Assessment of soil-structure interaction methods using full scale dynamic testing

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ABSTRACT: With computational modelling becoming an integral part of the seismic bridge design process, it is necessary to ensure that the modelling assumptions being used are appropriate. This requirement is particularly important when determining how foundation flexibility affects the response of a bridge to seismic loading. A study was performed to determine the ability of several common foundation models to represent the dynamic behaviour of a case study bridge in New Zealand, whose dynamic properties were previously determined through a full scale field testing programme. The foundation models were implemented in a computational model of the bridge and natural periods and mode shapes were determined in transverse axes of the bridge. Modal properties of the computational models were compared to the mode shapes and natural periods of the case study bridge identified during field testing in order to determine the ability of the various modelling approaches to correctly represent the stiffness distribution of the integrated bridge-foundation system. For modelling approaches that were unable to represent the modal properties of the test bridge, the necessary adjustments to achieve accurate representation are discussed. The effects of modelling choices that could be made when implementing these foundation models are discussed and their effects on the design base shear are investigated.

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 (El-Gamal and Siddharthan 1998, 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 (Moss et. al 1982), 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 (Samman and Biswas 1994), but most studies have investigated vertical excitation of in-service bridges or lateral excitation of bridge components (Elgamal et. al 1996, Halling et. al 2004). 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. The testing procedure, modal property identification methodologies, and dynamic characteristics for three component level tests and preliminary results from testing of the two in-service bridge tests are described in Hogan et al. (2011, 2012a, 2012b, 2013). The methodologies described by the Hogan studies were used to determine modal properties of Caitcheon's Bridge along the transverse axis using a dense network of accelerometers. An investigation was then performed to determine if existing simplified integrated structure-foundation models were capable of representing the modal properties of the test bridge. The effects of various modelling choices are explored and a brief discussion on the consequences to the design and assessment process that these modelling choices have is presented.

2 TEST BRIDGE DESCRIPTIONS

As discussed in Hogan et al. (2013) Caitcheon's Bridge is a three span, single lane bridge constructed in 1982 3 km south of Hunua, New Zealand (*Figure 1* and *Figure 2*). The bridge was selected for testing because the precast superstructure, flexible piers and seat type abutments are typical of many bridges throughout New Zealand. 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.

(a) Single lane deck

(b) Two column pier. Note the pre-cast superstructure

(c) Abutment with piles cast into backwall

Figure 1. Caitcheon's Bridge superstructure and substructure configuration.

The site was characterized with three cone penetrometer tests (CPT) at both abutments and at one pier (*Figure 3*). These CPT soundings were interpreted using methods described in Robertson and Cabal (2010) to determine soil type and modulus of elasticity. The top 2 m of the soil profile are predominantly clay and silty-clay layers while between depths of 2 and 8 m, the profile is dominated by silty-sand layers. These layers are underlain by a stiff clay layer in which the piles are founded. Young's modulus with depth was computed from the three CPT soundings taken at the bridge site using the empirical relationships provided by Robertson and Cabal (2010) (*Figure 4*).

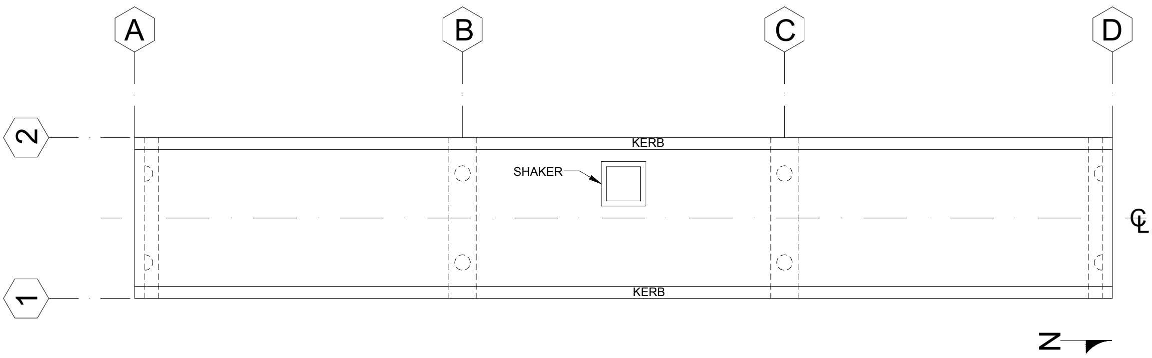
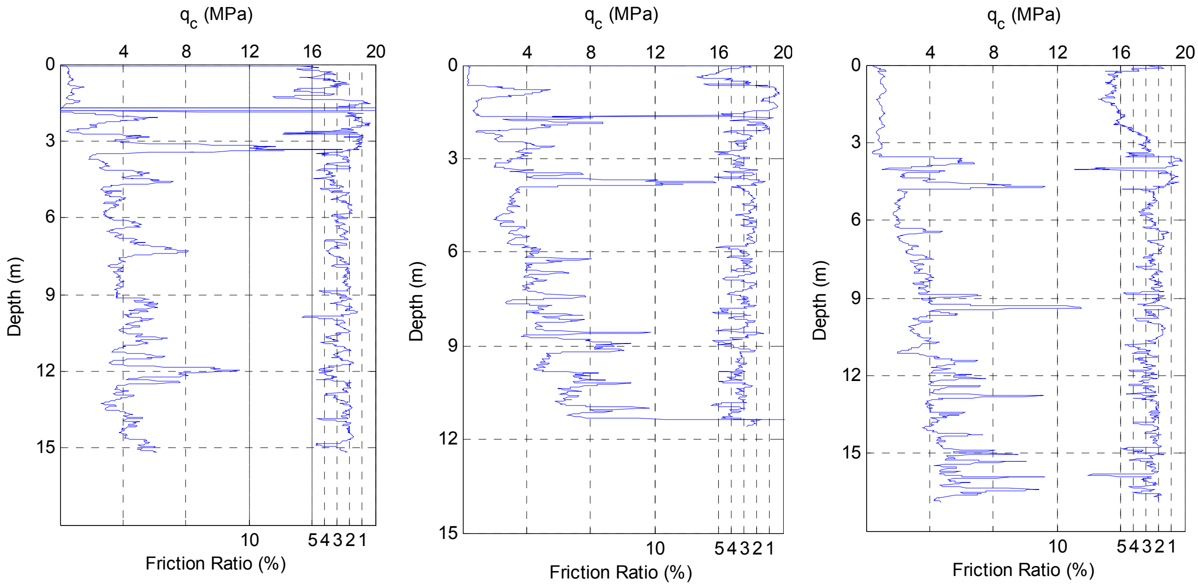
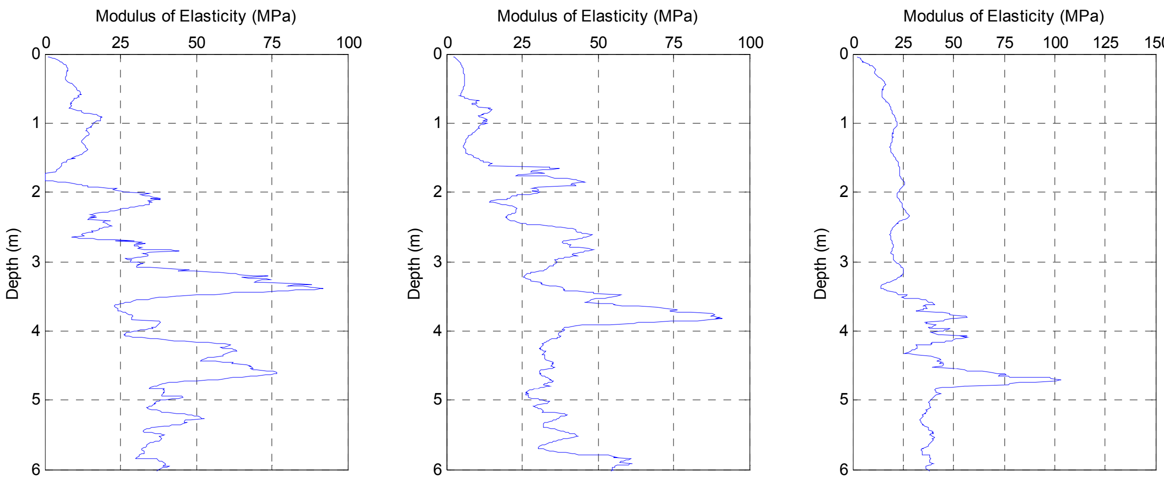


Figure 2. Caitcheon's Bridge plan view



(a) Abutment A (South abut.) *(b) Pier B* *(c) Abutment D (North abut.)*

Figure 3. Caitcheon's Bridge CPT logs.



(a) Abutment A (South abut.) *(b) Pier B* *(c) Abutment D (North abut.)*

Figure 4. Caitcheon's Bridge soil Young's modulus (Es) for first 6 m of depth.

3 TESTING METHODOLOGY

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. The force output of the shaker was dependent upon mass and driver frequency up to a maximum output of 98 kN. The bridge was excited in both principal directions by sweeping through a range of frequencies with the eccentric mass shaker. 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. Details of the excitation protocol are provided in Hogan et al. (2013).

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. By using this combination of sensors, installation of up to 160 sensors was achieved in less than four hours.

3.1 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). 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-normalize the acceleration data.

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).

4 IDENTIFIED MODES

After acceleration records were analysed and the false modes discarded, modes were identified. 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 the bridge in the transverse direction. In the transverse direction, Caitcheon's Bridge had a natural period of 0.1766 s (natural frequency of 5.66 Hz). As shown in *Figure 4* and *Figure 5*, this mode shape was primarily translational with a small degree of asymmetry caused by a torsional component centred around the northern abutment (Abutment D). This torsional component was much more pronounced as the distance from the transverse centreline increased.

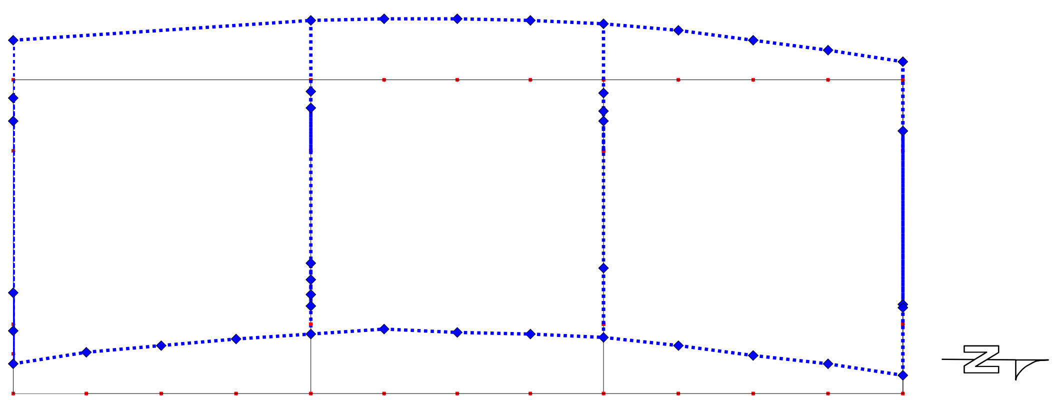


Figure 5. Transverse mode shape of Caitcheon's Bridge plan view. Diamonds represent sensor locations di placed by modal amplitude T = 0.1766 s (f_n = 5.66 Hz).

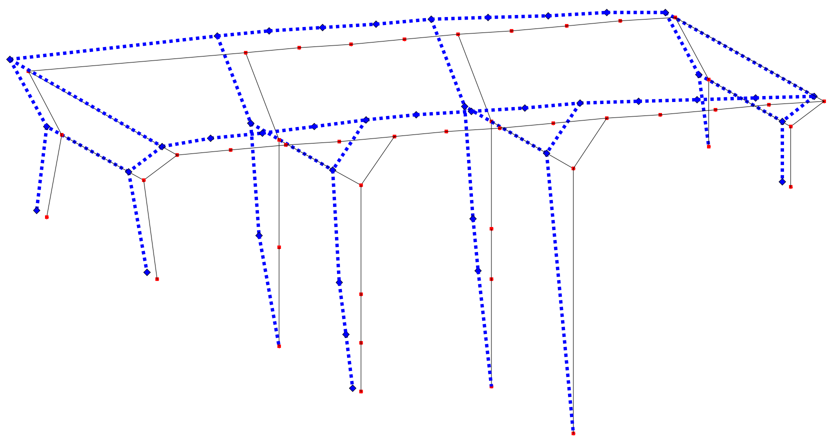


Figure 6. Transverse mode shape of Caitcheon's Bridge 3D view. Diamonds represent sensor locations displaced by modal amplitude T = 0.1766 s (f_n = 5.66 Hz).

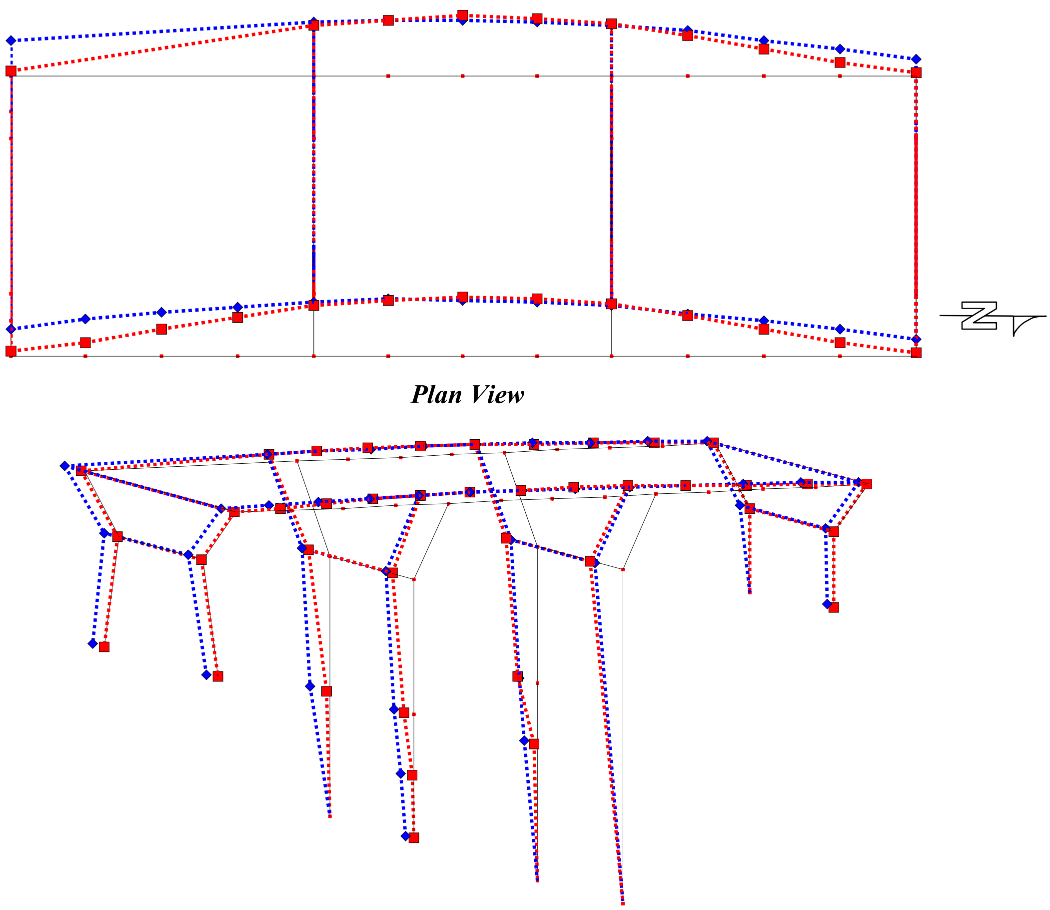
5 INTEGRATED STRUCTURE-FOUNDATION MODELING

5.1 Model Development and Results

With the establishment of the modal properties of the bridge, an investigation was performed to determine to what degree the stiffness of the foundation affected the modal properties of the bridge. By understanding the influence on foundation stiffness on modal properties of the bridge, an engineer can make better predictions of base shear and the distribution of seismic demands on the bridge, particularly the location of maximum shear and bending moments in the piers and abutments. It was also of interest to determine if existing, simplified models were capable of representing the modal properties determined from field testing.

In order to determine both the influence on foundation stiffness and the ability of existing integrated structure-foundation models to represent the modal properties of Caitcheon's bridge, two three dimensional computational models were constructed using OpenSees (PEER 2012), one model having foundation nodes fully constrained i.e. fixed base, and one model utilizing a Winkler spring idealisation of the surrounding soil. Pier caps, columns/piles and abutment seats were modelled using elastic beam-column elements with transformed gross section properties as no cracking of the concrete was observed during testing. Abutment backwalls and stem walls were modelled using shell elements while the deck was modelled using a grillage of elastic beam-column elements. The 12 mm thick elastomeric bearings located at the abutments and pier caps were modelled as elastic zero-length elements with horizontal, vertical, and rotation stiffness based upon stiffnesses published by several manufactures known to produce bearings commonly used in New Zealand and allowing for hardening effects due to aging.

Natural periods and mode shapes were determined for the fixed based model and compared to those identified from forced vibration testing (*Figure 7*). The fixed base model was able predicted a fundamental period of 0.12 seconds (8.36 Hz) which was 32% lower than the identified period of 0.176 seconds (5.66 Hz). The fixed base model was able to reasonably capture the modal amplitude of the middle span of the deck. As expected the fixed base mode shape under-predicted modal amplitudes at the two side spans of the deck and both abutments because the abutment foundations were constrained and the in-plane stiffness of the abutment wall did not allow significant differential movement between the top and bottom of the abutments. The modal amplitudes at Pier B were found to be under-predicted but were over-predicted at Pier C due to Pier C being at least half as stiff as any other structural element.



Plan View

3D

Figure 7. Comparison of mode shapes between field testing results and fixed base model. Blue diamonds represent modal amplitudes of field testing and red squares represent modal amplitudes from fixed base model.

Foundation flexibility of Caitcheon's Bridge pier group was investigated using a Winkler spring foundation. Piles were modelled using the section properties of the columns described in the fixed base model to a depth of 10.5 m and discretized into a number of elements to capture the changes in the soil profile. Elastic Winkler springs, modelled using zero length elements with a uniaxial elastic material, were attached at one end to the nodes of each pile element with the other end of the spring element fixed in all degrees of freedom. Spring constants were calculated by multiplying the moduli of subgrade reaction, determined using the method proposed by Carter (1984) and the Young's modulus with depth shown in *Figure 4*, by the tributary length of the adjacent pile elements. At the surface, the spring constant is one half of the spring immediately below it, due to the tributary length being half of the adjacent pile node. Spacing of pile nodes of 0.1 m or 0.22D was used, where D is the pile diameter of 450 mm.

Two adjustments were made to the Young's modulus profiles shown in *Figure 4* to account for topography effects at the bridge site. First as no CPT sounding was performed at Pier C, the material around Pier C was assumed to be the same as that surrounding Pier B, but starting at a depth of 1.2 m to account for the scour around the base of Pier C. The second adjustment was made to the Young's modulus profile at Abutment A (the southernmost abutment). Because the CPT sounding was taken at the mid-span of Abutment A and Pier B, the CPT profile did not include the top 2.5 m of soil around the Abutment A piles. From borehole logs and SPT N values provided in the construction drawings, the soil at each abutment had similar properties. Therefore, the entire CPT profile from CPT D was used to represent the soil at Abutment A. Pile group effects were not considered in the model as it has been found that at loads below soil yield, p-multipliers are essentially unity (Stewart et. al 2007).

Stiffness of the backfill material at the abutments was modelled using the approach described in Section 6.1.3, using the mean value of the stiffnesses calculated from the range of backfill material properties for granular soils tests described in Shamsabadi et al. (2007) and the Caitcheon's Bridge abutment geometry. From the mean value of these stiffnesses, the baseline backfill stiffness was estimated at an assumed maximum wall displacement of 1.0 mm to be 548,000 kN/m at Abutment A and 484,000 kN/m at Abutment D.

The friction slab at each abutment, also referred to as the settlement or approach slab, was modelled as a series of elastic spring elements located at the top of each pile. The spring stiffness of the friction slab was derived to include the frictional resistance of the top and bottom of the slab, and the passive resistance at the edge of the slab. The frictional resistance was derived using the relationships proposed by Al-Gahtani (2009) and the initial passive resistance of the slab was determined using the method provided by Douglas and Davis (1964) for the elastic passive resistance of an anchor block at depth. Passive resistance of the friction slab was only provided in the transverse direction, and a rotational spring was included to account for the torsional resistance of the backwall provided by the weight of the approach soil on the friction slab.

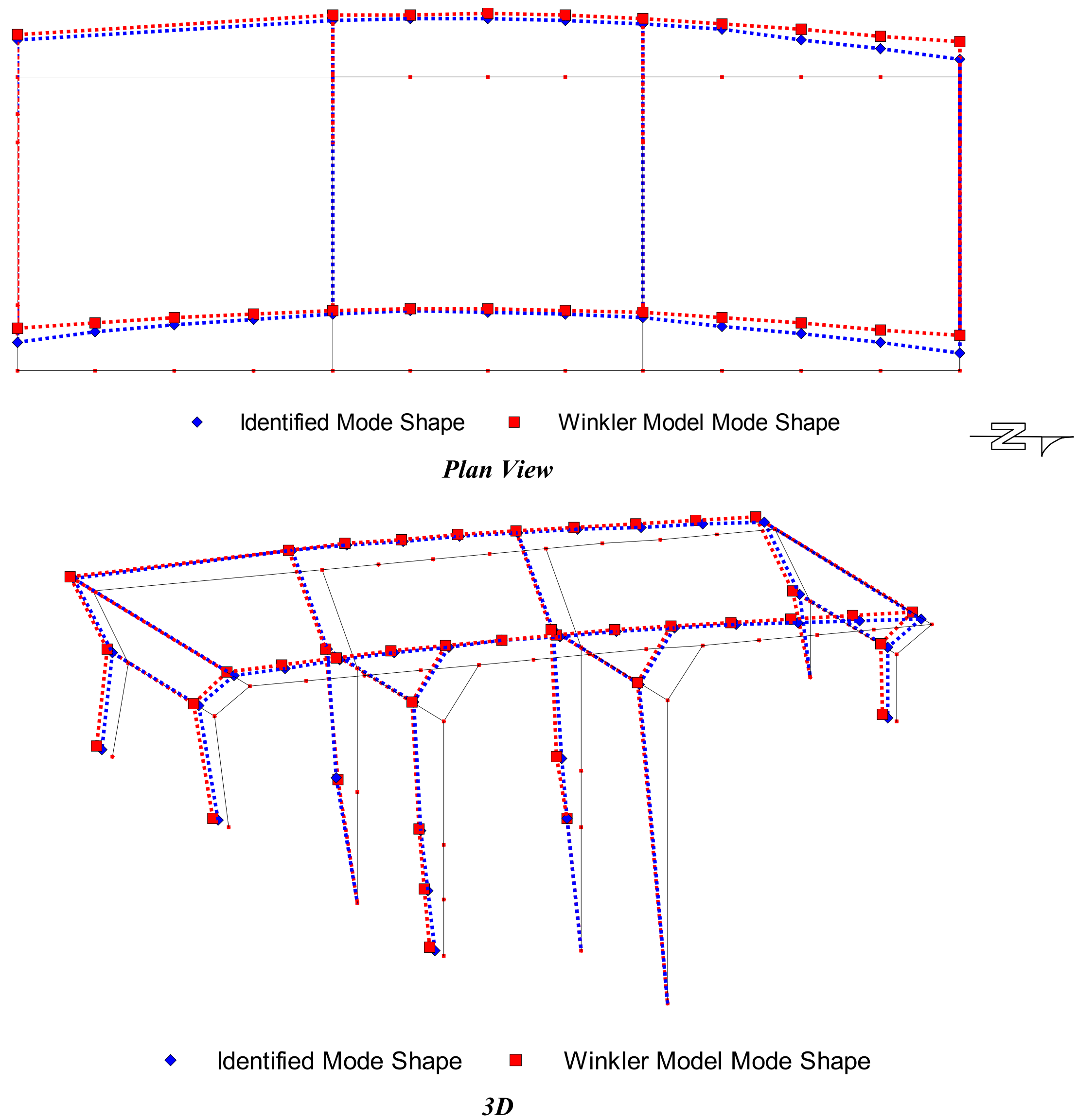


Figure 8. Comparison of mode shapes between field testing results and integrated structure-foundation model. Blue diamonds represent modal amplitudes of field testing and green squares represent modal amplitudes from structure-foundation model.

The period and mode shape of the transverse mode of the Winkler spring model using baseline values were similar to those found from field testing, with the period over-predicted by 11% and a MAC value of 0.983 when compared to the experimental mode shape, with a MAC value of unity representing perfect correlation between the two mode shapes and a MAC value of zero representing uncorrelated mode shapes (Allemang 2003). The discrepancy in mode shapes between the model and the field testing results arises from the over-prediction of modal amplitudes at the abutments by the Winkler spring model, suggesting that the abutments were too flexible. An increase in the soil stiffness around the abutment piles by 50%, the upper limit of specified soil stiffness range, reduced the modal amplitudes at the abutments, increased the MAC value to 0.991, and reduced the natural period to within 2.5% of the field testing results. While accounting for the foundation flexibility was found to be necessary in order to match the mode shape and period identified during testing, uniform changes in soil stiffness by ± 50% at each pile resulted in little change in period or mode shape. The insensitivity of modal parameters to changes in soil stiffness over the range of expected stiffness values suggests that the pile stiffness controls the foundation stiffness.

The removal of the friction slab stiffness from the model had as significant effect on the transverse natural period, increasing the natural period to 35% greater than that identified from field testing. The modal amplitude at the abutments increased by 14-20% compared to the baseline Winkler spring model and the MAC value decreased to 0.965. These changes in modal properties demonstrate the degree to which the friction slab provides resistance in the transverse direction and the importance of including the passive resistance developed along the edge of the friction slab as neglecting this passive resistance would place larger deformation demands on the piers.

5.2 Effects of Model Choice on Design and Assessment Considerations

The predicted modal properties from both the fixed base and the integrated structure-foundation model both provide insight into possible discrepancies between assumed and actual bridge response that could arise from modelling choices. While the period of the fixed base model was 40% lower than the period identified from field testing, the base shear determined from an equivalent static analysis would not change as the New Zealand design spectral acceleration for bridges on intermediate soils is the same for all structures with periods below 0.4 s. The differences in mode shape between the fixed base model and the one identified from field test suggest some inaccuracies in seismic demand distribution that may arise during the design or assessment process. Because the fixed base model over-predicts the modal amplitude at the top the piers and under-predicts it at the bottom, the piers will be expected to be subjected to higher curvatures than will actually occur. This increased demand on the piers will reduce the perceived demand on the abutments and could lead to the piers being designed with much more capacity than is necessary while the necessary capacity of the abutments may not be correctly assessed. This incorrect distribution of seismic demand makes the case for implementing some level of foundation flexibility into the model used for the assessment of a bridge even if geotechnical investigation is not available.

The mode shape predicted by the integrated structure-foundation model was very similar to the identified mode shape of the bridge, suggesting that the model correctly represents the distribution of seismic demand to each substructure component. However, the over-prediction of modal amplitudes at the ground level would suggest that location of fixity and maximum moment are shallower than the model predicts. Caution should then be taken when assessing the location of vulnerable reinforcement details such as lap splices. The modelling of the passive resistance developed along the friction slab also highlighted the degree to which the abutment stiffness resists loading in the transverse direction. Accounting for this friction slab stiffness would provide a means to limit transverse deformations and loading to potentially vulnerable piers in bridges of similar size to Caitcheon's Bridge.

6 CONCLUSIONS & FUTURE WORK

Forced vibration testing was performed on Caitcheon's Bridge to determine its in situ dynamic characteristics 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. The fundamental period in the transverse direction was identified and was found to be more influenced by the abutment stiffnesses than the piers.

Two computational models were constructed and the natural periods and mode shapes were compared to those determined from field testing. The fixed base model under-predicted natural period by 40% and over predicted the seismic demand on the piers. The integrated structure-foundation model was able to match the identified natural period of the bridge and provided a good overall prediction of modal amplitudes. The ability of this simple model to predict modal properties and the poor representation of seismic demand distribution of the fixed base model highlights the need to include some degree of foundation flexibility during bridge design and assessment even if limited information is known about the subsoil conditions.

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