Recommendations for seating length of seismically designed bridge girders

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ABSTRACT: Asynchronous movements of bridge girders have been observed in almost all major earthquakes. Such relative girder displacements in excess of the designed seismic gaps or seating lengths will respectively result in pounding and girder unseating. Spatial variation of ground excitations is one of the major causes of relative displacements not yet incorporated in most of current bridge specifications. This paper, considers the effects of relative displacements due to spatial variation of ground motions on a three-segment bridge. Ground excitations of soft and shallow soil and strong rock conditions were simulated based on the New Zealand design spectra. Excitations with high, intermediate and weak correlation were considered. The results were compared with those obtained from the current New Zealand Transport Agency Bridge manual and showed that the current manual can underestimate the seating length required.

1 INTRODUCTION

During severe earthquakes, bridges often suffer damage at expansion joints due to large relative displacements. Such damage was observed in the 1999 Ji-Ji earthquake (Hamada et al. 1999), the 2008 Wenchuan earthquake (Lin et al. 2008) and the 2010 Chile earthquake (Arias and Buckle 2010). Opening relative displacements in excess of the seating lengths will result in girder unseating whereas excessive closing relative displacement will cause pounding. Although damage due to pounding is not usually as catastrophic as that due to unseating, pounding should still be avoided as it can also contribute to unseating of bridge spans (Raheem 2009).

Relative displacements arise from different dynamic properties of adjacent structures, spatial variation of ground motions and soil-structure interaction (SSI) (Chouw and Hao 2008). Most current bridge design codes tend to neglect the effects of spatial variation of ground motions and SSI suggesting their minimum seating length recommendations could be inadequate. Chouw and Hao (2006) evaluated the Japan Road Specification (JRA) (2004) and found that the recommended minimum seating length was insufficient when spatially varying ground motions were considered. Sextos and Kappos (2009) reviewed the Eurocode 8-2 provisions (BS EN 1998-2 2005) and concluded that Eurocode 8-2 provisions could sometimes significantly underestimate the seating length demanded if spatially varying ground motions were neglected.

New Zealand is an earthquake-prone country and some bridge damage was reported in the 2011 Christchurch earthquake (Chouw and Hao 2012). Although no girder unseating was observed, it is still necessary to examine the current New Zealand bridge manual for the minimum seating length to avoid possible future catastrophic failures. The most up-to-date New Zealand bridge manual was issued by the New Zealand Transport Agency (NZTA) in 2005 and its provisions for minimum seating length were also based on uniform ground motions (i.e. ignoring spatial variation). This research compares the results from three experimental studies using seating lengths suggested by the NZTA Bridge manual to examine the adequacy of its provisions. Pounding was also considered in this research.

2 CURRENT DESIGN REGULATIONS

As more research addresses the characteristics of earthquake waves and the influence of soil conditions along the wave path, it is found that the spatial variation of the ground motions is common especially for long bridges. This is caused by finite speed of seismic waves, different site response and coherency loss of the waves (Kiureghian and Neuenhofer 1992). Although this phenomenon has received wide recognition, its adoption is still rare in most of the current bridge design codes owing to its inherent complexity and the substantially greater design effort needed compared with current practice. Meanwhile, researchers have found that uniform excitation is not easily justified in some cases, and can result in underestimation of the collision potential, causing damage to the deck joint or even loss of span as observed in almost all major earthquakes in the past.

Ideally, the bridge design codes should incorporate the effect of spatial variation of excitations. However, to the authors' best knowledge, only Eurocode 8 (BS EN 1998-2 2005), and Japan specifications (JRA 2004) address this problem. Eurocode 8 suggests the distance beyond which ground motions may be considered uncorrelated and defines the relative displacement for all supports of the bridge and the absolute displacement of a support considering the influence of ground displacements occurring in opposite directions at adjacent piers. The Japanese code addresses this problem by suggesting calculations of minimum seating length using a set of empirical formulae. Other bridge design codes, e.g. American Association of State Highway and Transportation Officials (AASHTO) (1998), California Department of Transportation (CALTRANS) (2010) and NZTA (2005) do not have specific guidelines to address this problem. An important reason for very limited implementation of spatially varying excitation is the highly complex combined effects between earthquake excitations, dynamic characteristics of the bridge and SSI. This leads to contradictory conclusions by different researchers regarding the impact of spatially varying ground motions on bridge structures. Owing to the multi-parametric nature and the complexity of spatially varying ground motions, the development of specific design provisions is not yet implemented in any modern seismic codes. Most design codes, such as the AASHTO, CALTRANS and the NZTA Bridge manual still assume uniform ground motions in seismic design analysis.

- The AASHTO (1998) bridge design specification prescribes a minimum seating length $S_{\mathrm{E,min}}$ (m) for the movement between the girders and between girder and adjacent abutment, defined by a function of the span L_{s} (m), the height of the column or pier H (m), and the skew angle α of the support (degrees), based on the following relationship:

$$S_{E,\min} = (0.203 + 0.00167\, L_s + 0.00666\, H)(1 + 0.000125\, \alpha^2) \tag{1}$$

- The Japan Code specifies the required seating length as the following:

$$S_E = u_{rel} + u_G \geq S_{E,\min} \tag{2}$$

$$S_{E,\min} = 0.7 + 0.005\, l \tag{3}$$

$$u_G = \varepsilon_G L \tag{4}$$

where u_{rel} is the differential displacement between the superstructure and substructure (m), u_{G} is the relative displacement of the ground occurring due to ground deformation between piers (m). l is the length of the effective span (m). For hard, medium and soft soil ε_{G} has the value of 0.0025, 0.00375 and 0.005, respectively. L is the distance between two substructures.

- The Eurocode 8 defines the minimum overlap lengths for end support on an abutment as follows:

$$S_{E,\min} = l_m + d_{eg} + d_{es} \tag{5}$$

$$d_{eg} = \varepsilon_s L_{eff} \leq 2 d_g \tag{6}$$

$$\varepsilon_s = \frac{2 d_g}{L_g} \tag{7}$$

$$d_g = 0.025\, a_g S T_C T_D \tag{8}$$

where l_{m} of 40 cm is the minimum support length securing the safe transmission of the vertical reaction; d_{eg} is the effective relative displacement of the span and the abutment due to differential seismic ground displacement; d_{g} is the design value of the peak ground displacement; a_{g} is the design ground acceleration; S is soil factor and with T_C and T_D together defined in Table 1, where type 2 spectrum is recommended only for regions where the design earthquake has a surface wave magnitude of less than 5.5; d_{es} is the effective seismic displacement of the support due to the deformation of the structure; L_g is the characteristic distance beyond which the ground motions may be considered as completely uncorrelated, and is defined in Table 1; L_{eff} is the is the effective length of deck, taken as the distance from the deck joint in question to the nearest full connection of the deck to the substructure. If the deck is fully connected to more than one pier, then L_{eff} shall be taken as the distance between the support and the centre of the group of piers. In this context, 'full connection' means a connection of the deck or deck section to a substructure member, either monolithically or through fixed bearing, seismic links, or shock transmission units.

Table 1. Recommended values of the parameters for types 1 and 2 elastic response spectrum (reproduced from Eurocode 8 (BS EN 1998-2 2005)).

Case	**S**		**T_C (s)**		**T_D (s)**		**L_g (m)**
Spectrum type	**1**	**2**	**1**	**2**	**1**	**2**	**1 and 2**
Soil class A	1	1	0.4	0.25	2	1.2	600
Soil class B	1.2	1.35	0.5	0.25	2	1.2	500
Soil class C	1.15	1.5	0.6	0.25	2	1.2	400
Soil class D	1.35	1.8	0.8	0.3	2	1.2	300
Soil class E	1.4	1.6	0.5	0.25	2	1.2	500

- The NZTA Bridge manual specifies the provisions for the minimum seating length to prevent span loss. The provision, in the absence of a linkage system is as follows:

$$S_{min} = 2.0E + 0.1 \geq 0.4 \text{ m} \tag{9}$$

where S_{min} is the minimum seating length; E is relative movement between span and support. For more details, please refer to the NZTA Bridge manual, Section 5.6.2.

As can be seen from Equation 1, the AASHTO code suggests the minimum seating length by a function of only the span length and the pier height. It does not consider the influences of the ground motions and dynamic properties of the bridge. It implies that no matter how significant the earthquake is and whether the bridge is stiff or flexible, as long as the bridge has the same span and pier height, the required seating length will be the same. In reality, this is not the case. Therefore, the recommendation of minimum seating length by the AASHTO is less holistic. The current Japan design specification considers the influence of the frequency ratio of the neighbouring structures and also implicitly accounts for the effect of spatially varying ground displacements. However, according to the research by Chouw and Hao (2006), it can still underestimate the necessary seating length, especially when the adjacent structure is more flexible. Eurocode 8 is currently the only seismic code worldwide that provides a clear and detailed framework for considering the effect of spatial variation of ground motions in bridge design. In the calculation of minimum overlap lengths, the relative displacement of the adjacent structures accounting for spatial variation is incorporated in Equation 5.

3 EXPERIMENTS

To evaluate the seating length requirement of the NZTA Bridge manual, a three-segment bridge test was conducted. The prototype structure was the Newmarket Viaduct replacement bridge currently under construction in Auckland, New Zealand. The bridge segment considered a length of 100 m and a pier height of 15.5 m. The models were constructed based on the similitude scaling laws (Dove and Bennett 1986) using polyvinylchloride (PVC). More information about the prototype structure and the scaling procedure are reported in (Li et al. 2012).

This experiment considered a bridge with three identical segments subjected to spatially varying ground motions with pounding. The abutments were not considered in this study. The segments, each with a scale ratio of 1:125, were axially aligned with zero gaps. With fixed foundations, the segments were found to have fundamental frequencies of 1.965 Hz, 1.966 Hz and 1.965 Hz. With a time scale ratio of 2, the models correspond to the prototype structure with a fundamental frequency of 1 Hz. Elastic pounding forces were measured by the measuring head which is an elastic steel spring strip with a strain gauge glued to the non-contact surface. The pounding heads were fabricated by a PVC cylinder. Figure 1 depicts the schematic drawing of the pounding heads and the measuring heads. More details are reported in (Li et al. 2012).

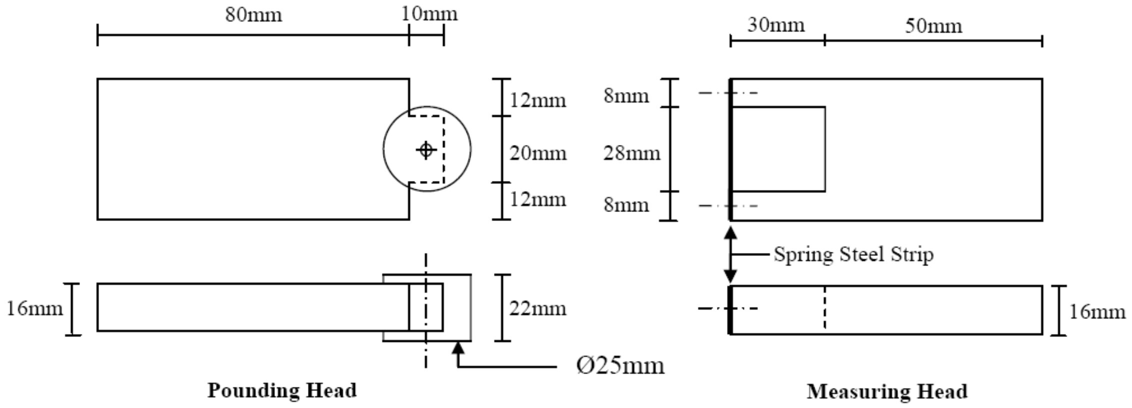


Figure 1: The measuring heads and pounding heads.

Figure 2 shows the experimental setup. The mass blocks (the green cylinders) were used to compensate for the mass of sandboxes which would be included in later tests, in order to present the shake tables with constant mass, ensuring the same table movement. In addition, a two-segment bridge test was also conducted by removing the third segment. Spatially varying ground motions and time-delayed ground motions were considered. For the time-delayed ground motions, an apparent wave velocity of 500 m/s was considered. The relative displacements were measured using the displacement transducers which are shown in Figure 2.

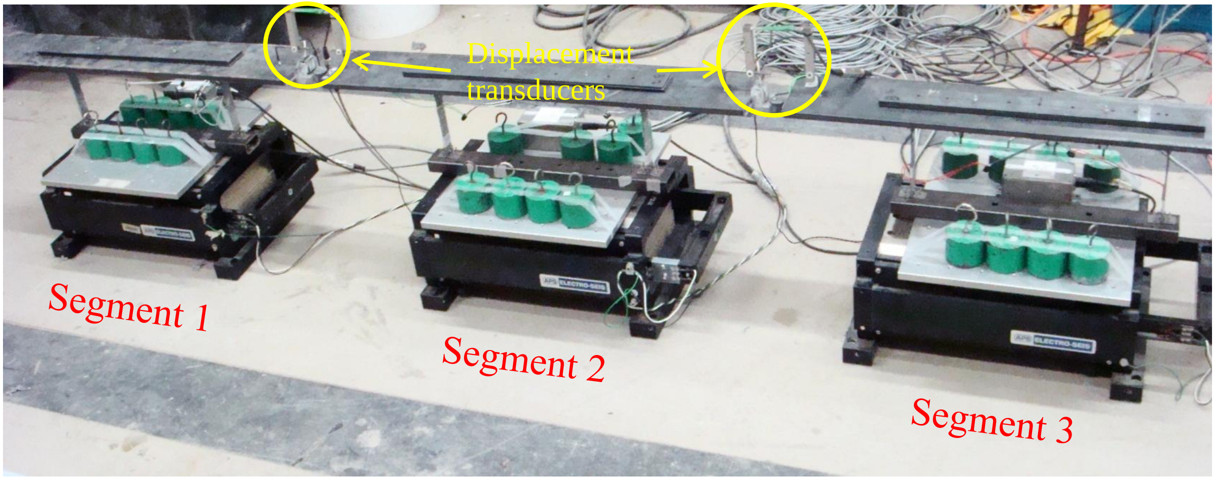


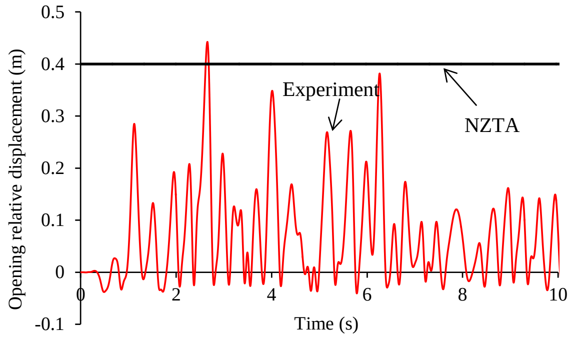
Figure 2: Setup for the three-segment bridge test.

The spatially varying ground motions used in this study were derived based on the New Zealand design spectra (NZS 1170.5 2004) and the empirical coherency loss function proposed by Bi and Hao (2012). More details about the parameters used in the coherency loss function for these experimental studies are provided in (Li et al. 2012). According to NZS 1170.5, soft soil (Class D), shallow soil (Class C) and strong rock (Class A) conditions were considered. The simulated ground motions correspond to a return period of 1000 years. Each experiment considered 20 ground motions for every soil condition. To address the coherency loss effect, high, intermediate and weak correlations of ground motions were considered for the ground motions of soft soil condition. For shallow soil and strong rock condition, only high correlation was considered.

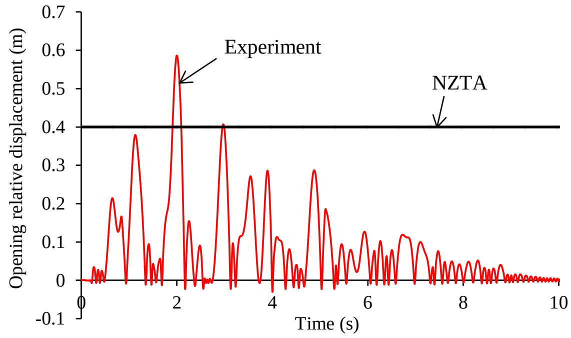
4 RESUTLS AND DISCUSSION

The conservatism of the approach proposed by the NZTA Bridge manual is illustrated through comparison between the experimental maximum relative displacements and the calculated seating length using Equation 9. Because the NZTA Bridge manual assumes that the ground excitation along the bridge supports is uniform, it is necessary to examine whether the minimum seating length provisions are still adequate in case of spatially varying ground motions.

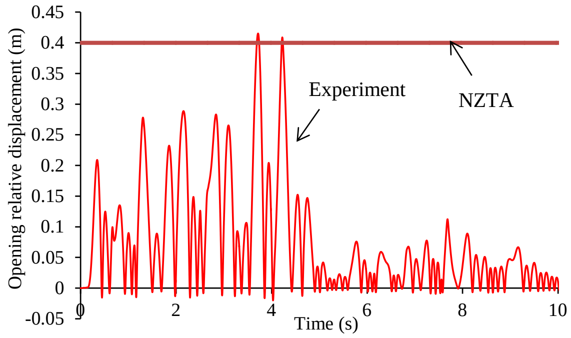
Figure 3 compares the maximum relative opening displacements obtained from the three-segment bridge test due to excitations of different soil conditions and coherency losses with those according to the NZTA recommendation for the minimum required seating lengths. Pounding was considered. When two identical segments are subjected to uniform excitations, the relative displacement between these considered segments should be zero. Hence, the minimum required length should be 40 cm according to Equation 9. All the figures involving ground motions of soft soil condition show that their maximum relative displacements between identical segments are higher than the NZTA recommended values, which reveals the possibility that the NZTA Bridge manual could underestimate the minimum seating length between bridge structures that experience earthquakes of soft soil condition regardless of coherency and number of segments involved. The recorded maximum relative displacement due to pounding involving three segments under ground motions of shallow soil condition also exceeds the recommended seating length, although only by a small amount. From the cases considered in this study, only the relative displacement due to ground motions of strong rock conditions (Figure 3(d)) can be reasonably safely accommodated by the seating length recommended by NZTA Bridge manual. According to Figure 3(f), the underestimation could be more than 50% for the case of a two-segment bridge subject to spatially weakly correlated excitation of soft soil condition.



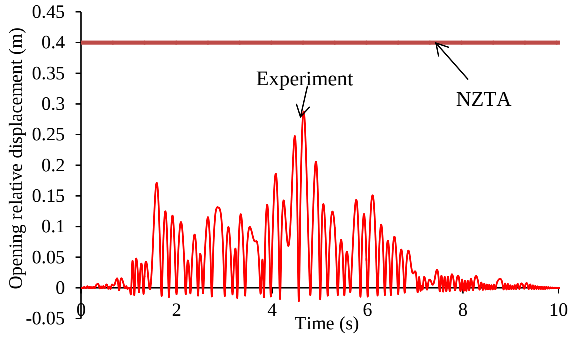
(a) Spatially varying, soft soil, highly correlated, segments 1 and 2



(b) Spatially varying, soft soil, weakly correlated, segments 1 and 2



(c) Time delayed, shallow soil, highly correlated, segments 1 and 2



(d) Spatially varying, strong rock, highly correlated, segments 1 and 2

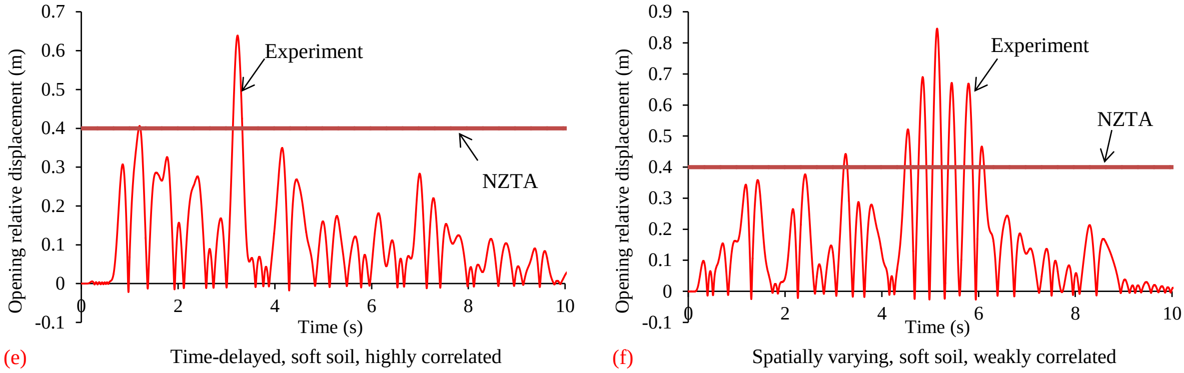


Figure 3: Minimum overlaps according to the NZTA recommendation and experimental results. (a)–(d) considering three-segment bridge and (e) and (f) considering two-segment bridge under different ground motions with pounding effect

Table 2. Comparison between seating length suggested by the NZTA Bridge manual and the recorded average maximum relative displacements with pounding effect

Opening relative displacement (cm)	Soil conditions of ground motions		
	Soft soil	**Shallow soil**	**Strong rock**
Two segments and spatial variation	62.9	35.1	23.7
Two segments and time delay only	65.7	37.5	25.2
Three segments and spatial variation	53.9	29	19.3
Three segments and time delay only	55.1	33.7	22.9
Minimum seating length from NZTA		40	

Table 2 shows that the NZTA Bridge manual underestimates the necessary seating length between adjacent segments in the situation of spatially varying excitation of soft soil condition. By following Equation 9, the minimum seating length was found to be 40 cm. Table 2 confirmed that the current NZTA Bridge manual unable to provide sufficient seating length for bridges, especially when ground motions of soft soil condition are anticipated. Although the average maximum relative displacement (33.7 cm) of a three-segment bridge due to time-delayed ground motions of shallow soil condition is below 40 cm, the potential of girder unseating can still be seen from Figure 3(f). Note the Table 2 only shows the experimental average maximum relative displacements for highly correlated support excitations. By comparing between Figures 3(a) and (b), weakly correlated excitations would result in larger relative girder displacements than highly correlated excitations, inducing higher risk of girder unseating. Therefore, the current provisions of the NZTA Bridge manual for minimum seating length should be amended.

5 CONCLUSIONS

The provisions of the NZTA Bridge manual for estimating minimum seating length were reviewed based on the physical results obtained from the a series of experiments. A total of 8140 experiments were performed. The effect of spatial variation of ground motions was addressed. By comparing the measured average maximum relative displacement resulting from each study with the corresponding estimation based on the NZTA Bridge manual, the following conclusions were reached:

1. The current NZTA approach is not capable of predicting sufficient seating length between adjacent structures if spatially varying ground motions and pounding is expected. The relative displacement will be underestimated when only uniform excitation is considered and adjacent segments have the same fundamental frequencies.

2. Spatial variation of ground motions is critical in determining relative displacement, and hence should be considered in the current NZTA Bridge manual.

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REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 1998. Load and resistance factor design (LRFD) specifications for highway bridges. Washington DC.

Arias, J. & Buckle, I. 2010. The M8.8 Offshore Maule Earthquake Chile Seismic Performance of Transportation Systems. http://nees.unr.edu/outreach/eeri/Chile_EQ_UNR_Presentation.pdf.

Bi, K. & Hao, H. 2012. Modelling and simulation of spatially varying earthquake ground motions at sites with varying conditions, *Probabilistic Engineering Mechanics*, 29: 92-104.

California Department of Transportation (CALTRANS). 2010. Seismic Design Criteria. Design manual - version 1.1. California Department of Transportation. Sacramento, California, USA.

Chouw, N. & Hao, H. 2006. Comments on bridge girder seating length under current design regulations. *JSCE Journal of Applied Mechanics*, 9: 691-199.

Chouw, N. & Hao, H. 2008. Seismic design of bridge structures with allowance for large relative girder movements to avoid pounding. *Bulletin of the New Zealand Society for Earthquake Engineering*, 42(2): 75-85.

Chouw, N. & Hao, H. 2012. Pounding damage to buildings and bridges in the 22 February 2011 Christchurch earthquake. *International Journal of Protective Structures*, 3 (2): 123-139.

Dove, R. & Bennett, J. 1986. Scale modeling of reinforced concrete category I structures subjected to seismic loading. *NUREG/CR-4474*. Los Alamos, New Mexico: Los Alamos National Laboratory.

Eurocode 8. 2005. Design of structure for earthquake resistance. Part 2 – Seismic design of bridges. *British Standard*. London.

Hamada, M., Nakamura, S., Ohsumi, T., Megro, K. & Wang, E. 1999. The 1999 Ji-Ji earthquake, Taiwan-Investigation into damage to civil engineering structure, Japan Society of Civil Engineering, 160pp.

Japan Road Association (JRA). 2004. Specifications for highway bridges – Part V Seismic Design 5th edition.

Kiureghian, AD. & Neuenhofer, A. 1992. Response spectrum method for multi-support seismic excitations. *Earthquake Engineering and Structural Dynamics* 21: 713–740.

Li, B., Bi, K., Chouw, N., Butterworth, JW. & Hao, H. 2012. Experimental investigation of spatially varying effect of ground motions on bridge pounding, *Earthquake Engineering and Structural Dynamics*, 41(14): 1959-1976.

Lin, C.J., Hung, H., Liu, Y. & Chai, J. 2008. Reconnaissance report of 0512 China Wenchuan earthquake on bridges. Proceedings of the 14th world conference on earthquake engineering, Paper 31, Beijing.

New Zealand Standards (NZS1170.5), 2004. Earthquake Actions.

New Zealand Transport Agency. 2005. Earthquake resistance design. *Bridge manual*. Wellington, New Zealand.

Raheem SEA. 2009. Pounding mitigation and unseating prevention at expansion joints of isolated multi-span bridges. *Engineering structures*. 31(10): 2345-2356.

Sextos, AG. & Kappos, AJ. 2009. Evaluation of seismic response of bridges under asynchronous excitation and comparisons with Eurocode 8-2 provisions. *Bulletin of Earthquake Engineering*, 7: 519-545.