SHAKE TABLE TESTS OF A STEPPING BRIDGE MODEL

Q. T. Ma[1], M. H. Khan[1]

[1] *Department of Civil and Environmental Engineering, The University of Auckland, New Zealand*
Email: q.ma@auckland.ac.nz

ABSTRACT :

The careful design and construction of a scaled model of the South Rangitikei Railway Bridge (SRRB) is reported herein. The model was designed following stringent similitude requirements and a series of dynamic tests including shake table tests were conducted. This paper presents the results of the snap back tests and shake table tests with harmonic excitation only. The aim of the exercise was not to exactly emulate to the actual SRRB, but merely to use the SRRB as a realistic existing example to investigate the general dynamic behaviour of rocking structures. Despite this, the snap back tests of the scale model matched previously published data on the SRRB's natural period prior to pier uplift. Furthermore, experimental results suggest the natural rocking period of the prototype SRRB could vary between 1.73 s and 4.33 s depending on the amplitude of lateral displacement. This finding reaffirmed previous research which hinted the natural rocking period of a rocking structure is amplitude dependent and a non stationary value. Shake table tests with sinusoidal excitation showed the model responded seemingly like a classical elastic structure with distinct modal response.

KEYWORDS: Rocking, Bridge, Shake table tests

1. INTRODUCTION

The potential use of a rocking mechanism for seismic isolation of structures has been highlighted by earthquake engineers since the 1960s (Housner 1963; Meek 1978). Observations post devastating earthquakes such as the 1960 Chilean Earthquake and the 1964 Great Alaskan Earthquake have seen examples of seemingly vulnerable structures surviving relatively unscathed due to fortuitous rocking isolation (Cloud 1963; Hanson 1973). The phenomenon of rocking is also known as foundation uplift in the geotechnical engineering community, where the phenomenon had been investigated exhaustively including the effects of soil compliance (Apostolou et al. 2007; Meek 1975; Wolf 1976). Numerous research since the 1880s have shown that while the mechanics of rocking object appear intuitive, it is in fact highly nonlinear, sensitive and complex. Various techniques have been proposed to model such systems subjected to base excitations, however dynamic experimental verification remained scarce.

Past studies based on modal quantities have revealed that the rocking of structural systems has the effect of lengthening the fundamental period of vibration, while having little influence on the periods of higher modes (Chopra and Yim 1985; Jennings and Bielak 1973). As rocking leads to a sudden decrease of system stiffness post a particular threshold, it has the effects of a fuse and typically lowers the base shear demand and accelerations experienced by the structure. However cautions should always be taken as in certain conditions as rocking systems could overturn or may in fact increase the demands on a structure. An infamous example of this is the unseating of a bridge span of the San Francisco Oakland Bay Bridge in the 1989 earthquake, a direct result of large residual displacements from unexpected rocking of Pier E9 (Astaneh-Asl and Shen 1993). Other pitfalls include the yielding or liquefaction of the soil beneath the rocking foundations; residual tilting and overturning; increased vulnerability due to structural-soil resonance from long period ground motion in soft soil sites, and increased structural demand for some structures due to possible local load reversal during uplift.

When the potential pitfalls are properly accounted for, rocking is considered to be advantageous amongst structural engineers. Despite this, the implementation of rocking mechanisms for seismic isolation worldwide had been rare. In contrast, New Zealand in the past 30 years has pioneered in the use of rocking mechanisms for seismic protection. Exemplar structures include the South Rangitikei Railway Bridge (SRRB) (Cormack 1988), an industrial chimney at Christchurch Airport (Sharpe and Skinner 1983) and the Deadman's Point Bridge at Cromwell, Central Otago (Sharpe and Binney 1984), all of which were specifically designed to rock in the outset to isolate the structures from strong ground motion.

1.1. The South Rangitikei Bridge

The SRRB completed in 1981 represents the first and one of the very few major modern structures with rocking implemented as a seismic isolation strategy. As a result, the SRRB remains of special interest to the earthquake engineering community. The SRRB provides a vital link on the North Island Main Trunk line across the Rangitikei River. Alongside the SRRB's aesthetically pleasing quality, the SRRB also has a number of unique engineering features. These include the pretension cables stretched along the bridge deck which increase the bridge's longitudinal stiffness, a special base pier detail which disconnect the bridge piers to the foundation and facilitate the pier uplift, and a specifically designed torsional energy dissipator to control and limit the stepping motion. Fig. 1 and 2 present a schematic and a photo of the pier base and an elevation sketch of the SRRB.

Opportunities now exist to apply recent findings and examine previous design assumptions on an in-service rocking structure.

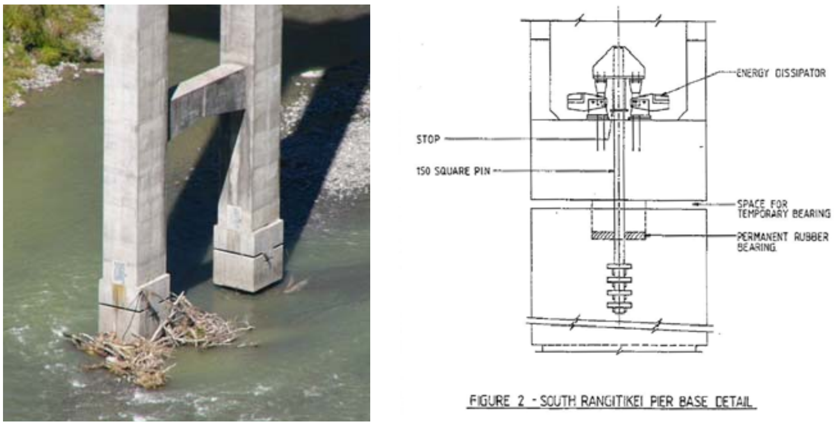

Figure 1 - a) The Pier base as built, b) Schematic of the Base detail (source: Cormack (1988))

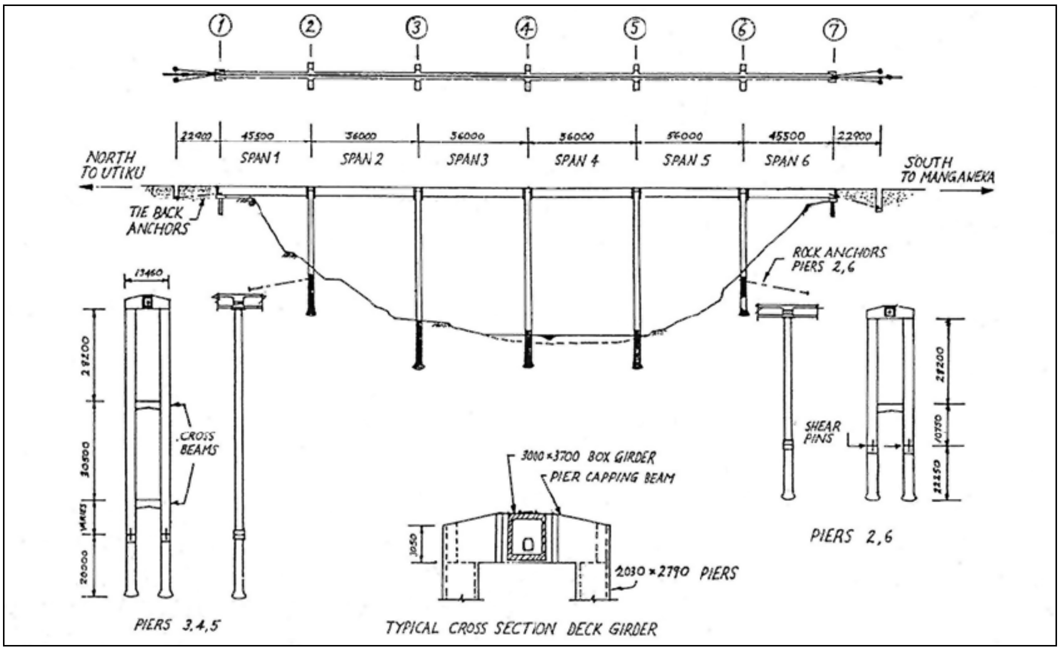

Figure 2 - Elevation of SRRB and typical cross sections (Source: (Tilby 1981))

2. SCALE MODEL TESTING

The physical testing of structures and structural assemblages at a reduced scale is arguably still one of most realistic technique for simulating the effects of earthquake loadings on structures. Scale model testing permits complex interactions between many physical quantities to be maintained. Furthermore, it allows for the discovery of new relationships as the testing is conducted with the minimum number of assumptions. The experimentation at a reduced scale for civil engineering structures is sometimes inevitable due to the cost involved in replicating a true size model and also the limited resources of many laboratories to replicate the seismic forces.

The use of scale models in earthquake engineering had been a well established practice, in particular in the design of concrete structures within their elastic limits. In the simplest case, a mechanical seismic problem is characterised by six physical quantities, namely, length, time, accelerations, mass, stiffness and force. Since these six quantities ultimately contain only three fundamental dimensions, force, length and time, only three dimensionless groups are required to identify the problem according to Buckingham π's theorem.

Consequently for scaled model testing, if the values of these dimensionless groups are maintained, then the characteristics and complex interactions of the physical quantities are deemed to be scaled consistently and the model should behave exactly as the prototype. For convenience, new variables, scaling ratios (N) are defined as the prototype quantities over model quantities as in Equation 2.1.

$$N_i = \frac{i_{prototype}}{i_{mod\,el}} \tag{2.1}$$

where N_i = Scaling ratio of physical quantity i
$i_{\text{prototype}}$ = The "i" quantity of the prototype (e.g. i can equal mass, then $m_{prototype}$ is the mass of the prototype structure)
i_{model} = The "i" quantity of the scale model

Subsequently, the three dimensionless groups can be recast into three fundamental similitude expressions which govern the consistent scaling of a mechanical seismic problem. These expressions are presented below in Equations 2.2 to 2.4.

$$N_E = \frac{N_M N_{\ddot{y}}}{N_h^{\,2}} \tag{2.2}$$

$$N_h = N_{\ddot{y}} N_t^{\,2} \tag{2.3}$$

$$N_F = N_M N_{\ddot{y}} \tag{2.4}$$

A detailed discussion and derivations of the above can be found in a report by Dove and Bennett (1986).

Now whilst it is relatively easy to develop these theoretical similitude requirements, great difficulties are encountered in fulfilling them simultaneously. For instance, Equation 2 demands that the mass of the model to be scaled proportionally to the square of geometric scaling. This is clearly impossible using similar material as the mass of the model would be proportional to the volume, thus related to the cube of the geometric scaling.

As there are three similitude criteria and six possible scaling ratios, three or more of the scaling ratios have to be nominated for implementation. In the current experiments, a scaling scheme utilising direct geometric scaling, dissimilar material and time scaling was used. A summary of the scaling parameters used can be found in Table 2.1. The chosen scheme utilised artificially added mass to ensure conformance to the similitude laws. $N_{\ddot{y}}$ was also deliberately set to 1 unlike some shake table experiments to ensure gravity effects are consistently scaled. Additional air resistance effects were ignored in the current experiment.

Also worth noting is that when investigating responses in the nonlinear range, even when basic similitude is achieved, there are other material properties that can affect the accuracy of result. This includes the effects of creep, shrinkage and the variations of ultimate strength due to the different rate of loading. Additionally, the construction of the model often introduces further unexpected distortions. For example, consider a steel structure constructed from standard hot rolled sections, the prototype steel sections will have a particular residual stress distribution due to hot rolling, which are not replicated from a scaled model most likely formed by welding together non-standard plates.

Table 2.1 Governing scaling law and parameter used for the dynamic model

Physical Quantity		**True Replica Laws**	$N_h = N_h$, $N_{\ddot{y}} = 1$, $N_E = N_E$	**Chosen scaling ratio**
Length	h	N_h	N_h	152.4
Time	t	$\sqrt{\dfrac{N_h}{N_{\ddot{y}}}}$	$\sqrt{N_h}$	12.35
Accelerations	$\ddot{y}$	$\dfrac{N_h}{N_t^2}$	1	1
Mass	M	$\dfrac{N_E N_h^2}{N_{\ddot{y}}}$	$N_E N_h^2$	245903
Stiffness	E	$\dfrac{N_M N_{\ddot{y}}}{N_h^2}$	N_E	10.59
Force (any forces)	F	$N_E N_h^2$	$N_E N_h^2$	245903

2.1. Design and Construction of the SRRB Scale Model

Like all scale modelling exercise, the design of the SRRB scale model was an extremely delicate exercise including many theoretical and practical considerations. Amongst these included the constraints of the testing equipment, the constructability of the scale model and the practicality of making small measurements.

Firstly, the scale model was restricted to approximately 3 m long to fit onto the shake table, as a result a very large geometric scaling ratio of over 100 was required. This precluded the use of concrete or micro-concrete for the model, as the resulting sections thicknesses are as small as 3 mm. Whilst it is preferred to use similar material for the model as it has a greater probability in mimicking the full nonlinear stress strain behaviour of the specimen, fortunately for the current bridge model, this was inconsequential as the bridge prototype is expected to remain materially elastic for all practical input motions, an established result from Cormack (1988). In such case, the model material was only required to behave similarly in the elastic range.

Polyvinyl Chloride (PVC) was selected as the base material for the scale model. PVC was chosen for its desirable strength to weight characteristics, availability in a range of convenient standard sheet sizes and its workability to form the required sections. A high shear and peeling strength epoxy was selected to match the PVC and it was very effective for joining the sheets together into the required hollow sections for the bridge deck and piers. Once the material was decided upon, finite element simulations confirmed the elastic assumption of the prototype under the anticipated loading. Subsequently, finite element simulations of the PVC scale model ensured the model will not exceed the stress limits of the PVC material. A series of uniaxial tensile tests and bending tests were completed on sample sections to accurately estimate the modulus of elasticity. These tests also verified the use of the epoxy for providing an integral structure and to ensure the sections will not fail under the anticipated loading. These experiments concluded that the PVC sections have an elastic modulus of 2635 MPa and thus giving rise to a N_E value of approximately 10.59.

In parallel to the construction of the pier and deck sections, a specifically designed steel supporting structure was fabricated to support the bridge model. The supporting structure consisted of seven steel plates welded on seven precisely measured columns which were in turn welded onto a ground beam. The two end steel plates acted as the end support for the bridge while the others carried the five individual piers. Moreover, two 4 mm diameter steel rods were inserted to each of the central five steel support plates to simulate the actions of the shear pins in the prototype structure. Fig. 3a and b show a schematic of the support structure and a photo of a typical central pier support showing the shear pin.

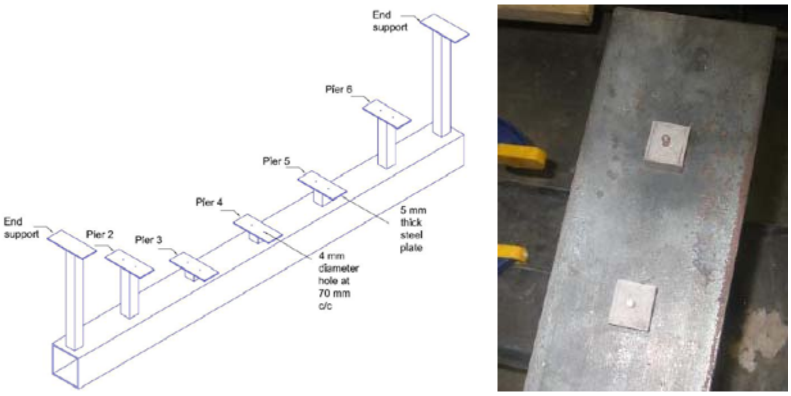

Figure 3 - a) Schematic of the support structure, b) Typical central support showing the shear pin

2.2. *Artificially Added Mass*

Geometric scaling almost always leads to a shortfall of mass according to similitude requirements. In the current experiments, a number of small steel blocks were added to allow an even distribution of the added masses in order to satisfy the similitude requirement. An even distribution of the added mass was vital for this experiment as previous research indicated a rocking problem is sensitive to the location of centre of gravity. Due the space limits around the model, an elaborate setup was devised to fix the added masses onto the structure. Typically, pairs of masses were clamped to the bridge and rubber pads were added to minimise the contact area and any unintended increases in stiffness or fixity. A photo of the completed bridge model with the added mass is shown in Fig. 4a.

2.3. *Measuring the Structural Response*

In order to capture the local and global response of the model SRRB, a total of 21 portal displacement gauges were attached to the model bridge. Eleven portal displacement gauges were installed at each pier cap and at the middle of each bridge span to capture the lateral displacements of the bridge. Ten other portal gauges were attached to the base of each pier legs, at its side to measure any uplifts. These two arrangements are shown in Fig. 4b and the data were recorded through an analogue to digital converter at a sampling rate of 1000 Hz.

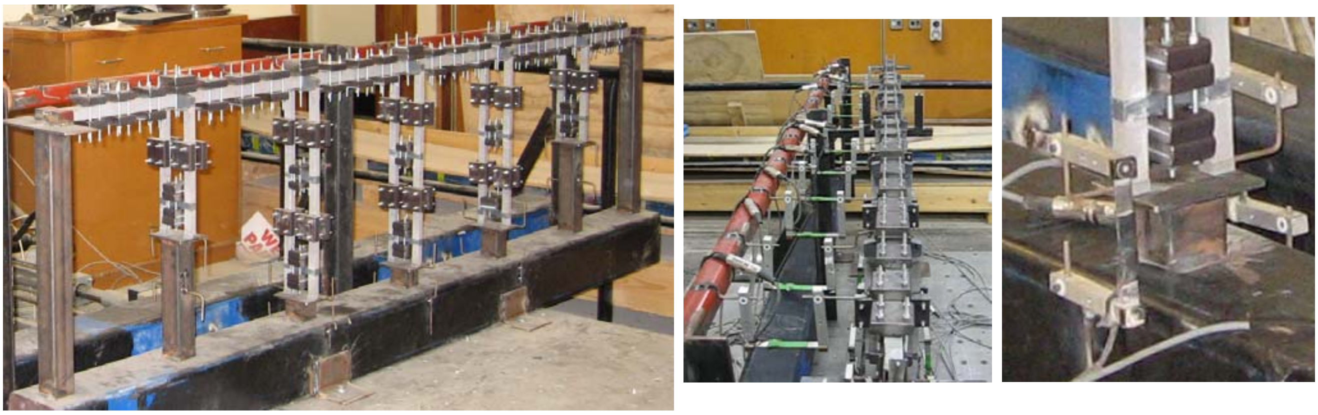

Figure 4 – a) The completed model with the added mass, b) the displacement gauges and c) uplift gauges

3. DYNAMIC TESTS

Three types of dynamic tests were conducted on the model bridge. These included snap back tests and shake table tests with sustained harmonic excitations and shake table tests with earthquake like excitations. Due to space limitations only the first two types of tests are discussed herein.

3.1. Snap back Tests

In the snap back tests, the model bridge was displaced laterally to a range of displacements via a piece of string attached at the midpoint of the bridge deck. The model was held still then released suddenly by the cutting of the string. Subsequent to the release of the model, the bridge enters into free vibrations and eventually comes to rest. Fig. 5 (Top) shows the typical lateral displacement time histories at the midpoint along the bridge deck and the corresponding pier uplifts. In this figure, it was evident that the left and right pier leg uplifts alternately as the bridge oscillated between left and right, a hallmark signature of rocking motion.

Fig. 5 (Bot.) presents the typical displacement shapes for a particular free vibration cycle. This figure shows the behaviour of the bridge is asymmetrical and this is expected due to the uneven heights of Pier 3 and 5. Preliminary analyses showed the time required for the bridge to rock from its peak displacement back to its upright position, a quantity known as the quarter period in previous research, varied between 0.035 s and 0.09 s depending on the amplitude of vibration. This is shown in details in Fig. 8. Considering similitude, this result corresponded to a natural rocking period between 1.73 s and 4.33 s for the prototype structure. Considering only the data points when uplift did not occur, the SRRB has an approximate fundamental period of 1.63 s. This closely matched a reported value of 1.6 s by Cormack in 1988. These results further reaffirm Housner's findings which stated the natural period of a rocking structure is amplitude dependent and is non stationary.

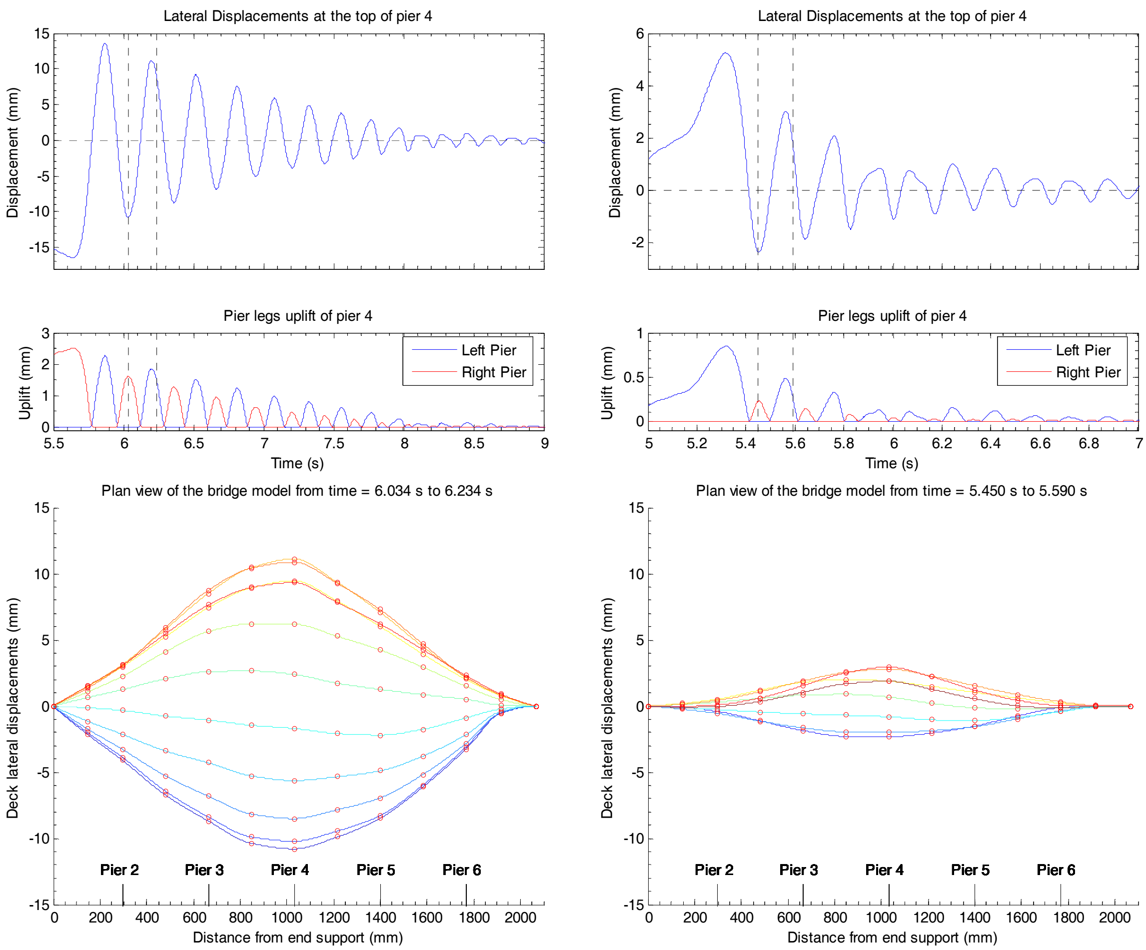

Figure 5 – (Top) Typical time history response of the model, (Bot.) Displacement shapes of the model

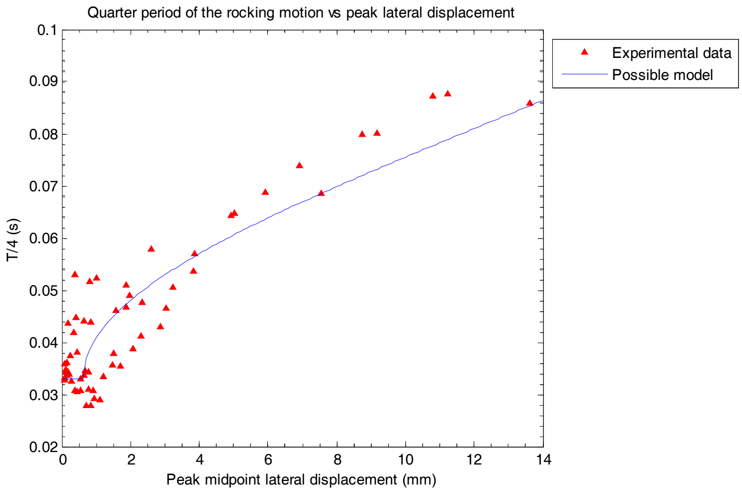

Figure 6 - Quarter periods of the model bridge versus peak midpoint lateral displacements

3.2. Shake table tests with sustained harmonic excitations

In these tests, the shake table applied series of harmonic base motion perpendicular to the longitudinal axis of bridge model. Nine rounds of shaking were applied to the bridge, each round with a different intensity or frequency. A summary of the table motion and the dominant modal response is presented in Table 3.1 below.

Table 3.1 Summary of shake table tests and results

Test Run	Excitation Freq. (Hz)	Steady State Disp. (mm)	Dominant Response
1	2	41.20	1st Mode
2	4	12.49	1st Mode
3	6	4.40	1st Mode
4	8	3.16	3rd Mode
5	10	2.42	3rd Mode
6	4	15.91	1st Mode
7	6	7.09	1st Mode
8	8	5.39	3rd Mode
9	10	3.81	3rd Mode

Test runs 1-5 and 6-9 had a peak table acceleration of 0.2g and 0.4 g respectively.

At a glance, the response of the model resembled that of a classical elastic structure. The typical response is characterised by the transition from a transient to a steady state harmonic motion synchronised to the excitation. Depending on the proximity of the excitation frequency to the nominal "natural frequencies", the steady state motion resembled the corresponding natural mode shapes. This behaviour is illustrated by the scale model displaying a first mode response in the 2 - 6 Hz tests and a third mode response in the 8 and 10 Hz tests. This corresponded well with the results from the previous snap back tests which suggested the model has a maximum first-mode rocking frequency of 7.14 Hz when uplifts are just initiated in the first mode shape. Fig. 7 presents a typical displacement time history of the shake table tests with the corresponding third mode displaced shape.

Whilst the results appear intuitive, the mechanics which led to the motion are complex and are beyond the scope of the current paper. Unlike a classical structure, one may not exploit modal orthogonality to decompose the motion of the structure, "natural frequencies" for the rocking bridge model exist in bands and are non-stationary, furthermore the mechanics of coordinating the rocking of the individual piers to enable first and third mode displaced shapes are unknown and theoretically complex.

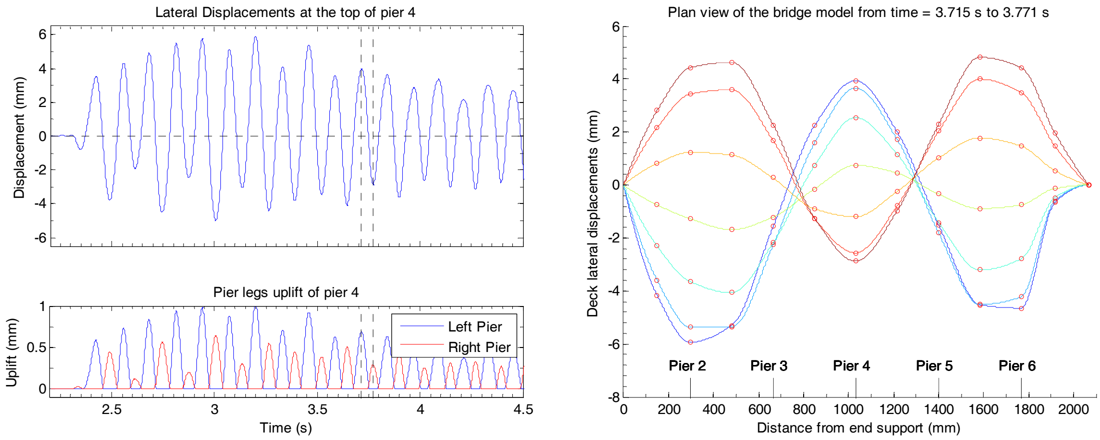

Figure 7 - Typical displacement time history and displaced shaped from shake table tests

4. CONCLUSION

The careful design and construction of a dynamically consistent scaled model of the SRRB is presented. The final scale model used a different material to the prototype structure and an elaborate scheme was devised to attach artificially added masses to the scale model to ensure inertial effects are properly replicated. This scale model was not intended to exactly emulate to the actual SRRB but merely to use the SRRB as an example of a realistic rocking structure. Despite this, the model behaved closely to the expected dynamic response of the actual SRRB. Uplifting of the piers and global asymmetrical response were clearly demonstrated in the snap back tests. The snap back tests showed the SRRB may have a natural rocking period of between 1.73 s and 4.33 s, and the ambiguity arose from the fact that natural rocking period is amplitude dependent and non-stationary. Shake table tests with sinusoidal input showed the rocking model seemingly responded like a classical elastic structure with distinct modal response. Further investigation is recommended to examine the subtle differences.

5. ACKNOWLEDGEMENT

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