Effect of Higher Vibration Modes on Seismic Response of a Structure with Uplift

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SUMMARY:
Recent investigations showed that minor structural uplift could act as a control mechanism and reduce the earthquake impact. Previous studies focused mainly on the effect of uplift on single-degree-of-freedom (SDOF) systems. In this investigation, a multi degree-of-freedom (MDOF) model is considered based on a scale six-storey building. To reveal the higher mode contribution to the response of a structure with uplift an additional equivalent SDOF model was considered. Shake table test was performed and the ground motions applied were simulated based on Japanese design spectra. The bending moments of both models with two different support conditions, i.e. fixed and allowable uplift on a rigid base, were compared. It was found that uplift can reduce the contribution of the fundamental mode and increase that of the higher modes. A SDOF model may overestimate the maximum bending moment at the base of a structure with uplift. The relationship between earthquake characteristic and structural response is discussed.

Keywords: higher vibration modes, structural uplift, experimental modelling, shake table test

1. INTRODUCTION

Structures often rely on their self-weight to withstand overturning. When a base overturning moment exceeds the available resistance due to the gravity load, a portion of the foundation may intermittently lose contact with the ground. In the conventional seismic design, this uplift is not encouraged and the foundation of a structure is assumed to be fixed at the base. With this assumption, a tall and slender building either has to be detailed to behave as a ductile structure or be designed to provide a very large reserve of elastic strength. Under a strong earthquake the damage to the building resulting from ductile behaviour may be difficult to repair and lead to the building demolition. In real scenario, minor uplift of foundations can occurred although foundations are not designed to uplift intentionally (Apostolou et al., 2006). Unless the foundation is massive, or tension piles are used, some uplift of foundations during a major earthquake cannot always be avoided.

Previous research showed that uplift of foundation may have a beneficial effect on seismic performance of structures. This benefit was initially recognized by Housner (1963). He had reported the good performance of several elevated water tanks during the 1960 Chile earthquake and used a rectangular rigid and free standing block to investigate the rocking behaviour of structure. Later, a number of numerical analyses of rocking rigid blocks were performed (Aslam et al., 1980, Yim et al., 1980, Yang et al., 2000). These studies showed that the rocking mechanism was capable of dissipating earthquake energy, and the rocking behaviour mainly depends on the dimensions of the structures. With the improvement of the experimental technique, investigations of uplift started to focus on the behaviour of physical models, e.g. Hung et al. (2011) investigated the rocking behaviour of bridge piers with and without plastic hinge using pseudo-dynamic and cyclic loading tests. The results of these experiments indicated that uplift could limit the earthquake force transmitted into the bridge pier and reduce its strength and ductility demand. In some recent studies of structural uplift, soil deformation beneath the foundation during an earthquake was also considered. Shake table tests of a small scale SDOF model resting on sand in a box were performed by Qin et al. (2011). This investigation also confirmed that uplift had a positive effect on structural performance.

Uplift was used to explain the survival of several ancient architectures in the past destructive earthquakes, e.g. the Greek and Roman monuments in the Eastern Mediterranean regions and Islamic minarets in the Middle Eastern regions (Yim et al., 1980). In 1980s the modern applications of uplift were introduced in New Zealand. The first structure designed with uplift potential for seismic protection was probably the reinforced concrete industrial chimney at the Air New Zealand Engineering Base in Christchurch (Sharpe and Skinner, 1983). Uplift was also allowed in the design of South Rangitikei Rail Bridge in Mangaweka (Chen et al., 2006) in New Zealand. A recent implementation of uplift was in the retrofitting of the Lions Gate Bridge in Vancouver, Canada, and this project was completed in 2001 (Dowdell and Hamersley, 2000).

An early design approach for rocking structures was proposed by Priestley et al. (1978). They suggested that a SDOF oscillator could be used to represent a rocking rigid block. Effective period and damping of the structure are determined according to its rocking behaviour. The maximum structural displacement can then be estimated by utilising displacement spectra. This design approach has been adopted by the FEMA 356 Guidelines (2000). In the design of the industrial chimney (Sharpe and Skinner, 1983), only the deformation related to the fundamental mode was considered. Recently, Kelly (2009) proposed design guidelines for rocking walls based on a rigid body assumption. Research on uplift so far mainly focused on rigid block or SDOF systems and only a few numerical analyses were performed using MDOF structures. Chopra and Yim (1985) initiated a numerical study on multi-storey buildings with uplift. In their study equivalent SDOF models were also considered for a comparison. Two spring-damper elements were used to simulate the flexible support condition. They found that the contribution of higher modes to structural response might increase when uplift was permitted. The results also showed that the base shear increased due to a contribution of higher modes. This increase was stronger in the case with uplift than without uplift.

In this work, higher mode contribution to the structural response with uplift is studied. A six-storey prototype was represented using scale MDOF and SDOF models. Shake table tests were performed. Responses of each model with different support conditions, i.e. fixed at the base and allowable uplift on the rigid base, were compared to reveal the higher mode contribution.

2. METHODOLOGY

2.1. Prototype

The prototype considered was a six-storey office with 3 m floor height, 48 m^2 floor area and 6 m × 6 m spread footing. The columns were 310UC118 for the top three levels and 310UC158 for the bottom three levels. The beams were 410UB53.7. A 170 mm thick reinforced concrete slab was on each floor. The beam and column arrangement is displayed in Figs. 2.1(a) and (b). According to the New Zealand Standard 1170.5 for earthquake actions (2004), the seismic masses were 24 tonnes for the roof and 29 tonnes for each floor. The lateral stiffness in direction *x* for the top and bottom three levels was 3.21E7 N/m and 4.44E7 N/m, respectively. The natural frequencies of the 1st to 6th modes are 1.47, 4.08, 6.62, 8.63, 9.90 and 11.52 Hz, respectively.

2.2. SDOF System

From the relationship between base shear and bending moment the effective mass m_n^* and height h_n^* for the n^{th} mode can be determined using the following equations:

$$m_n^* = \frac{(L_n^h)^2}{M_n} \text{ and } h_n^* = \frac{L_n^r}{L_n^h} \tag{2.1}$$

where

$$M_n = \sum_{j=1}^{N} m_j \varnothing_{jn}^2 \tag{2.2}$$

is the modal mass,

$$L_n^h = \sum_{j=1}^{N} m_j \emptyset_{jn} \tag{2.3}$$

is the coefficient associated with base translation and

$$L_n^r = \sum_{j=1}^{N} m_j \emptyset_{jn} h_j \tag{2.4}$$

is the coefficient associated with base rotation. In equations (2.2) to (2.4), m_j is the j^{th} seismic mass, h_j is the height of j^{th} floor above the ground, and $\emptyset_{jn}$ is j^{th} element of the n^{th} natural vibration mode $\boldsymbol{\emptyset}_n$.

To reveal the effect of higher modes, a SDOF system (Fig. 2.1(c)) only representing the fundamental mode of the MDOF system was considered as a reference system. The SDOF system has an effective mass m_1^* of 142.6 tonnes that was equal to 84 % of the prototype mass. The effective height h_1^* was 12.4 m. The SDOF system and the prototype have the same foundation.

The uplift resistance M_c due to gravity is defined by the following equation:

$$M_c = m_t * g * b/2 \tag{2.5}$$

where m_t is the total mass of the system, g is gravitational acceleration and b is the width of foundation mat in the excitation direction .

An extra mass of 26.4 tonnes was added on the foundation mat of the SDOF system to ensure that the uplift resistance of the both systems was equal.

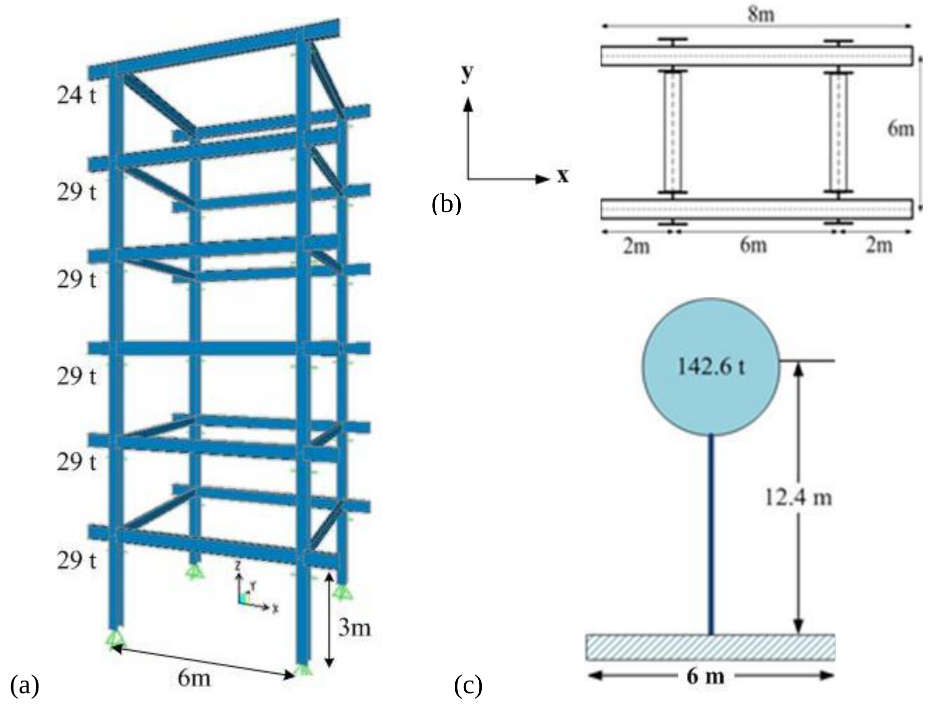

Figure 2.1. Prototype: (a) 3-D view, (b) the beam and column arrangement and (c) the SDOF system

2.3. Model Scaling

Since the late 60's and early 70's, shake table testing has been widely adopted in earthquake engineering research around the world as a result of the advance in the electro-hydraulic servo equipment development (Artistizabal-Ochoa and Clark, 1980). It is one of the most effective approaches to reproduce the earthquake imposed force on structures at the moment. However, the hydraulic power limits the capacity of the shake table. Therefore, for the majority of available shake tables the use of scale specimens is required.

Experimental modeling is based on the fact that the model and the prototype obey the law of similitude. To achieve this similarity, dimensional analysis based on Buckingham π theorem (Buckingham, 1914) was performed. Three basic dimensions: mass M, length L and time T in the dynamics of scale model were considered. All other dimensions of the selected physical parameters were then derived based on these three basic dimensions as displayed in Table 2.1. This chosen dimensionless group based on Cauchy Number (Equation 2.6) is established. While the conventional Cauchy Number is more appropriated for fluid mechanic, the reformulated one is more suitable for structural engineering.

$$\pi = \frac{a*m}{l*k} \tag{2.6}$$

where *t, a, l, m, k* are defined in Table 2.1.

By matching this dimensionless group, the scale factors of considered physical parameters were determined.

In this work, the dimensions of prototype and its equivalent SDOF system were scaled down 15 times to obtain the MDOF and SDOF experimental models. The models were constructed using aluminium angle sections for beams and PVC for columns. The rigid beam assumption was achieved. Only the lateral stiffness in the excitation direction of the prototype was considered (the *x* direction showed in Figs. 2.1(a) and (b)). The scale factors and dimensions are given in Table 2.1. Figs. 2.3(a) and (b) show the MDOF and SDOF models, respectively.

Table 2.1. Scale factors for different parameters

Parameters	Symbols	Dimensions	Scale Factors
Length	l	L	15
Mass	m	M	4800
Time	t	T	2
Acceleration	a	LT^{-2}	3.75
Stiffness	k	MT^{-2}	1200

2.4. Ground Excitations

To investigate the effect of earthquake characteristic, the ground excitations were simulated based on Japanese design spectra (JSEC, 2000; Chouw and Hao, 2005). These spectra were introduced after the 1995 Kobe earthquake. During this earthquake, ground motions were collected within the distance of 100 km from the epicentre. The spectra were constructed by enveloping the spectrum values of the severe ground motions recorded in the Kobe earthquake. There are three spectra for different soil conditions. In this project, ground excitations for hard and medium soil conditions were considered.

Fig. 2.2 displays the scale response spectra with a damping ratio of 4 %. The natural frequencies for the first two modes of MDOF experimental model are 2.94 Hz and 8.16 Hz, respectively after applying the scale factor. In this figure, they are indicated by vertical dashed lines. The spectrum values of hard soil condition excitation beyond 2.5 Hz are larger than those of medium soil condition excitation. Below 2.5 Hz, however, the spectrum value of the medium soil condition excitation is larger.

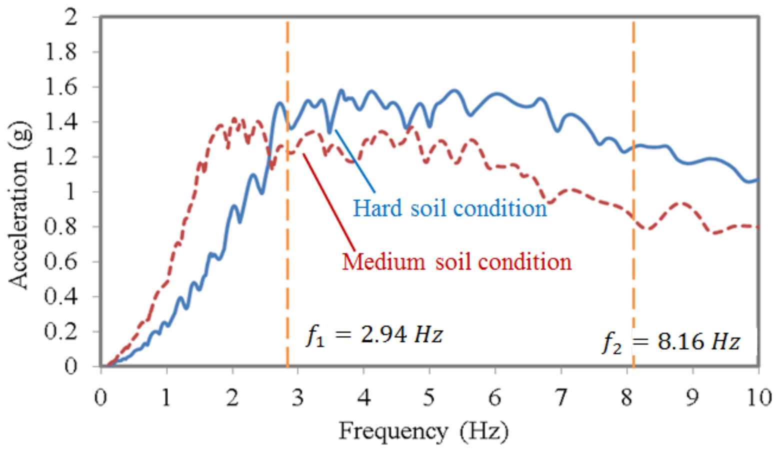

Figure 2.2. Reponses spectra of the simulated excitations with a damping ratio ζ = 4%

2.5. Test Setups

Strain gauges were glued for determining the bending moments at column bases of the two models (Fig. 2.3). The possible torsion movement of the models can be detected by comparing the bending moment developed in adjacent columns. Sand papers were attached on the rigid base and bottom of the foundation to increase the friction at the footing-base interface. Thus, a sliding during uplift could be minimised.

(a) (b)

Figure 2.3. (a) MDOF and (b) SDOF models with circles indicating the locations of strain gauges.

3. RESULTS AND DISCUSSION

3.1. Uplift effect on the structural response

The bending moments at the right front supports (see Fig. 2.3, circled by dashed lines) were considered. Fig. 3.1 shows the Fourier spectra of the bending moments due to the hard soil condition excitation. In the response of MDOF model with fixed base, the Fourier amplitude at the fundamental

frequency of 2.93 Hz is 149 Nms (see Fig. 3.1(a)). On the other hand, with allowable uplift, the amplitude decreases by 59.7 % comparing to the fixed-base response. However, an adverse effect of uplift on the Fourier amplitude was observed at the adjacent lower frequency of 2.56 Hz, where a 60.2 % increase can be observed. Although the Fourier amplitude at the lower frequency increases, the peak amplitude is still less than that of the fixed-base structure. When uplift takes place, the amplitude is 89 Nms at the frequency of 2.56 Hz, while the maximum fixed-base amplitude is 67 % larger.
Similar effect of uplift is observed in the SDOF model response (see Fig. 3.1(b)). In the fixed base case the peak Fourier amplitude of the bending moment is 147 Nms at 2.93 Hz. Uplift causes a 40 % reduction of Fourier amplitude at 2.93 Hz and a 64 % increase at 2.56 Hz. These contrasts in the bending moment development of the both models confirm that uplift can reduce the fundamental mode response, and thus decrease the forces activated in the structure.

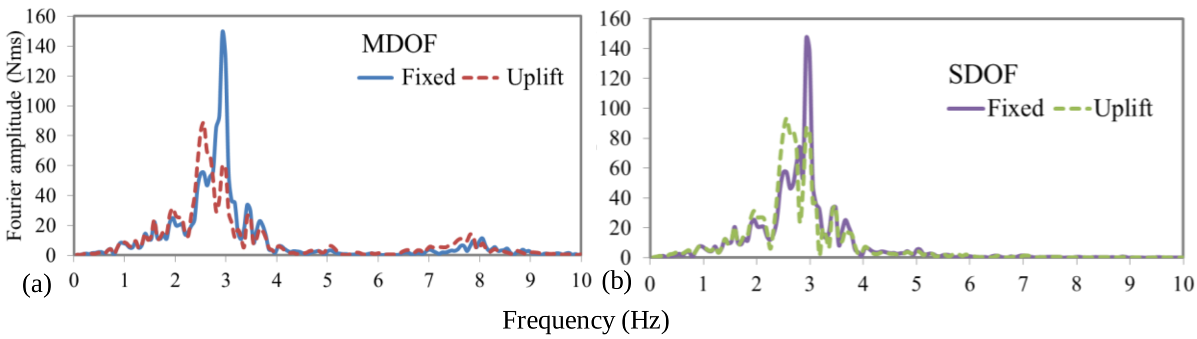

Figure 3.1. Fourier spectra of the bending moments at the base of (a) MDOF and (b) SDOF models due to the hard soil condition excitation

3.2. Effect of higher modes

As expected the SDOF model cannot simulate the higher frequency responses. In the response of the MDOF model (Fig. 3.1(a)), considering frequency range of 6 Hz to 10 Hz (where the second mode is located), the dominant frequency shifts from 8 Hz to 7.8 Hz due to uplift. The peak Fourier amplitude in this region also increases from 11 Nms to 14 Nms. Uplift causes a slight amplification of higher frequency response.

Fig. 3.2 displays the bending moment time history of both models. In general, the bending moments in the model with allowable uplift are generally smaller than the model with fixed base. As shown in Fig. 3.2(a), the influence of higher modes on the response of the MDOF model is more obvious when uplift was permitted, especially in the time window between 7.2 s and 8.2 s. At 7.2 s, the bending moment of the SDOF model with uplift is 68 Nm (Fig. 3.2(b)), while the moment is 44 Nm when both uplift and higher mode contributions are considered (Fig. 3.2(a)). Similar reduction due to higher modes can also be observed at 7.4, 7.6, 7.7, 7.9 and 8 s referring to the dashed lines in Figs. 3.2 (a) and (b).

3.3. Influence of earthquake characteristic

Table 3.1 lists the reduction of the maximum bending moment due to uplift under hard and medium soil condition excitations. The reduction was calculated from the following equation:

$$Reduction(\%) = \frac{M_{fixed} - M_{uplift}}{M_{uplift}} \times 100 \quad (3.1)$$

where M_{fixed} and M_{uplift} are the maximum bending moments without and with uplift effect, respectively.

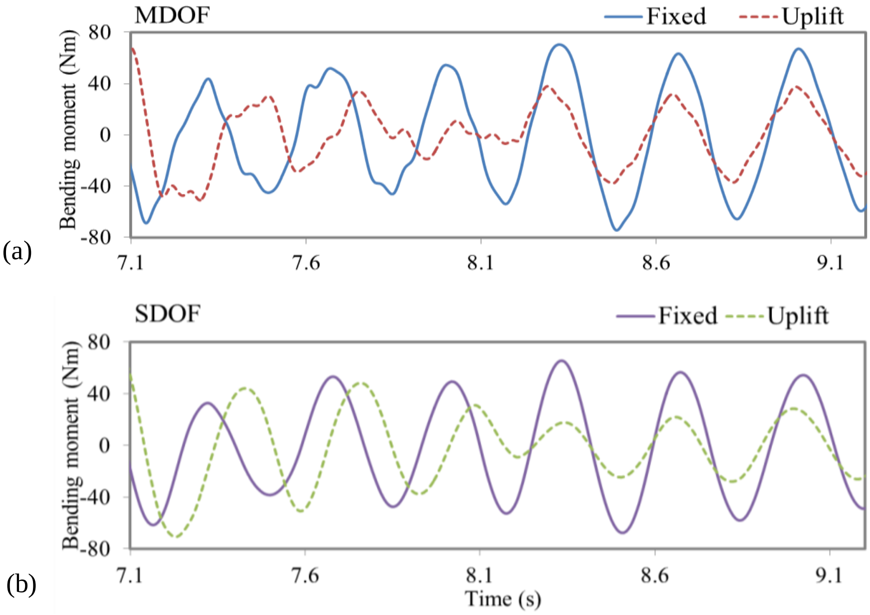

Figure 3.2. The bending moments of (a) MDOF and
(b) SDOF models with different support conditions due to the hard soil condition excitation

A stronger reduction in the bending moment of the MDOF model is found compared to that of the SDOF model. The results show that a SDOF model will overestimate the activated maximum bending moment at the base. The full potential of uplift to reduce the force activated in the structure cannot be revealed by a SDOF model.

A comparison between the reduction values of the two models shows that, the hard soil condition ground excitation causes a larger reduction in the maximum bending moment comparing to medium soil condition excitation. When the higher mode contribution is included, a larger difference in reductions due to the two excitations is observed.

Table 3.1. Reduction of the maximum bending moments due to uplift under different excitations

	Hard soil condition	Medium soil condition	Difference between two excitations
SDOF	10.57 %	7.24 %	3.33 %
MDOF	19.20 %	15.25 %	3.95 %

Fig. 3.3 displays the Fourier spectra of the bending moments due to the medium soil condition excitation. The uplift effect on the bending moment is similar to that in the hard soil condition case (see Fig. 3.1). In the MDOF (see Fig. 3.3(a)), the peak Fourier amplitude at 2.81 Hz reduces by 28 % due to uplift. However, the Fourier amplitudes increase by 34.5 % and 27 % at 2.63 Hz and 2.44 Hz, respectively. When SDOF is considered, there is a 43 % reduction of Fourier amplitude at 2.8 Hz, and 82 % and 84 % increases at 2.63 Hz and 2.44 Hz, respectively. Fig. 3.4 shows that the increase at lower frequencies is greater than the decrease at the fixed-base fundamental frequency. However, as shown in Fig. 3.1, the increase at lower frequency is less under the hard soil condition excitation.

These observations can be explained using the response spectra in Fig. 2.3. During uplift, the fundamental frequency of the structure is likely to shift temporarily to lower frequency range. The spectrum values of medium soil condition excitation are greater than that of the hard soil condition excitation in the frequency range below 2.5 Hz. Therefore the impact of hard soil condition excitation decreases, opposite to the medium soil condition excitation, where Fourier amplitude increases when uplift takes place. To clarify a possible link between the structural responses including uplift effect and the spectrum values further investigations are required.

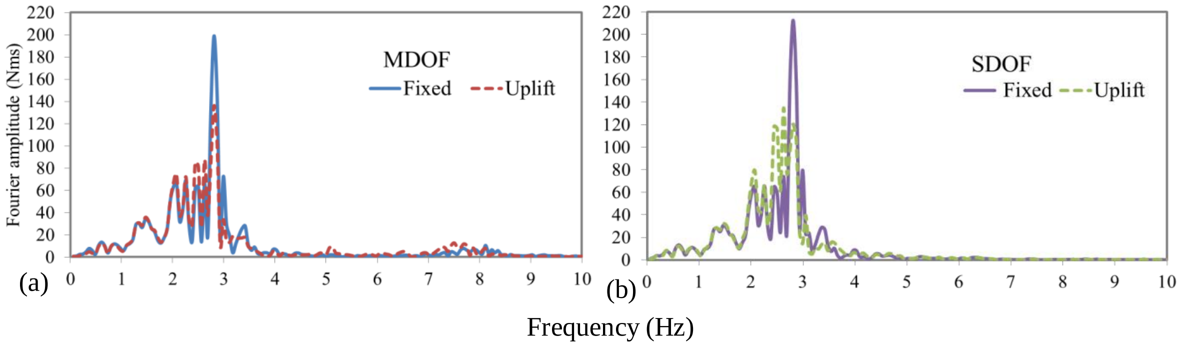

Figure 3.3. Fourier spectra of the bending moments as the (a) MDOF and (b) SDOF model base due to the medium soil condition excitation

4. CONCLUSIONS

The influence of higher modes on the structural response including uplift effect is addressed. Two scale models were used. The ground excitations were simulated based on the Japanese design spectra for hard and medium soil conditions. A shake table was used to simulate the ground excitation.

The investigation reveals:

- Uplift temporarily shifts the natural frequencies of the structure to a lower frequency range.
- Uplift can reduce the contribution of the fundamental mode and increase that of the higher mode.
- Larger reduction of maximum bending moment at support due to uplift is observed when higher mode contribution is included.
- In the investigated cases the largest reduction of the maximum bending moment due to uplift was observed when a higher mode contribution and hard soil condition excitations are considered.

REFERENCES

Apostolou, M., Gazetas, G. and Garini, E. (2006). Seismic response of slender rigid structures with foundation uplifting. *Soil Dynamics and Earthquake Engineering*, **27**, 642-654.

Artistizabal-ochoa, J. D. and Clark, A. J. (1980). Large-scale earthquake simulation tables. *7th World Conference on Earthquake Engineering.* Istanbul, Turkey.

Aslam, M., Scalise, D. T. and Godden, W. G. (1980). Earthquake rocking response of rigid bodies. *ASCE Journal of the Structural Division,* **106,** 377-392.

Buckingham, E. (1914). Illustrations of the use of dimensional analysis. *On Physically Similar Systems, Physics Review,* **4,** 354-377.

Chen, Y.-H., Liao, W.-H., Lee, C.-L. and Wang, Y.-P. (2006). Seismic isolation of viaduct piers by means of a rocking mechanism. *Earthquake Engineering and Structural Dynamics,* **35,** 713-736.

Chopra, A. K. and Yim, S. C. S. (1985). Simplified earthquake analysis of multistory structures with foundaiton uplift. *Journal of Structural Engineering,* **111,** 906-930.

Chouw, N. and Hao, H. (2005). Study of SSI and non-uniform ground motion effect on pounding between bridge girders. *Soil dynamics and Earthquake Engineering,* **25,** 717-728.

Dowdell, D. J. and Hamersley, B. (2000). Lion's Gate Bridge north approach: seismic retrofit. *Behaviou of Steel Structures in Seismic Areas:Proc., 3rd Int. Conf.:STESSA 2000***,** Balkema, The Netherlands**,** 319-326.

FEMA 356. (2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, American Society of Civil Engineers for the Federal Emergency Management Agency.

Housner, G. W. (1963). Behavior of inverted pendulum structures during eathquakes. *Bullentin of the Seismological Society of America,* **53,** 403-417.

Hung, H.-H., Liu, K.-Y., Ho, T.-H. and Chang, K.-C. (2011). An experimental study on the rocking response of bridge piers with spread footing foundation. *Earthquake Engineering and Structural Dynamics,* **40,** 749-769.

Japan Society of Civil Engineers (JSCE) (2000). Earthquake resistant design codes in Japan, Tokyo, Maruzen.

Kelly, T. E. (2009). Tentative seismic design guidelines for rocking structures. *Bullentin of the New Zealand Society for Earthquake Engineering*, **42:4**, 239-274.

New Zealand Standard NZS1170.5 (2004). *Structural Design Actions Part 5: Earthquke Action*.

Priestley, M. J. N., Evison, R. J. and Carr, A. J. (1978). Seismic response of structures free to rock on their foundations, *Bullentin of the New Zealand National Society for Earthquake Engineering*,**11:3**, 141-150.

Qin, X., Chen, Y. and Chouw, N. (2011). Experimental investigation of nonlinear structure-foundation-soil interaction. *9th Pacific Conference on Earthquake Engineering*, University of Auckland, New Zealand.

Sharpe, R. D. and Skinner, R. I. (1983). The seismic design of an industrial chimney with rocking base. *Bullentin of the New Zealand Society for Earthquake Engineering*, **16,** 98-106.

Yang, Y., Hung, H. and He, M. (2000). Sliding and rocking response of rigid blocks due to horizontal excitations. *Structural and Engineering and Mechanics*, **9:1**, 1-16.

Yim, S. C. S., Chopra, A. K. and Penzien, J. (1980). Rocking response of rigid blocks to earthquakes. *Earthquake Engineering and Structural Dynamics*, **8,** 565-587.