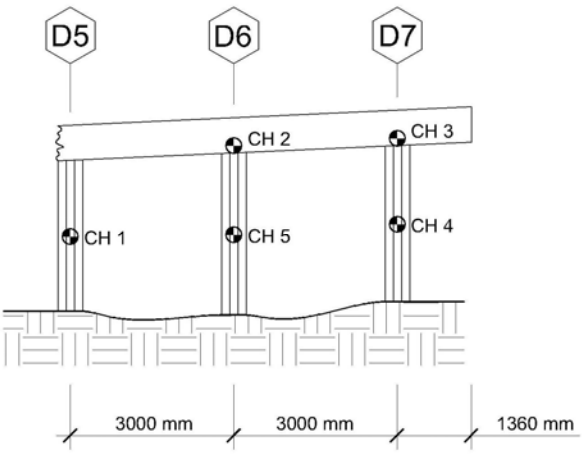

Figure 3: Layout of accelerometers on pier group

(a)

(b)

Figure 4: (a) Three column pier group being prepared for snapback testing. Wooden blocks indicate location of accelerometers (b) Three column pier group released from snapback testing

Table 1: Heights of columns/sensors on snapback pier group

Column/Sensor	D5	D6	D7	CH 1	CH 2	CH 3	CH 4	CH 5
Height Above Ground (m)	2.33	2.68	2.81	0.9	2.33	2.68	1.52	1.12

4.2. System Identification Analysis Methods

The collected snapback data was analysed using a MATLAB based system identification toolbox (SIT) developed at the University of Auckland (Beskhyroun 2011). To reduce the effects of noise the data set was filtered with a lowpass filter of 6 Hz. The data set for each test was then subjected to five different system identification algorithms which were used to find natural frequencies and mode shapes. Three of the five algorithms were frequency domain based and included peak picking (PP), frequency domain decomposition (FDD) (Brincker et al. 2001), and enhanced frequency domain decomposition (EFDD). A window size of 512 was used for these methods as it was found to provide the best resolution, while reducing inaccuracies created by zero padding.

The two time domain based algorithms applied were NexT/ERA (Caicedo et al. 2004) and stochastic subspace identification (SSI) (Katayama 2005). Stable poles identified by SSI were identified as modes by two different methods. In both methods the algorithm was run fifty times starting with a Hankel Matrix of 40 and system order of 100 which reduced by two with each iteration until the final iteration was run with a system order of two. Stable poles identified in each of these iterations were compared by one of two methods. In the first variation of SSI, the stable poles identified around the singular values generated from the singular value decomposition (SVD), were compared. If two consecutive poles within ±0.125 Hz of the singular value had frequencies within 1% and a modal assurance criteria (MAC) value (Allemang 2003) greater than 0.90 both poles were kept and averaged. If both poles did not meet these criteria the first pole was discarded and the second pole was compared to the subsequent one. This series of comparisons was continued until all stable poles in the frequency range had been compared and averaged. The resulting mode shape and natural frequency are the combination of several stable poles and therefore provided a robust method of system identification.

While the first method used singular values to identify stable poles, the second variation of SSI breaks up the entire frequency range tested into 0.25 Hz bands. Stable poles are compared within each band and averaged using the same method as the previous SSI variation. Those bands with the most stable poles are considered to contain true modes and are then used to compare to the other algorithms used. SIT was used to calculate two modes for each system identification method in order to correctly identify the fundamental mode of the pier group. Modes were differentiated between true structural response and false noise modes using a two step process. First power spectral densities (PSD) were calculated for each channel and resonant frequencies were identified. A visual inspection of the generated mode shapes was then performed. If the mode shape did not include impossibilities, such as the pier cap moving in two different directions simultaneously, and the corresponding eigenfrequencies were close to the resonant frequencies identified in the PSD, the mode was considered to be a true mode. Once the true modes were established for a given system identification method, MAC values and differences in identified frequencies were compared between the five snapbacks to determine repeatability. Modes shapes were accepted if they had a MAC value of 0.90 or higher and the identified eigenfrequencies were within two standard deviations. Using these criteria, 85% of mode shapes generated by the system identification algorithms were accepted. These mode shapes were then averaged and compared to the average mode shapes generated by the other methods. Finally, these were averaged to generate a confident mode shape and fundamental period for the pier group.

4.3. Snapback Results and Identified Modal Properties

Before subjecting the snapback data to the analysis methods described above, an initial investigation was performed in both the time and frequency domain to determine period and verify that the system identification methods were giving realistic results. Representative response data shown in both the time domain (Figure 5) and the frequency domain (Figure 6) primarily consists of a single frequency content of 2.27 Hz. As the majority of the mass in the system comes from the pier cap at the top of the columns, the expected mode shape would be similar to a single degree of freedom (SDOF) oscillator with a period of 0.44 s.

The system identification analysis outlined in Section 4.2 identified a fundamental period of 0.43 s which is consistent with the frequency domain data shown in Figure 6. The identified mode shape shown in Figure 7 is primarily translational with a torsional component is similar to the expected SDOF mode shape. This torsional component is a result of two factors: 1) Column D7 is approximately 250 mm longer than Column D6 making it more flexible, and 2) the pier cap extends 1.36 m beyond Column D7 thereby adding significantly more mass at the east end of the pier cap.

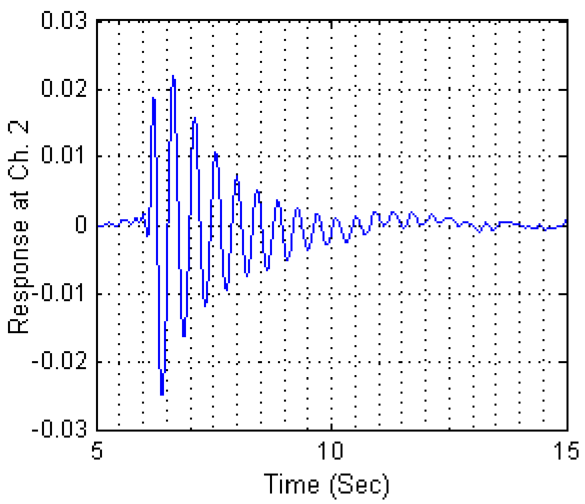

Figure 5: Sample acceleration time history response from pier group snapback

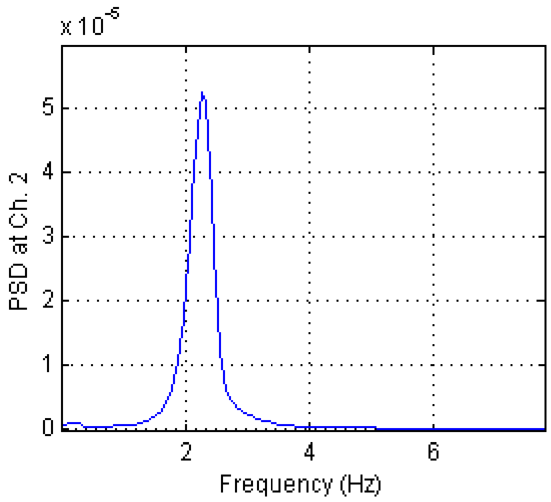

Figure 6: Sample power spectrum density plot from pier group snapback

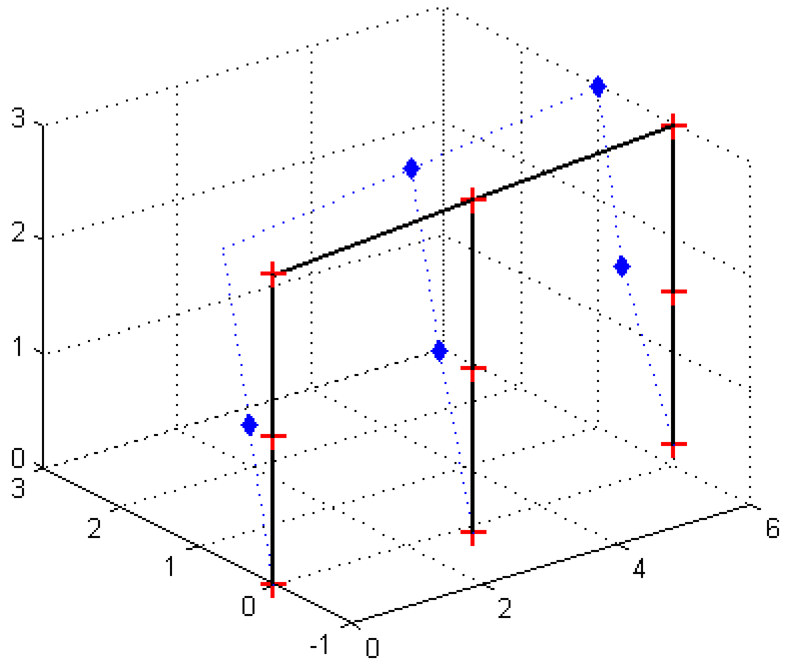

Figure 7: Mode shape generated by SIT for mode 1 of pier group. Blue diamonds indicate modal amplitudes at sensors

5. SNAPBACK MODELLING

5.1. Model construction

The identified mode shape and natural period of the pier group was used to calibrate a finite element model constructed in OpenSees (PEER 2012). Superstructure components were divided into five elements per component and pile elements were 0.5 m long. This element mesh provided adequate mass distribution with no further refinement of the model was found using smaller element meshes. Section properties for the pier group piles and pier cap were determined from construction drawings and site measurements. The elastic moduli of used for the structural elements were based upon recommended values from the New Zealand Concrete Standard 3101:2006 (Standards New Zealand 2006).

Soil characteristics were defined based upon three CPT logs taken as part of the site investigation for the temporary bridge constructed in 2008 alongside the original Puhinui Stream Bridge. Elastic moduli were computed at every half metre depth using methods provided by Robertson and Cabal (2010). Adjusting for the elevation of the pier group and assuming deposits were approximately uniform across the site, four idealized soil layers were determined for the pier group (Figure 8). These soil layers were modelled using elastic springs attached at the pile nodes. Springs were attached to each side of the pile. Moduli of subgrade reaction for the Winkler springs were determined using the approach proposed by Vesic (1961). Using methods proposed by Davies and Budhu (1986) and assuming a constant soil modulus to a depth of 7 m, the active length of the piles was approximated as 3.5 m.

Spring constants were determined by multiplying the modulus of subgrade reaction of the spring by the tributary length of the adjacent pile elements. With the exception of the surface spring, this tributary length was equal to the height of the pile element. At the surface, the tributary length used to determine the spring constant was one half of the element height. Stiffness parameters for both the used in the OpenSees model are summarized in Table 2.

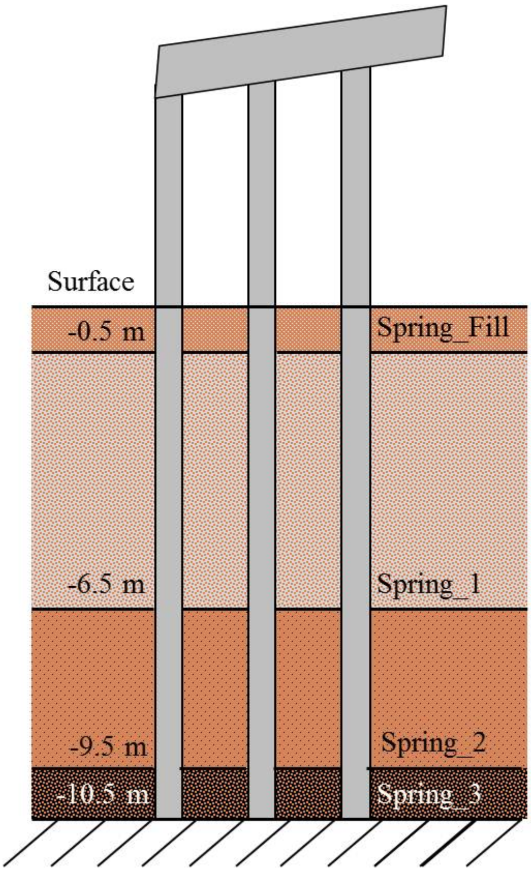

Figure 8: Idealized soil layers and spring constants used in OpenSees Model

Table 2: Stiffness parameters used in OpenSees model

Parameter	Ec Col/Piles	Ec Pier Cap	Spring_Fill	Spring_1	Spring_2	Spring_3
Stiffness	29,000 MPa	25.000 MPa	2,000 kN/m	3,500 kN/m	8,000 kN/m	25,000 kN/m

5.2. Model Outputs and Parametric Study

The model produced a fundamental mode shape shown in Figure 9 at a period of 0.4028 s. The period is 3% lower than the period determined experimentally, and the mode shape has a MAC value of 0.975 when compared to the experimental mode shape, with 1.0 being perfect correlation of mode shapes. While these natural period and mode shape did not exactly match the experimental data, the errors in fundamental period and mode shape are small enough that the model can be considered a good representation of the experimental data.

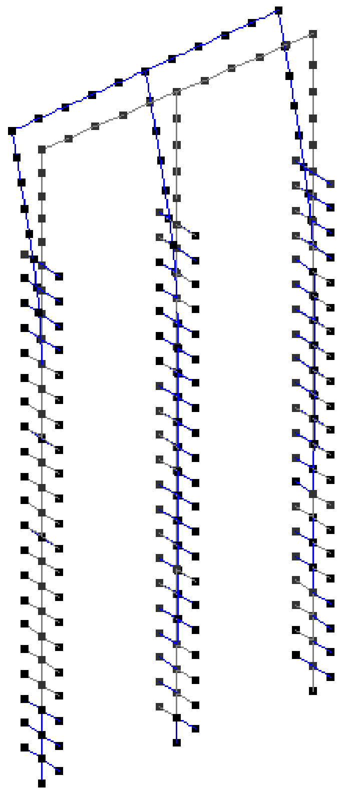

Figure 9: OpenSees model: Mode 1 (T = 0.4208 s)

With the model providing an adequate representation of the experimental structure, the stiffness of the columns, piles, and top two spring levels were adjusted by ± 25% from the baseline stiffness values in Table 2 to determine the sensitivity of the model to these parameters. Shifts in period and MAC values from the baseline model were determined for each variance in stiffness parameter. Mode shapes was found to be insensitive to these variances in stiffness as the MAC values between the model and the experimental mode shapes did not change with variances in stiffness parameters. Period shifts for each variance in stiffness parameters are shown in Figure 10. A decrease of all stiffness parameters by 25% only increased the fundamental period by 14%, therefore the fundamental period is also relatively insensitive to stiffness variance in this range. This parametric study does indicate that the fundamental period of the model could be matched to the snapback period by either a reduction of column and pile stiffness of 6%, a 12% reduction of soil spring stiffness, or some combination of the two.

The influence of foundation flexibility on modal response was also investigated by fixing all degrees of freedom at the ground surface of the model and comparing mode shapes and fundamental periods between the fixed base and the baseline models. Column stiffness was varied by ± 25% of the baseline stiffness for both the fixed base and the baseline model and fundamental period and MAC values were determined for each variation of stiffness. Fundamental periods for each variation of the two models are shown in Figure 11. MAC values between the fixed base and baseline model fundamental mode shape were 0.88 for all stiffness variations. As was expected, the respective mode shapes for the Winkler spring model and the fixed base model were insensitive to uniform changes in column stiffness because for each model there was no variation in boundary condition or inter-column stiffness. The difference in mode shape between the two models can be attributed to the elimination of the ground surface displacement and rotational components of the mode shape in the fixed base model. While the two models have similar mode shapes, the per cent difference between the fundamental periods of the two models is as high as 109% depending upon column stiffness variation (Figure 11).

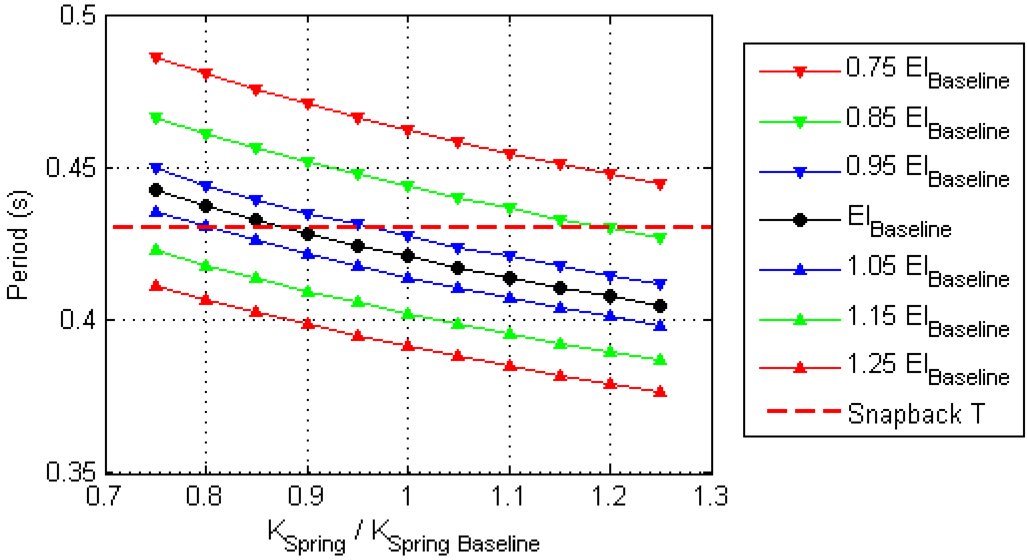

Figure 10: Period shift of model due to changes in Winkler soil spring and column stiffness values

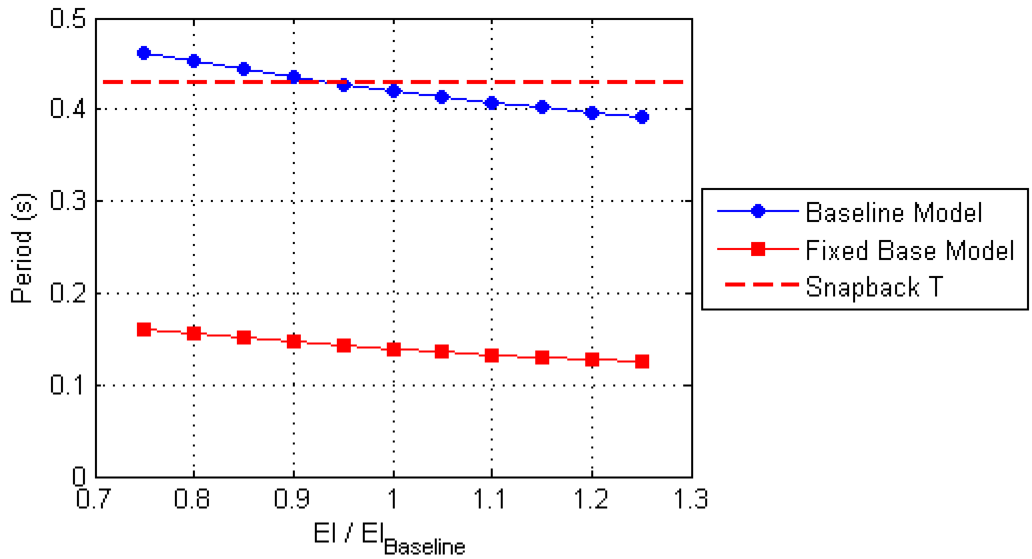

Figure 11: Period shift between baseline and fixed base model for range of column stiffness values

6. CONCLUSIONS & FUTURE WORK

Forced vibration and snapback testing of the Puhinui Stream Bridge were able to capture dynamic characteristics of a bridge span and a three column pier group respectively. The experimental mode shape and fundamental period of the pier group were used to develop a finite element model of the pier group which was able to provide a good representation of the experimental mode shape and period using a simple linear beam-on-Winkler-springs model of the foundation. A parametric study was performed to determine the sensitivity of the model to changes in column, pile, and soil spring stiffness. The model was found to be relatively insensitive to changes of ± 25% in stiffness of these elements. Finally, the Winkler springs model was compared to a fixed base model and a per cent difference in period of 109% was identified, highlighting the significant effect of foundation flexibility on the response of the pier group.

Modelling of snapback testing will be extended to include the effects of damping on the response of the system. The damped foundation model will be extended to model the response of the bridge span to the forced vibration testing. Both the unaltered and altered states will be modelled so that the contributions of the removed soil can to be directly quantified and the robustness of the model can be tested using two different boundary conditions.

7. ACKNOWLEDGMENTS

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