Seismic design of New Zealand highway bridges under spatially varying ground excitations

December 2012

B Li, J Butterworth and N Chouw

The University of Auckland, Department of Civil and Environmental Engineering

NZ Transport Agency research report 504

ISBN 978-0-478-40700-6 (electronic)
ISSN 1173-3764 (electronic)

NZ Transport Agency
Private Bag 6995, Wellington 6141, New Zealand
Telephone 64 4 894 5400; facsimile 64 4 894 6100
research@nzta.govt.nz
www.nzta.govt.nz

Li, B, J Butterworth and N Chouw (2012) Seismic design of New Zealand highway bridges under spatially varying ground excitations. *NZ Transport Agency research report 504.* 81pp.

Assoc Prof Nawawi Chouw, The University of Auckland, Department of Civil and Environmental Engineering, Auckland Mail Centre, Private Bag 92019, Auckland 1142, New Zealand, Email: n.chouw@auckland.ac.nz, Telephone: 64 9 923 3512

Keywords: fundamental frequency, NZTA *Bridge manual*, relative displacement, seismic design, spatially varying ground excitations

An important note for the reader

Acknowledgements

The authors would like to acknowledge the great support of New Zealand Transport Agency, the students and technicians of the Department of Civil and Environmental Engineering for their assistance in the experiments and Dr. Kaiming Bi and Prof. Hong Hao of the University of Western Australia for their help in the simulation of the ground motions. The authors would like to acknowledge the contribution of the steering group members, Rudolph Kotze, Ian Billings, Raed El Sarraf, Quincy Ma and Charles Clifton. The authors also would like to thank the peer reviewers, Dr. Peter Lipscombe of URS New Zealand Limited and Dr. Sulo Shanmuganathan of OPUS, Auckland, for providing their constructive comments and suggestions which have improved the clarity of the report.

Abbreviations and acronyms

AASHTO	American Association of State Highway and Transportation Officials
CALTRANS	California Department of Transportation
JRA	Japan Road Association
MEJ	modular expansion joint
MSSS	multiple span simply supported
NZS	New Zealand Standard
NZTA	New Zealand Transport Agency
PVC	polyvinylchloride
SDOF	single degree of freedom
SRSS	square root of sum of square
SSI	soil-structure interaction

Contents

Executive summary

Pounding between adjacent bridge structures with insufficient separation distance has been identified as one of the primary causes of damage in many major earthquakes. It takes place because the closing relative movement is larger than the structural gap provided between the structures. More severely, loss of bridge span could happen if the relative opening movement is larger than the girder seating length. Research carried out between 2009 and 2011 evaluated the influencing factors of differential movement of bridge segments through mainly experimental analyses. Relative displacements of the adjacent bridge segments can cause pounding and unseating of the decks, which is a potential threat to human life. To enhance the integrity of new bridges, the NZTA *Bridge manual* needs to provide adequate guidance for the minimum seating length required to prevent span fall-off from supports. To prevent damage due to pounding between adjacent structures, this report proposes an approach using modular expansion joints.

The objectives of the research were to:

1 Simulate spatially varying ground motions for New Zealand soil conditions.

2 Analyse bridge seismic performance under spatially varying ground motions.

3 Identify the dominant influence factors for relative movement of multi-span bridges.

4 Evaluate the NZTA *Bridge manual*'s suggested seating length between adjacent bridge structures.

5 Recommend the calculation of the minimum seating length that may be implemented on a bridge.

This project focused on the influence of the spatial variation of ground motions on the bridge response because most current design specifications ignore this significant factor. Three shake tables and three almost identical models with a scale ratio of 1:125 were used in the investigations. The outcomes are recommendations for the NZTA *Bridge manual*. The experimental bridge models were designed based on the dynamic properties of the Newmarket Viaduct replacement bridge using the similitude scaling rules. Relative displacements and pounding forces between one bridge with movable abutments and between two and three segments with spatially varying ground motions were considered. A field test was carried out on a 1:22 scale bridge. The measured average maximum relative opening displacements, after back-scaling to the prototype bridge, were compared with the estimates of minimum overlaps based on the current NZTA *Bridge manual*.

Twenty excitations were simulated for each New Zealand soil condition, ie soft soil, shallow soil and strong rock based on New Zealand design spectra (NZS 1170.5 2004). Spatial variation of ground motions was simulated using an empirical coherence loss function, taking into account the consequence of a finite seismic wave velocity, coherency loss and site-response effect. Different fundamental frequencies were achieved by altering the mass attached to the model. The effect of bridge structure-soil interaction (SSI) was considered by conducting a field test with the large-scale model.

More than 400 tests were conducted for each study. For the study of SSI effects on the 1:22 scale model, a total of 4200 tests were performed. For the three-segment bridge study 420 tests were conducted, and for the tests involving one bridge and two abutments, a total of 3500 tests were performed.

For the 1:125 scale model testing, relative displacement and pounding forces between two segments or between one segment and its adjacent abutments, bending moments at pier support and the deck displacement were measured. For the 1:22 scale model testing, the deck displacement and the bending moment at the pier supports were recorded. The data was compared to reveal the influence of each factor. To ensure the generality of the results, the recorded data was normalised against its reference values obtained by averaging the maximum responses of the same ground motion case, ie the same soil condition and coherency loss. The results were then used to derive recommendations for the NZTA *Bridge manual*.

Preliminary investigations involving a two-segment bridge were conducted to identify possible influencing factors for a relative response by the bridge. The ground motions used in these tests were based on the Japan design spectra and the effect of SSI was also investigated by using sand and clay as supporting soil. A total of 2430 pounding tests covering various soil conditions, coherency loss and foundation types were performed.

Conclusions

A series of experiments showed that spatial variation of excitation, dynamic characteristics of the structures and SSI effect had a significant influence on the relative response, pounding damage and unseating potential. The combination of these factors could result in a more severe bridge response. As the current NZTA *Bridge manual* specifications may not be sufficient for predicting minimum seat length and clearance between bridge structures, it may need revision.

Recommendations

Spatial variability of excitations should be addressed by the NZTA *Bridge manual* in the design of overlaps and clearance for coping with large relative responses between adjacent bridge structures. Two equations have been proposed based on the experimental results for calculating the necessary seating length and required gap accounting for spatially varying ground motions for abutment to girder joints and girder to girder joints. Abutment excitation due to ground excitation should be considered in the development of the abutment to girder relative response. For bridges of high importance, a rigorous modelling of the spatially varying ground motion is recommended for designing necessary seating length and joint gap. The modelling steps are described in section 6.5 of this report. Expansion joints with a large movement range, such as modular expansion joints, can be used to cope with large relative displacement.

Abstract

Bridge damage, especially due to pounding and unseating at expansion joints has been observed in almost all major earthquakes. It is the result of large relative displacements of girders, in excess of the designed gap width and seating length. Research shows that relative displacements of neighbouring bridge segments depend on the fundamental frequencies of the adjacent structures, spatially varying ground motions and soil-structure interaction (SSI). To evaluate the significance of the influence of these factors, three identical bridge models with a scale ratio of 1:125 were tested using shake tables. Another study involved one of these models pounding with movable abutments. Lastly, another scaled model of 1:22 was field tested to study the SSI effect in comparison with the fixed-base results. The scaled models were designed in accordance with the principles of similitude. The results obtained by isolating and varying each individual influence factor are presented and discussed in this report. The results show that the spatially varying effect of ground motions increases the maximum relative displacements and pounding forces between adjacent bridge girders even if adjacent segments have the same fundamental frequency. Recommendations are made for new bridge design which will take into consideration the spatially varying effect of ground excitations.

1 Introduction

Extensive damage to bridge expansion joints following major earthquake events is a common occurrence. Such damage attributed to pounding was observed in the 1994 Northridge earthquake (Hall 1994), the 1995 Kobe earthquake (Park et al 1995), the 1999 Kocaeli earthquake (Youd et al 2000), the 1999 Chi-Chi earthquake (Uzarski and Arnold 2001), the 2008 Wenchuan earthquake (Lin et al 2008), the 2010 Chile earthquake (Arias and Buckle 2010) and the 2011 Christchurch earthquake (Chouw et al 2011; Chouw and Hao 2012). During an earthquake, bridges may suffer from damage at their expansion joints due to differential movement of the adjacent girders. Gaps and sliding bearings are needed between bridge segments to accommodate expansion and contraction due to temperature change. Although an expansion joint can allow some movement in strong earthquakes, it has often proved to be insufficient. With a conventional expansion joint, the available gap is only a few centimetres (normally less than 4cm) to ensure a smooth traffic flow. When the closing relative movement exceeds the gap size, pounding will happen. Bridge decks are also at risk of collapsing due to unseating of girder ends. Unseating occurs when adjacent spans move away from one another in excess of the supporting length. The pounding damage and the probability of unseating, primarily depend on the relative displacement of adjacent spans. The relative response of structures to earthquake excitations is attributable to several factors: the dynamic properties of the structures; spatially varying ground excitation; and the supporting soil conditions and interaction between soil and structures. However, in previous investigations, the variation in ground motions at different bridge supports and soil-structure interaction (SSI) effect was often ignored. The New Zealand bridge design document, the *Bridge manual* (NZTA 2005), does not instruct bridge designers to consider spatially varying ground motion and SSI effect. The outcomes of this research show the importance of incorporating spatially varying ground motion effects into bridge design.

1.1 Spatial variation of excitation

The inconsistency of excitations results from a variety of reasons, such as delayed arrival of seismic waves at the adjacent bridge supports as a result of finite wave speed, known as the wave passage effect; coherency loss due to the reflection and refraction of waves through a heterogeneous soil medium, and superposition of waves from multiple sources; and varying site responses due to differences in local soil conditions (Kiureghian and Neuenhofer 1992).

Spatially varying ground motions have been identified as a significant factor in the pounding response of adjacent bridge girders (Chouw and Hao 2005). However, in numerous studies of bridge pounding uniform ground excitation has been assumed. Damage due to bridge pounding observed in major earthquakes clearly indicates the importance of spatially varying excitations in bridge response and has attracted the attention of many researchers. Among these researchers, Kiureghian and Neuenhofer (1992) believe that in most cases, the spatial variation tends to reduce the response compared with uniform support motions and this decrease could often be by a significant amount close to 30%. In contrast, most other studies have found that spatially varying ground excitations have a significant influence on structure behaviour and hence should be considered in the design. Dumanogluid and Soyluk (2003) showed that local soil conditions could significantly influence the characteristics of seismic ground motions such as amplitude, frequency content and duration of seismic waves. The wave passage effect has a strong effect on flexible structures while coherency loss effect is more pronounced to stiff structures (Zerva 1990).

In some past studies of the spatial variation effect of ground motions, only the effect of phase shift was considered. For instance, Jankowski et al (1998) investigated for simplicity the pounding response of an isolated elevated bridge under spatially varying earthquake loading considering only the wave passage effect. In fact, the incorporation of coherency loss effect in the studies of spatially varying ground motion became popular after the establishment of the SMART-1 array (Strong motion array Taiwan) (Bolt 1982). Hao et al (1989) proposed an empirical coherency loss function. Hao (1998) also concluded that the spatial variation of ground motions was the main reason for relative displacement development between bridge segments with similar dynamic properties. Zanardo et al (2002) developed a 3D finite element model and carried out a 3D nonlinear time-history analysis to determine the effect of spatially varying ground motions on the pounding forces. Chouw and Hao (2008c) also did a very thorough analysis to determine the effect of non-coherent ground motions on adjacent bridge frames. By investigating a single segment frame and a reduced model of a seven-span bridge using a linear response history analysis method, Yang et al (2002) revealed that the incoherency and the local site effect could result in a greater structural response than the wave passage effect. Zhang et al (2009) studied a multi-supported bridge subjected to spatially varying ground motions considering wave passage, incoherence and site-response effects, and concluded that the bridge response depended heavily on the wave passage, incoherency and local site effects.

1.2 Soil-structure interaction

The seismic loading experienced by a structure depends on the response of the structure in relation to the soil. Soil-structure interaction (SSI) is described as the interplay of the characteristics of the structure, the supporting soil and the ground motions. SSI occurs in the presence of deformable soil supporting a structure and affecting its seismic response. When a structure responds to a dynamic loading the interaction of the footing with the ground generates waves. These waves propagate and dissipate part of the energy. The structure therefore loses part of its vibration energy. This energy loss is named radiation damping and cannot occur in a crude numerical model when the structure is fixed at the base.

Conventional design codes considered SSI to be beneficial for seismic response due to wave spreading in the ground, leading to an increase of damping. For simplicity, SSI was often ignored in design code specifications as it was believed that, in comparison with the fixed base condition, SSI would only reduce the response of structures. The design was therefore considered more conservative and as such had a margin of safety. Mylonakis and Gazetas (2000) reviewed a set of past earthquake time-histories and concluded that an increase in the fundamental period due to SSI might increase the displacement, leading to a detrimental effect on the seismic demand imposed on the structure. Hence, it might be inappropriate to neglect the SSI effect.

A study on the performance of multi-span simply supported bridges (MSSS) using an analytical model was carried out by Saadeghvaziri et al (2000). They concluded that SSI had a detrimental effect on the seismic response (in the longitudinal direction) of the bridges. Their report recommended that dynamic analysis of MSSS bridges should explicitly account for SSI effects. Other studies (Chouw and Hao 2008c; Li et al 2010) also consistently concluded that neglecting SSI in design could cause the relative movement of bridge spans to exceed the designed limits, causing pounding and unseating.

A study on the effect of SSI on pounding between adjacent bridge girders (Chouw and Hao 2005) involved using finite element modelling for the bridge and foundation structure, and the boundary element method for the subsoil. The numerical analysis of the models similarly concluded that ignoring SSI led to inaccurate calculations of pounding forces, and the excitations simulating the soft soil condition led to more SSI than in medium and hard soil conditions.

1.3 Pounding and unseating

Damage to bearings and expansion joints due to the pounding of adjacent segments is an issue of economic significance (Lin et al 2008; Moehle and Eberhard 2000). A considerable amount of research has been conducted on bridge pounding. Some researchers have found that pounding can decrease bending moment demand placed on piers. Malhotra (1998) adopted two uncoupled rods to represent bridge segments with a fundamental frequency of 0.67Hz for the shorter piece and 1Hz for the longer one. Their behaviour was modelled by two single degree of freedom models. It was concluded that seismic pounding generally decreased column deformation due to the loss of energy by impact. They also found that pounding did not increase the longitudinal separation at the hinges. However, Malhotra did not consider spatially varying ground excitations and SSI, and pounding with abutments was also excluded. In contrast, a study conducted by Raheem (2006) on mathematical modelling of pounding between adjacent structures showed that pounding depended on the characteristics of ground motions and dynamic properties of adjacent structures, and it could amplify the global response of bridges. This research also found that pounding could induce larger acceleration of girder segments and shear force at columns than when it was not present. Chouw and Hao (2008c) confirmed that the impediment to girder movement due to pounding did reduce pier bending moments. By considering the expansion joint gaps ranging from 1 to 10cm, Chouw and Hao (2008b) showed that pounding could increase bending moments in piers as the gap size increased.

Researchers have also been greatly interested in pounding forces at the expansion joints. Jankowski et al (1998) studied pounding in isolated elevated bridges and concluded that for their model bridge activated girder force was smallest for very small gap sizes and increased as the gap became larger until the gap size was so large that pounding did not take place.

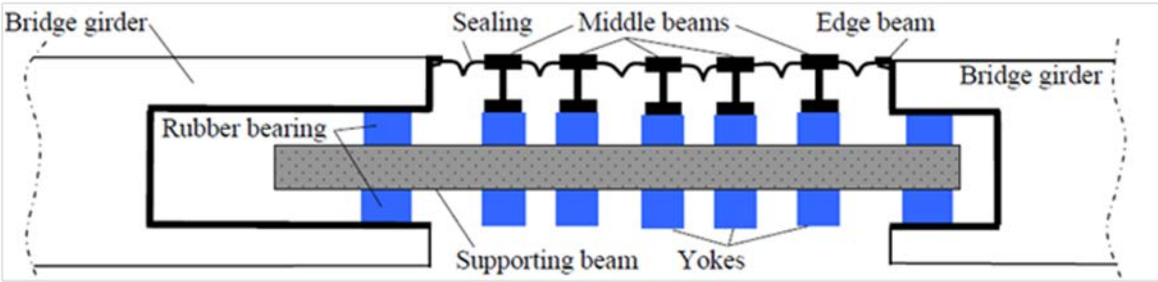
Pounding between an abutment and its adjacent girder has also been investigated, though mainly numerically. Maragakis and Jennings (1987) studied a short skew bridge and concluded that the impact between the deck and the abutment contributed to the planar rigid body rotation of the deck. Kim et al (2000) modelled a multi-span bridge with abutments and found that the largest relative displacements occurred between the abutment and the nearby girder, but not between the girders. A case study on pounding between sliding decks and abutments was performed by Wang and Shih (2007) who concluded that frictional sliding on bearing pads together with plastic deformation in backfill soils were far more beneficial in preventing deck fall-off disaster than pier strength in large earthquakes.

Bridge girder unseating occurs when the opening differential movement of adjacent girders is greater than the seat length. It is a major cause of bridge collapse during earthquakes. Reconnaissance on the 2008 Sichuan earthquake in China showed that unseating due to insufficient seating length was the most common cause of bridge damage (Lin et al 2008). In the 2010 Chile earthquake, girder unseating was also frequently observed among the 7250 bridges investigated (Arias and Buckle 2010). A study by Moehle et al (1995) concluded that the unseating could be caused by pounding.

1.4 Modular expansion joint

Over the past few decades a number of new expansion joints have been designed which allow for large relative displacements between bridge spans. These joints are termed modular expansion joints (MEJs) (Rizza 1973). An MEJ consists of one or more supporting beams spanning between adjacent bridge decks. These beams support a number of centre beams with small gaps between them. An example of an MEJ is shown in figure 1.1. The sum of these multiple small gaps provides a large gap for relative movement between the spans (Connor and Dexter 1999). The gaps between the centre beams are filled with a flexible rubber membrane to ensure water tightness (Chouw and Hao 2008a). Traditionally MEJs were designed and used in bridges expected to undergo large thermal contractions and expansions. However an MEJ system also has potential to be applied in seismic design to completely prevent deck joint pounding. It can also avoid downtime in a seismic event. Tests on the effect of earthquake loading on an MEJ system have been conducted by Spuler et al (2011) and Clyde and Whittaker (2000), who found that the modular expansion joint could survive in most earthquakes. However, in these tests, only the device was considered. The authors are not aware of experiments on MEJs with adjacent structures under earthquake loading.

Figure 1.1 Modular expansion joint (reproduced from Chouw and Hao 2008a)



The investigation of Hao and Chouw (2007) identified significant factors that influence the behaviour of MEJs in seismic events. These are spatially varying ground motions, coherency loss, apparent wave velocities, ratio of fundamental frequencies of adjacent structures and SSI. Chouw and Hao (2008b) studied the influence of SSI on the required separation distance of two adjacent bridge frames connected by an MEJ. It was concluded that using MEJs with a large gap was likely to completely preclude bridge girder pounding, and hence to prevent local damage at the girder ends. However, a large girder movement results in large bending moment in bridge piers, reducing the advantages of using MEJs in bridges to resist earthquake loading. Bi et al (2010) investigated the minimum total gap that an MEJ must have to avoid pounding at the abutments and between bridge decks considering the effect of spatially varying ground excitation. This study found that the required MEJ total gap depended on the dynamic properties of the participating adjacent structures and the spatially varying ground excitations.

1.5 Experimental studies on pounding

A few experiments have been conducted to investigate the effect of structural pounding for buildings and bridges. Van Mier et al (1991) conducted a series of experiments on the impact between a pre-stressed concrete pile and a concrete striker of variable mass, and proposed a simple elastoplastic contact model to predict the pressure response of the contact zone. Papadrakakis and Mouzakis (1995) tested two-storey reinforced concrete buildings with zero gap under sinusoidal excitation using a shake table and verified the proposed analytical model using the experimental results. Chau et al (2003) conducted shake table experiments on pounding between adjacent two-storey steel towers considering harmonic excitation and the 1940 El Centro earthquake ground motions. The results were compared with those obtained from an analytical model based on the Hertz contact law. Jankowski (2010) conducted an experimental study on pounding between two buildings made of different materials, and concluded that the coefficient of restitution depended heavily on both the pre-impact velocity and the material used.

Very limited experimental research was found on bridge pounding. Crewe and Norman (2006) physically investigated a scaled bridge model considering spatially varying ground motions and compared the results with a numerical model. Guo et al (2009) conducted a shake table test on a 1:20 scaled two-segment base-isolated bridge model to investigate the pounding behaviour of adjacent superstructures. The model used in their study was scaled only in the geometric sense – a feature common to most scaled models in this field where consideration of scaling more than just the geometry is rare.

Crewe and Norman (2006) carried out an experimental study on bridge pounding considering the wave passage effect of ground motions and concluded that assuming uniform ground motions would lead to unsafe bridge design. Johnson et al (2008) shake-table tested a quarter-scale, two-segment reinforced concrete bridge and found that motion incoherency had little effect on symmetric bridge response. Though several experimental investigations on pounding between bridge spans were performed, to the authors' best knowledge there was very limited experimental research conducted on abutment pounding. Zhu et al (2002) performed a physical test on pounding between a girder and an abutment to verify their proposed three-dimensional non-linear model. Al-Homoud and Whitman (1999) conducted a study involving a centrifuge test of a span pounding with a retaining wall, in order to verify their analytical model developed for an analysis of the seismic response of rigid highway bridge abutments founded on dry sand. However, in both experiments a rigid abutment was used, and the effect of movable abutments on the pounding response remained untested.

1.6 Bridge design regulations

Many bridge design manuals suggest that adjacent segments should have identical or at least similar fundamental frequencies to encourage in-phase movement. However, the out-of-phase movement is not only dependent on the dynamic properties of the bridge segments, but also on the characteristics of ground motions and SSI (Chouw and Hao 2008a).

The Californian Department of Transportation's seismic design criteria (CALTRANS 2010) suggests that, without considering variations in soil along the bridge, the fundamental frequency of the more flexible segment must be at least 0.7 times that of the stiffer segment to reduce pounding potential. However, the recommendation is only valid under the assumption of uniform ground motions and fixed-base structures. Chouw and Hao (2008a) studied a bridge consisting of two segments of different deck lengths and concluded that the same fundamental frequency of adjacent segments alone was not sufficient to ensure in-phase movement. A consideration of the frequency ratio of the adjacent segments is not enough to determine the structural response; their own frequencies are also important.

In long span bridges, where spatially varying ground excitations are significant, current regulations tend to underestimate the structural response of adjacent girders. Part 2 of Eurocode 8 (BS EN 1998-2 2005) recommends that for bridges longer than 600m or bridges built on significantly varying soil conditions, the effects of spatially varying ground excitations must be considered in the design. However, Crewe and Norman (2006) proved that even shorter bridges can suffer from spatially varying ground excitations.

Japan Road Association's (JRA) (2004) *Specification for highway bridges* is one of the few regulatory documents to provide guidance on how to incorporate the spatial variation of ground motions, although only empirically. Regrettably, SSI tends to be omitted in bridge design codes, although research confirms that SSI can play an important role in the response of bridges subjected to an earthquake (Saadeghvaziri et al 2000). Hao and Chouw (2008) showed an example of a bridge crossing a valley with piers of different lengths for adjacent spans. The study revealed that even if adjacent bridge structures have the same fixed-base fundamental frequency, differential movement cannot be prevented if the adjacent structures have different interaction with the supporting ground, eg due to their different slenderness (see Chouw 2008). A fixed-base fundamental frequency is defined as the fundamental frequency of the bridge structures with an assumed fixed base. Chouw and Hao (2006) also revealed the shortcoming of JRA regulation using numerical procedures and a two-segment bridge model. It was found that the JRA specification is likely to underestimate the required seat length when considering the simultaneous effects of SSI and spatially varying ground excitations at different supports.

Despite the vulnerability of New Zealand's bridges to seating loss, especially those in zones of high seismic risk, New Zealand designers are neither required to consider spatially varying ground motions nor given a clear specification on how to deal with SSI. The NZTA (2005) *Bridge manual* is currently under revision to incorporate the amendments in the NZS 1170; part 5 (2004). In addition to the part from the NZS 1170; part 5, the manual should also suggest the minimum seating length to incorporate the effects of spatially varying ground motions and SSI.

1.7 Similitude modelling

Scale models provide a useful alternative for evaluating the proposed design of large structures when testing them is not feasible. To correctly reflect the dynamic behaviour of the prototype structure, the scale model should be constructed based on similitude requirements. These are the correlations between the model and prototype structure geometry, material properties, initial and boundary conditions and applied loading. The response of the scale model can then be expressed as a function of the response of the prototype structure (Dove and Bennett 1986). Similitude requirements are derived based on Buckingham's π theorem (Buckingham 1914), which states that every dimensionally homogeneous equation involving n physical quantities can be simplified to a functional relationship containing n-u independent dimensionless products, where u is the number of independent fundamental units. The system is characterised by six physical quantities, ie length, time, acceleration, stiffness, mass and force, among which, length, time and stiffness are selected to be the independent fundamental units and require pre-definition. The rest of the quantities are to be calculated through the dimensionless functions. A detailed description of the principle and the derivation of the similitude relations can be found in the report by Dove and Bennett (1986).

However, in many situations, satisfaction of all similitude requirements is impossible, and some of the similitude requirements have to be relaxed to allow feasible experiments. An approach proposed by Moncarz and Krawinkler (1981) suggested that in a structure with an assumed uniformly distributed mass (appropriate to the prototype bridge), the seismically effective mass can be decoupled from the structurally effective mass.

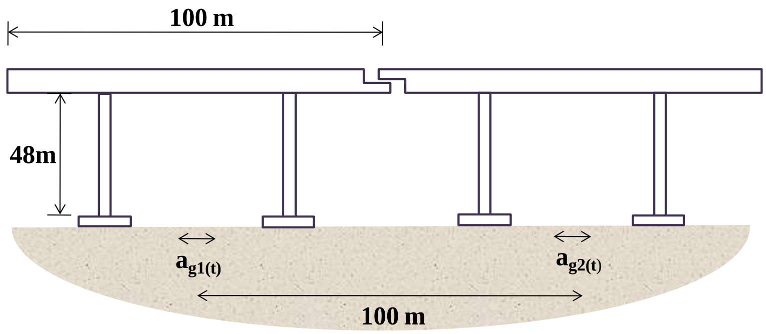
2 Pounding of a two-segment bridge

Past research shows that relative displacement of neighbouring bridge segments depends on the fundamental periods of the adjacent structures, spatially varying ground motions and soil-structure interaction (SSI). To evaluate the significance of these influence factors, two identical, scaled models were tested using shake tables. The scaled models were designed in accordance with the principles of similitude. The results obtained by isolating and varying individual influencing factors are presented and discussed in this section.

2.1 Prototype structure and model

In this study, two identical similitude models were utilised to investigate pounding characteristics of adjacent bridge structures, considering spatially varying ground motions and SSI. The prototype bridge structures, which are constructed with reinforced concrete, consist of two identical segments connected by an expansion joint in the middle of the bridge. Each segment comprises a deck with a length of 100m, and two piers, which are 48m high. The distance between the centres of the two segments is 100m. The whole bridge rests on surface foundations. Figure 2.1 shows the prototype structures.

Figure 2.1 Prototype structures on surface foundation

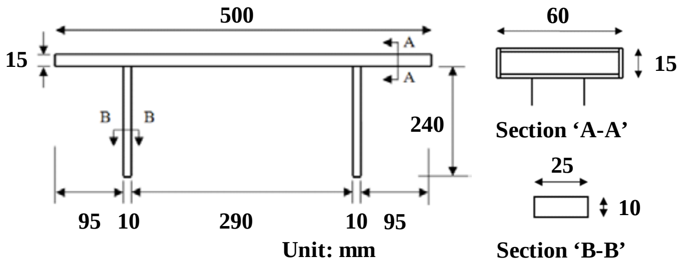


The scaled models were built using PVC primarily for its fairly small modulus of elasticity but high strength. The similitude laws derived for the prototype and model structures used in this research are listed in table 2.1, where *S* represents the scale factor of any considered quantity, and is equal to the dimension of the prototype over the dimension of the model. The scale factor for modulus of elasticity was determined once the model material was decided. The dimensions of the models were established according to the similitude laws, and are shown in figure 2.2.

Table 2.1 The scale factors for the considered quantities

Physical quantity	Similitude law	Scale factor
Length (*L*)	S_L	200
Modulus of elasticity (*E*)	S_E	10.72
Acceleration ($\ddot{y}$)	$S_{\ddot{y}}$	1
Time (*t*)	$S_t = \sqrt{S_L / S_{\ddot{y}}}$	14.14
Mass (*M*)	$S_M = S_L^2 S_E / S_{\ddot{y}}$	428800
Force (*F*)	$S_F = S_M S_{\ddot{y}}$	428800

Figure 2.2 Schematic drawing of the model

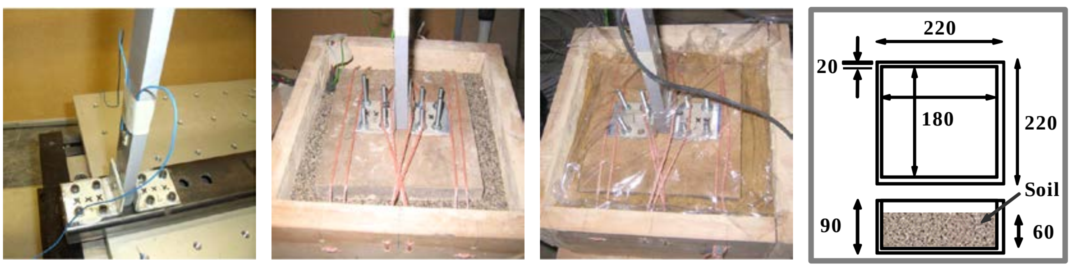


To incorporate the SSI effect, the models were tested with different foundations. Mixed river sand and clay was used to simulate the supporting soil, and the results were compared with the fixed base foundation. The soil was contained in square wooden boxes with an internal dimension of 180mm. For each test, the soil was filled to a depth of 60mm. Figure 2.3 depicts all the foundations and the schematic drawing of the sand box. Owing to the intrinsic difficulties of soil modelling, the sand and clay used in this test remained unscaled.

Table 2.2 Fundamental periods of the models with their corresponding added mass

Added mass	Fundamental period (s)		
	Fixed base	Sand	Clay
1kg	0.078	0.090	0.091
1.5kg	0.089	0.100	0.098
2kg	0.097	0.110	0.105
2.5kg	0.105	0.115	0.113
3kg	0.116	0.122	0.126
Ref. model	0.115	0.122	0.123
3.5kg	0.119	0.127	0.136
4kg	0.128	0.136	0.139
4.5kg	0.132	0.145	0.144
5kg	0.141	0.151	0.154

Figure 2.3 Considered support conditions: (a) fixed base foundation, (b) footing on sand, (c) footing on clay and (d) dimensions of the sand box (in mm)



(a) (b) (c) (d)

Two longitudinally aligned models were positioned with an almost-touching gap. To investigate the significance of different dynamic characteristics of the structures, a range of fundamental period ratios was achieved by fixing a 3kg mass on one model while varying the amount of added mass of the other with increments of 0.5kg from 1kg to 5kg. The girder with a constant mass was defined as the reference model. The fundamental periods of the reference model and the variable mass model for each case are shown in table 2.2. Eighteen sets of simulated earthquakes categorised as soft, medium and hard soil condition excitations, were sequentially applied with alternations between uniform and spatially varying ground motions. The test procedures were repeated for three foundation types.

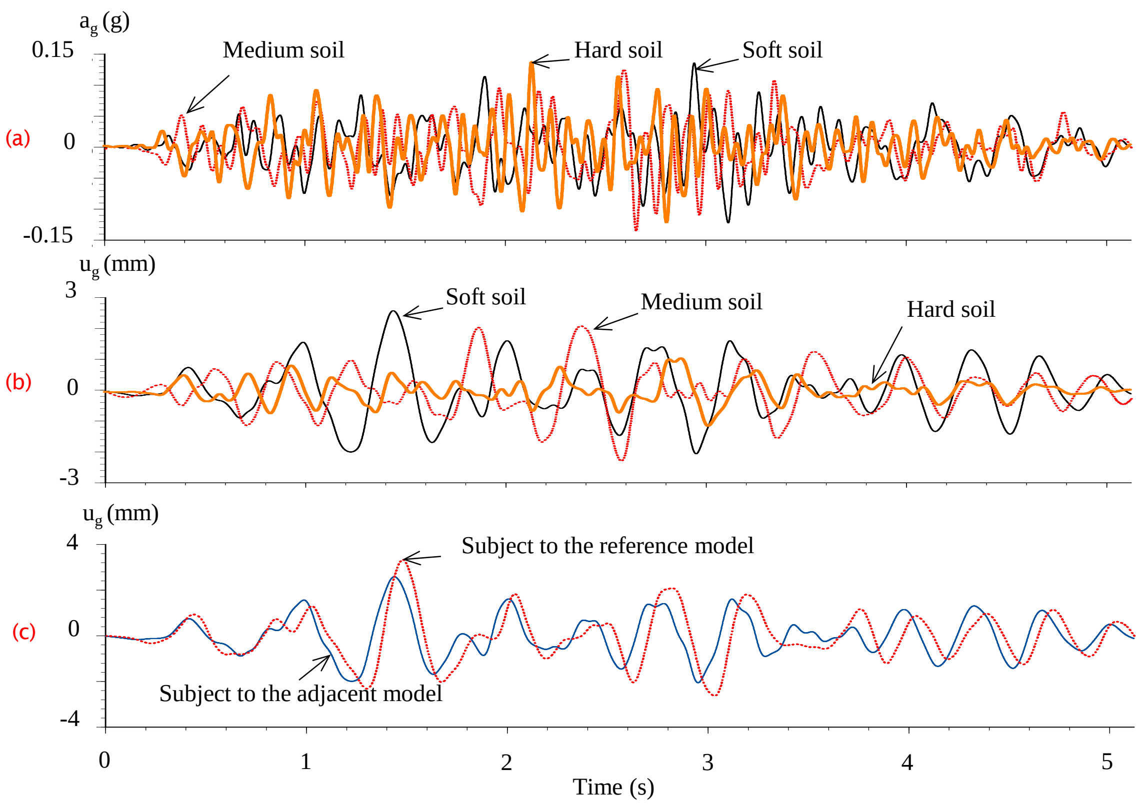
The quantities of interest including the relative displacements, the pounding forces and the bending moments of the piers were measured by the gauges described in section 3.1.3. Experimental control, data acquisition and analysis of results were implemented with MATLAB programs.

2.2 Results and discussion

2.2.1 Simulated earthquakes

The applied simulated earthquakes were scaled from the ones formulated from the Japan design spectra (JRA 2004), which were established using 63 strong earthquakes (magnitude 6.5 or larger) recorded in Japan with a focal depth less than 60km. Three sets of spatially correlated ground motions were selected for each soft, medium and hard soil conditions, respectively. These simulated earthquakes were based on the assumptions that the ground motions are highly correlated, and the wave passes two supports which are 100m apart with an apparent velocity of 500m/s. Figure 2.4 shows the spatially varying ground motions applied to different piers of the bridge models. The simulation of the ground motions is described in section 3.1.4.

Figure 2.4 Ground motions: (a) ground accelerations for different soil conditions, (b) ground displacements for different soil conditions and (c) spatially varying ground displacements for soft soil condition



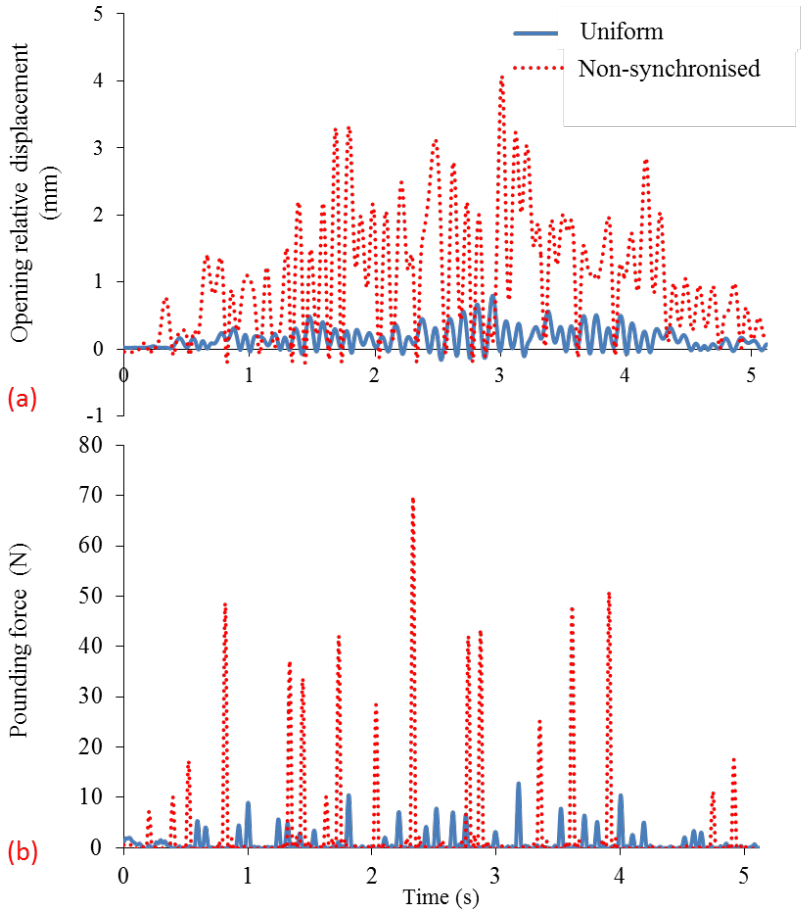
The similitude law for time requires scaling the time frame of the original ground motions by 14.14. However, it is unrealistic for the shake tables to produce all the motions within 1.5s, which would have been the duration of the correctly scaled earthquake. A compromise was achieved by scaling the time frame by

4.0, which leads to a motion duration of 5.12s. As a result, the models were generally subjected to waves with longer wavelengths than true similitude demanded and lower frequency components.

2.2.2 Influence of spatially varying ground motions

Figures 2.5(a) and (b) depict respectively the relative displacements and pounding forces between two models with almost equal fundamental periods when subjected to uniform and spatially varying ground motions with a fixed base foundation under hard soil excitations. Results from figure 2.5 show that under spatially varying excitations, the maximum opening relative displacement is approximately 4mm, and the greatest pounding force reaches 70N while the maximum structural response due to uniform ground motions is only 0.5mm and 10N, respectively. The models still encounter some small response under uniform ground motions because the fundamental periods of the models are not identical. Much greater relative displacements and pounding forces are caused by spatially varying excitations than uniform excitations, although the two models have almost the same fundamental periods. Spatially varying excitations produce larger structural response not only for the case of equal fundamental period, but also for other considered cases where the fundamental periods of the models are different.

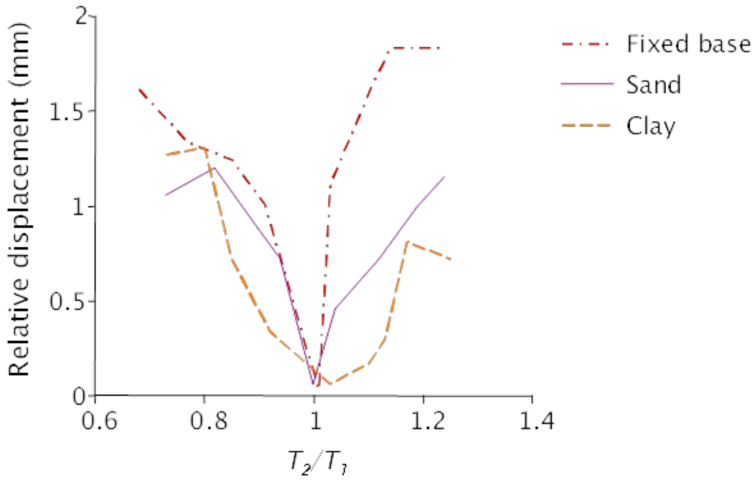
Figure 2.5 Consequence of spatially varying ground motions: (a) relative displacements and (b) pounding forces of the structures with almost equal fundamental frequency ($T_{ref.}$ = 0.115 s and $T_{adjacent}$ = 0.116s) and fixed base under hard soil excitations



2.2.3 Influence of SSI

The SSI is considered by including subsoil flexibility in the investigation. Figure 2.6 shows the opening relative displacements of the model bridge girders with a fixed base foundation, surface footings on sand and on clay considering no pounding. These results clearly show the tendency for the smallest opening relative displacement to occur at the period ratio $T_1 = T_2$ for all types of foundations. If only considering SSI, the recommendation of equal fundamental periods to reduce relative displacement is still reasonable. Fixed base foundation leads to the largest opening relative movement for each period ratio compared with foundations on sand and clay, where clay gives rise to the smallest response. The fundamental period increases from the fixed foundation to the foundation on clay.

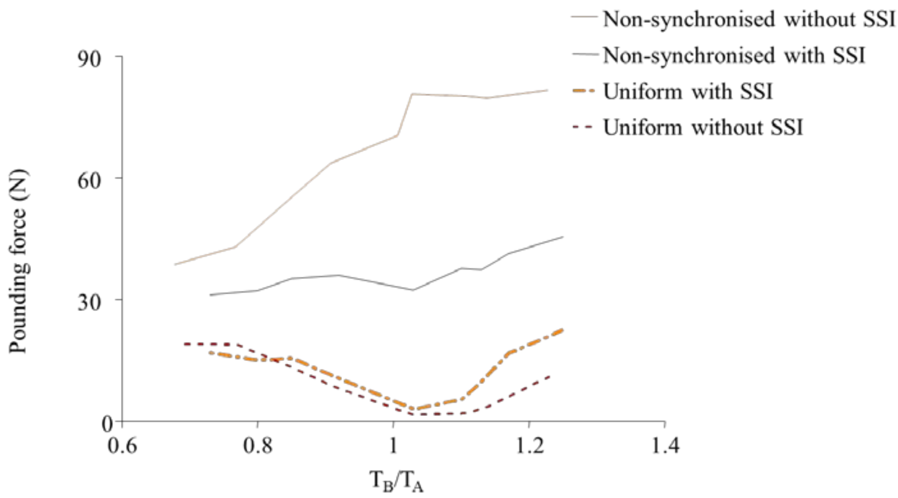
Figure 2.6 Consequence of SSI effect on the opening relative displacement between the girders under uniform soft soil excitation without pounding



2.2.4 Simultaneous influence of spatial variation of excitations and SSI

Figure 2.7 compares the pounding forces incurred with a fixed-base foundation and a foundation on clay due to uniform and spatially varying ground motions. The spatially varying excitations cause the most vigorous response for the models with fixed-base foundations while the models with foundations on clay result in the smallest pounding force under uniform excitations. In this study, SSI reduces pounding in the case of spatially varying excitations and enhances pounding when uniform ground motions are relevant. By inspecting the pounding forces with the effect of spatially varying excitation (the solid lines), it is clear that when the period ratio is high or the adjacent structure is flexible, the pounding force becomes larger. Figure 2.7 also implies that the recommendation of $T_1 = T_2$ will not avoid out-of-phase movement if spatially varying ground motions are involved. Furthermore, with the effect of spatial variation of excitation, identical segments do not necessarily result in minimum separation. The conclusion from this is that ignoring spatially varying excitation will underestimate the required seating length especially when the recommendation of equal fundamental frequency is followed.

Figure 2.7 Consequence of spatial variation of excitation and SSI effect on pounding force under soft soil condition excitation



2.3 Summary

This experimental study addressed the effect of fundamental periods, spatially varying ground motions and SSI on relative movement and pounding force between adjacent girders. A description is given of two scaled models, whose design was based on the similitude theorem. A series of tests focusing on different influence factors were conducted using shake tables. A total of 2430 pounding tests considering various soil conditions, coherency loss and foundation types were performed. The investigations revealed:

1. The results confirmed that the recommendation of current design regulations to encourage identical fundamental frequencies for adjacent structures to avoid pounding is valid only in the case of uniform ground motions.
2. Spatially varying excitations can violate the advantage of having equal fundamental periods and result in the most severe pounding damage and largest required seating length.
3. SSI results in smaller opening relative displacements in the case of uniform ground motions.
4. SSI can reduce the pounding force due to spatially varying ground motions but may have an adverse effect when uniform ground motions are assumed.
5. Under spatially varying ground motions, pounding force between girders can increase if the adjacent segment becomes more flexible.

3 Pounding of a three-segment bridge

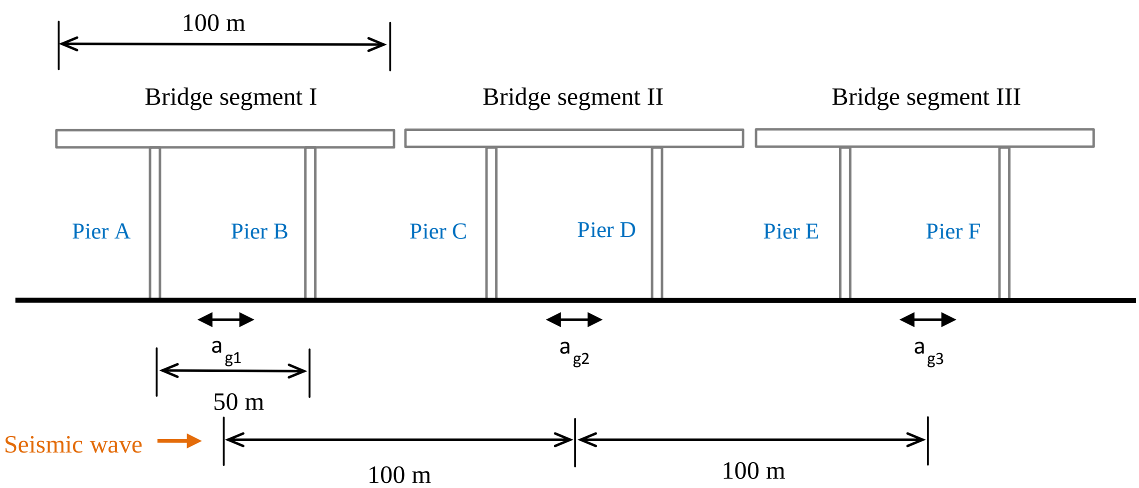
The previous section investigated the effect of pounding at one location. The objective of this section is to evaluate the influence of spatial variation of ground motions on the pounding behaviour at multiple locations. Three adjacent bridge segments are considered. The investigation is performed using three shake tables. The input spatially varying ground excitations are simulated based on the New Zealand design spectra for soft soil, shallow soil and strong rock conditions using an empirical coherency loss function. The development of the pounding forces recorded in this experiment is simulated both analytically and numerically by the Hertz-damped model and ANSYS programs, respectively.

3.1 Experimental model

3.1.1 Prototype structure and model

In order to investigate the response of a bridge segment subject to the effect of pounding from adjacent segments on one or both sides, a system consisting of three identical segments was considered (figure 3.1). It was assumed that each of the bridge segments experienced the ground excitation $a_{go}(t)$ acting at the middle of the bridge segment, where *o* is the number of the considered sites, ie *o* = 1, 2 and 3. Each of the bridge segments had a length of 100m. To limit the number of influencing factors, a consideration of abutments was deferred to a later study described in chapter 4.

Figure 3.1 Considered system



The prototype bridge structure was based on one segment of the Newmarket Viaduct replacement bridge currently under construction in Auckland, New Zealand. The modelled segment was a single cell box girder with a total length of 100m. The two bridge piers were 15.5m high and 50m apart. The construction drawings of the Newmarket Viaduct were used to construct the numerical model from which the dynamic characteristics of the prototype structure were determined. The details are summarised in table 3.1, where I_{deck} and I_{pier} are respectively the second moments of area of the bridge deck and pier about their respective principal axes. $K_{bending}$ is the bending stiffness of the bridge structure measured as the ratio of longitudinal force to longitudinal displacement of one segment at deck level. The compressive strength of the concrete is assumed to be 50MPa.

The experimental models were constructed based on the prototype bridge structure using scale modelling. For the models used in this study, the relevant similitude requirements are presented in table 3.2, where N_Q is the scale ratio (prototype over model) of the considered physical quantity Q.

Table 3.1 Dimensions and dynamic characteristics of the prototype structure

Quantity	Value	Quantity	Value
Segment length	100m	Pier mass	1.93×10^5kg
Pier height	15.5m	I_{deck}	9.337m^4
Pier width	3.44m	I_{pier}	0.387m^4
Pier thickness	1.48m	$K_{bending}$	7.189×10^7N/m
Segment mass	1.702×10^4kg/m	Fundamental frequency	0.98Hz
Effective mass (at height of bridge deck)	1.895×10^6kg	Modulus of elasticity (50MPa concrete)	30GPa

Table 3.2 Original and revised similitude requirements and scale ratios of the model structure

Physical quantity	Original similitudes	Revised similitudes	Scale factor (N)
Length (L)	N_L	N_L	125
Time (t)	N_t	N_t	2
Modulus of elasticity (E)	N_E	N_E	12
Mass (M)	$N_M = \frac{N_E N_L^2}{N_a}$	$N_M = N_E N_L^2$	187,500
Acceleration (a)	$N_a = \frac{N_L}{N_t^2}$	$N_a = \frac{N_L}{N_t^2}$	31.25
Force (F)	$N_F = N_M N_a$	$N_F = N_M N_a$	1,464,000

Table 3.3 Scaled values of the parameters

Parameter	Value	Parameter	Value
Segment length	800mm	Girder depth	20mm
Pier height	124mm	Girder width	120mm
Pier width	21mm	Deck thickness	3mm
Pier thickness	3mm	I_{deck}	3.82×10^4 mm^4
I_{pier}	45mm^4	$K_{bending}$	1.53×10^3 N/m
Seismic mass (total model)	10.11kg	Fundamental frequency	1.96Hz

However, it was found that the model mass required was beyond the capacity of the shake tables. Therefore, the seismically effective mass was decoupled from the structurally effective mass based on an approach proposed by Moncarz and Krawinkler (1981). Because of the small shake table size, the geometry scale ratio was determined to be 125. To ensure that the model excitations were strong enough, the time scale ratio was determined to be 2. PVC was selected for constructing the scale models mainly because of its low modulus of elasticity of 2.5GPa, which led to a high mass scale ratio and hence a low model mass. Revised similitude requirements for the various quantities and their corresponding scale ratios are summarised in table 3.2. The parameters of the model bridge are listed in table 3.3.

3.1.2 Pounding and measuring head

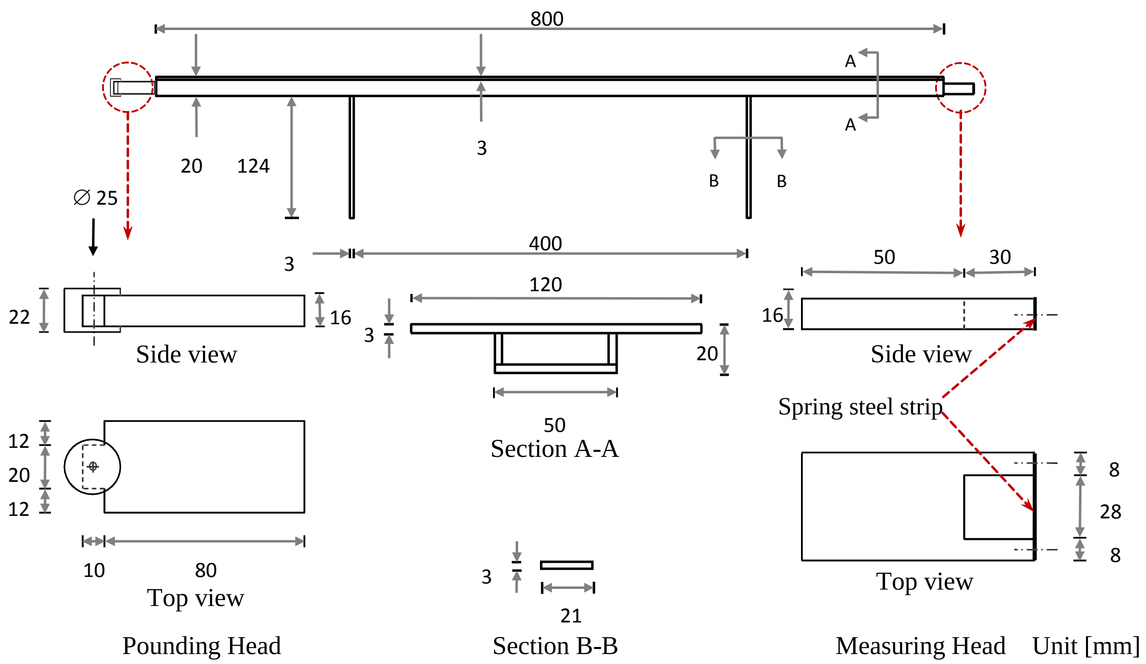
In order to measure the pounding force at the interface between each segment of the bridge model, a pounding head and a measuring head were fabricated using PVC. The measuring head is a force sensor made by a strain gauge bonded to the back of a highly elastic spring steel strip with a 10 × 1mm cross section. The pounding head consists of a 25mm diameter PVC cylindrical block for even contact with the pressure sensor. Each head was glued into the end of the adjacent bridge girder. The measuring head was calibrated by applying load increments of 1N to a total of 10N. Detailed dimensions of the heads, together with the model dimensions are shown in figure 3.2. This arrangement of pounding and measuring head prevented structural degradation during pounding contact, providing results consistent with an upper bound elastic collision regime.

3.1.3 Equipment and testing

Three APS Model 400 Electro-Seis shake tables, each with a frequency range of 0-200Hz, and a peak-to-peak stroke of 158mm were utilised to apply the earthquake motions to the models. The shake tables are unidirectional with platform dimensions of 350 × 350mm. Displacement transducers were installed between adjacent segments to measure the relative girder displacement. Three unidirectional accelerometers with a measuring capacity of ± 2g were fixed to each girder end behind the pounding head. Strain gauges were glued to one side of each pier at the base.

For this experiment, only fixed base supports were considered. To focus on the influence of multiple pounding SSI was not considered. To verify that all models had the same fundamental frequency, snap back tests were conducted on each model and the fundamental frequencies were determined from the ensuing free vibration. The measured fundamental frequencies for each model were 1.965Hz, 1.966Hz and 1.965Hz with corresponding damping ratios of 1.9%, 2% and 2.1%, respectively.

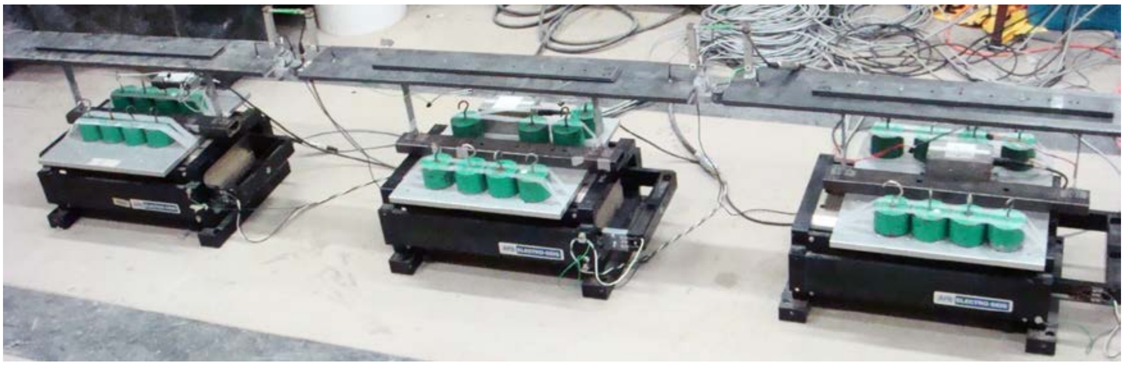
Figure 3.2 Schematic drawing of the model with details of the pounding and measuring heads



In order to investigate the site response effect of seismic waves, ground motions representing near-fault earthquakes in different soil types were applied to the models. The considered soil types, based on the New Zealand design spectra (NZS 1170.5 2004), were categorised as Class A (strong rock), Class C (shallow soil site) and Class D (soft soil site). The bridge models were subjected to uniform ground motion, spatially varying ground motion and uniform ground motion with time delay. To incorporate the effect of coherency loss, spatially varying ground motions with the soft soil condition were further categorised as highly, intermediately or weakly correlated, while ground motions of shallow soil and strong rock conditions were considered with only high correlation. For the pounding test, the initial gaps between any two segments were approximately zero, whereas during the non-pounding tests the segments were separated sufficiently to just prevent subsequent contact. The shake tables were found to be load sensitive. In other words, the total mass of the test specimen had a noticeable impact on the shake table performance. Therefore, for comparability with tests considering SSI in the future, in which soil boxes will be placed on the tables, an additional dummy mass of 8kg was added to each shake table, to be replaced by soil boxes of the same mass. The shake tables were operated under acceleration time histories and the data was recorded and analysed under control of MATLAB data logging and control programs. The final setup of the shake tables, bridge models and dummy mass (the cylinders) is shown in figure 3.3.

In order to compare the structural responses of the inner segment due to pounding from both sides with those due to pounding from only one side, bridge segment III was removed from the setup in figure 3.3. The tests were then repeated with the same ground motions.

Figure 3.3 Setup of the test system



Segment I　　　Segment II　　　Segment III

3.1.4 Spatially varying ground motions

It is common in engineering practice to simulate spatially varying ground motions which are compatible with specific design response spectra. Many stochastic ground motion simulation methods have been proposed. For example, Hao et al (1989) and Deodatis (1996) simulated spatially varying ground motions in two steps: first the spatially varying ground motion time histories were generated using an arbitrary power spectral density function, and then adjusted iteratively to match the target response spectrum. Usually a few iterations were needed to achieve a reasonably good match. Recently, Bi and Hao (2011) further developed this method by simulating spatially varying ground motions which were compatible with the ground motion power spectral densities that were related to the target design response spectra, instead of arbitrary power spectral density functions. Compared with the methods suggested by Hao et al (1989) and Deodatis (1996), less or even no iteration was needed in the latter approach (Bi and Hao 2011). It is clearly computationally more efficient and has been used in the present study to simulate spatially varying ground motion time histories that are compatible with the design spectra specified in the New Zealand standard for earthquake loading (NZS 1170.5, 2004).

The spatial variation properties between ground motions recorded at two locations *i* and *j* on the ground surface was modelled by an empirical coherency loss function (Hao et al 1989)

$$\gamma_{ij} = \exp(-\beta d_{ij})\exp(-\alpha d_{ij}^{1/2} f^2)\exp(-i2\pi f \frac{d_{ij}}{c_a}) \quad \text{(Equation 3.1)}$$

where β is a constant, d_{ij} is the distance between the two locations *i* and *j* in the wave propagation direction, *f* is the frequency in Hz, c_a is the apparent wave velocity, and α is a function of the following form:

$$\begin{aligned} \alpha(f) &= \frac{a}{f} + bf + c \quad & f \le 10\ Hz \\ \alpha(f) &= \alpha(10) & f > 10\ Hz \end{aligned} \quad \text{(Equation 3.2)}$$

Simulation of the spatially varying ground motions was carried out with the empirical coherency loss function derived from the recorded time histories at the SMART-1 array during event 45 (Hao et al 1989). The corresponding parameters are given in the second row of table 3.4, which corresponds to highly correlated ground motions. For comparison, two modified coherency loss functions are also used in the study, representing intermediately and weakly correlated ground motions, respectively. Details of the numerical approach for simulating the ground motions are given in Bi and Hao (2011).

Table 3.4 Parameters for coherency loss functions

Coherency loss	β	a	b	c
Highly (SMART1-event45)	1.109×10^{-4}	3.583×10^{-3}	-1.811×10^{-6}	1.177×10^{-4}
Intermediately	3.697×10^{-4}	1.194×10^{-2}	-1.811×10^{-5}	1.177×10^{-4}
Weakly	1.109×10^{-3}	3.583×10^{-3}	-1.811×10^{-5}	1.177×10^{-4}

The apparent wave velocity c_a measures the time delay between ground motions at two supports *i* and *j* separated by distance d_{ij}. Previous studies have found that in general c_a is a function of frequency (Hao et al. 1989). Because the relation between the apparent wave velocity and frequency is rather random, almost all the previous studies of ground motion spatial variation effects on structural response have assumed a constant apparent wave velocity c_a. In this study, c_a is assumed to be 500m/s. The simulated spatially varying ground motions are for bridges in Wellington.

Structural responses were studied under five cases, representing different local site conditions and different ground motion spatial variations. In each case, 20 sets of spatially varying ground motion time histories were simulated in order to obtain relatively unbiased structural responses. In the simulations, the sampling and upper cut-off frequencies were set to 100 and 25Hz, respectively. Duration of 20.48 s was selected in order to have a convenient total number of points (2048) for a Fast Fourier Transform (FFT).

Figure 3.4 shows one set of the simulated ground motion time histories based on the response spectra specified in the New Zealand earthquake loading standard. The peak ground acceleration was normalised to 0.72g. The coherency was highly correlated. The separation distance between different supports was 100m. Figure 3.5 compares the response spectra of the simulated ground motions and the target design spectra. Comparison between the coherency loss of the simulated motions and the empirical coherency loss function is shown in figure 3.6. The numerical results show that the simulated ground motions were compatible with the New Zealand design spectra and the empirical coherency loss function.

Figure 3.4 Simulated ground motions: (a) Accelerations a_{g1}, a_{g2} and a_{g3} and (b) displacements u_{g1}, u_{g2} and u_{g3} for weak correlation of excitations of soft soil condition

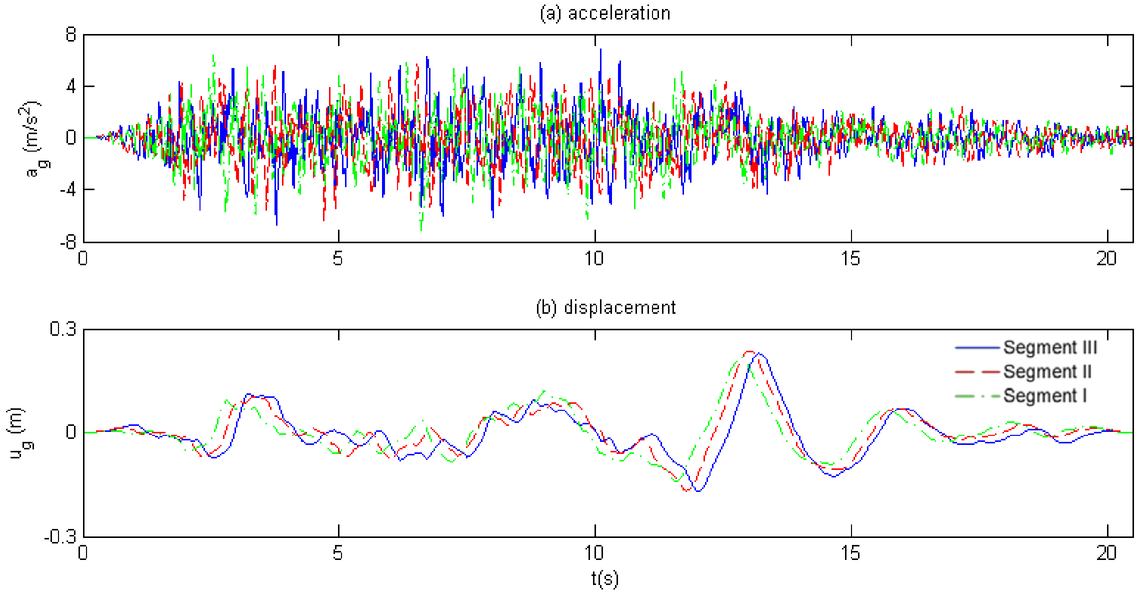


Figure 3.5 Comparison of the response spectra of the simulated ground motions (thin curves) and the target design spectra (bold curves) for three different soil conditions

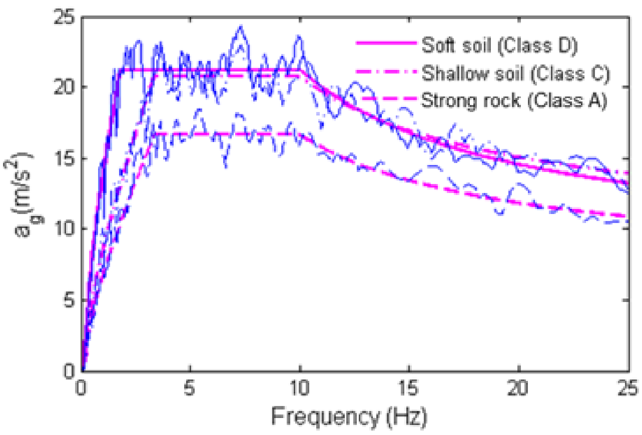
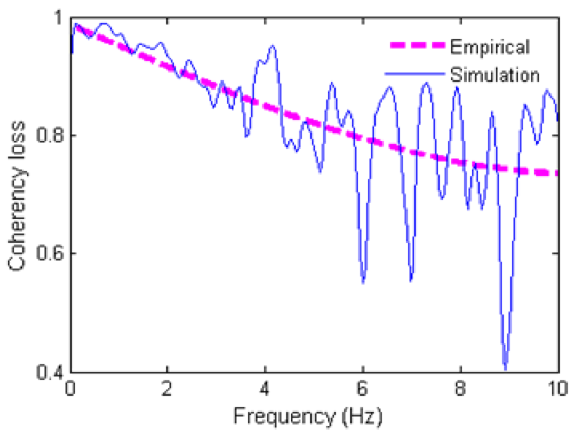


Figure 3.6 Comparison between coherency loss of simulated highly correlated ground motions with distance of 100m and model coherency loss function



3.2 Results and discussion

The results were obtained from testing three identical models with fixed supports subjected to uniform excitations, uniform excitations with time delay and spatially varying excitations. The simulated time histories, comprising ground motions of soft soil condition with high, intermediate and weak correlations, and ground motions of shallow soil condition and strong rock condition with high correlation, were modified according to the scaling law and used to excite the shake tables. The considered cases are summarised in table 3.5. To assure generality of the results, for each case, 20 sets of ground motions were used, resulting in a total of 100 sets of spatially varying ground motions to be considered. For ground motions of strong rock condition, the effects of pounding on one or both sides were studied.

To enable a more general interpretation of the results, the structural responses were normalised by the reference responses, which were obtained by averaging the maximum responses of the bridge segment I for the reference cases. The reference case was the situation when the bridge segment I was excited by the ground motions based on design spectra for different soil stiffnesses at site I without pounding. From the corresponding 20 maximum values, ie the peak absolute displacements and the peak bending moments obtained respectively from the 20 sets of ground motions at site I, the averages of the maximum values were calculated and used to normalise the corresponding results. The averaged maximum displacement was used to normalise the girder displacement and the relative displacement between any pair of bridge structures, and the averaged maximum bending moment was used to normalise the bending moment at all supports of the three bridge segments. The normalised results can be interpreted as amplification or reduction factors caused by the spatial variation of ground motions and pounding between adjacent bridge segments. The reference absolute displacement and the reference bending moment at the pier support are shown in table 3.5. The normalised results are compared between the different conditions to illustrate the effect of the excitation characteristics. The amplification or reduction factors derived from the normalised results are only valid for the cases considered in this report or for very similar cases. The results are compared to illustrate the consequences of different considered ground motions and the number of bridge segments involved for the bridge response.

Table 3.5 Reference responses due to ground motions on different soil conditions for normalising the results

Case	Soil condition	Ground motion correlation	Reference response due to ground motions	
			Absolute displacement (mm)	Bending moment (Nm)
1	Soft soil	High	4.15	0.798
2	Soft soil	Intermediate	4.15	0.798
3	Soft soil	Weak	4.15	0.798
4	Shallow soil	High	3.02	0.524
5	Strong rock	High	2.33	0.417

3.2.1 Effect of spatial variation of ground excitations

To investigate the influence of spatially varying ground motions on the bridge responses, the results obtained with spatially varying ground motions were compared with those due to uniform ground motions. Figure 3.7 shows the time histories of the absolute displacements of segment I when the two adjacent segments were subjected to 0.2s delayed but perfectly correlated, spatially varying and uniform ground motions with and without pounding. The 5s window of the time histories is for the purpose of clear presentation of the most significant responses. As shown in this figure, uniform ground motions resulted in the largest absolute displacements of segment I (the solid line) when pounding was not involved, while the bridge model responded least to the time delayed ground motions with pounding (the dotted line). This is because pounding can, if present, impede girder movement, leading to smaller absolute girder displacements, compared with the non-pounding response, where the dissipation of earthquake-induced energy depends only on structural damping. When considering spatially varying ground motions (the dashed line), the absolute displacement of segment I appears to be less than that caused by uniform ground motion most of the time, except at 1.6s and 2.3s. The results imply that uniform ground motions can cause a larger absolute displacement than spatially varying ground motions due to the lack of impediment by pounding.

Figure 3.7 Time histories of the absolute displacements of segment I under highly correlated ground motions of soft soil condition

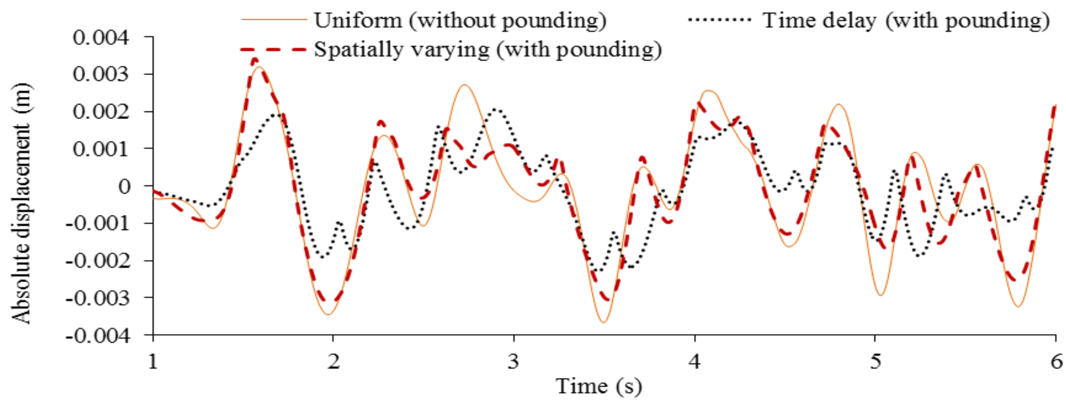


Figure 3.8 compares the bending moments of pier A at the support between the cases where pounding is and is not considered. The results indicate that spatially varying excitations caused smaller bending moments than uniform ground motions (by comparing the dashed line with the solid line). However, there were instances when the bending moments (at circled peaks) caused by the time delayed ground motions with pounding were greater than those resulting from uniform ground motion without pounding (by comparing the dotted line with the solid line). The results show that pounding can reduce or magnify the bending moment. However, if focusing only on the maximum response (marked by a cross), pounding reduces the bending moments at the support of pier A.

Figure 3.8 Time histories of the bending moment at the support of pier A under highly correlated ground motions of soft soil condition

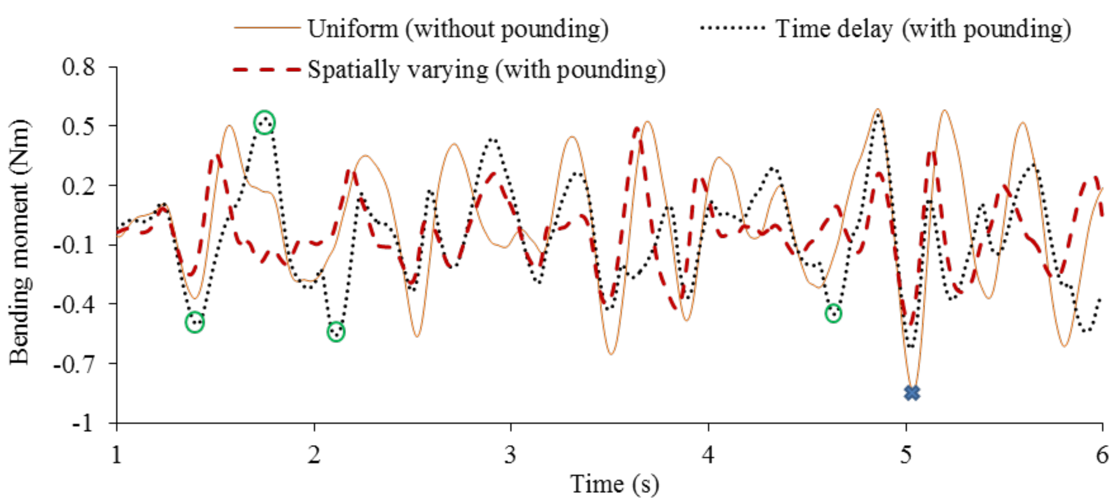
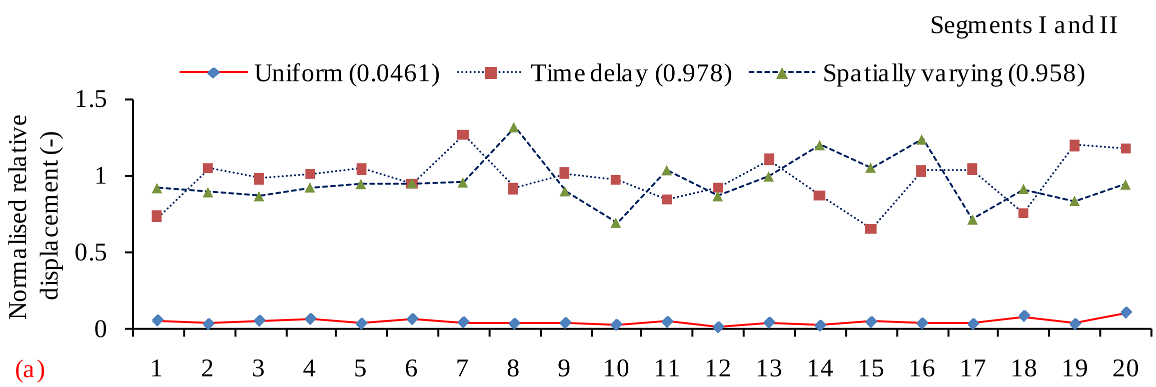
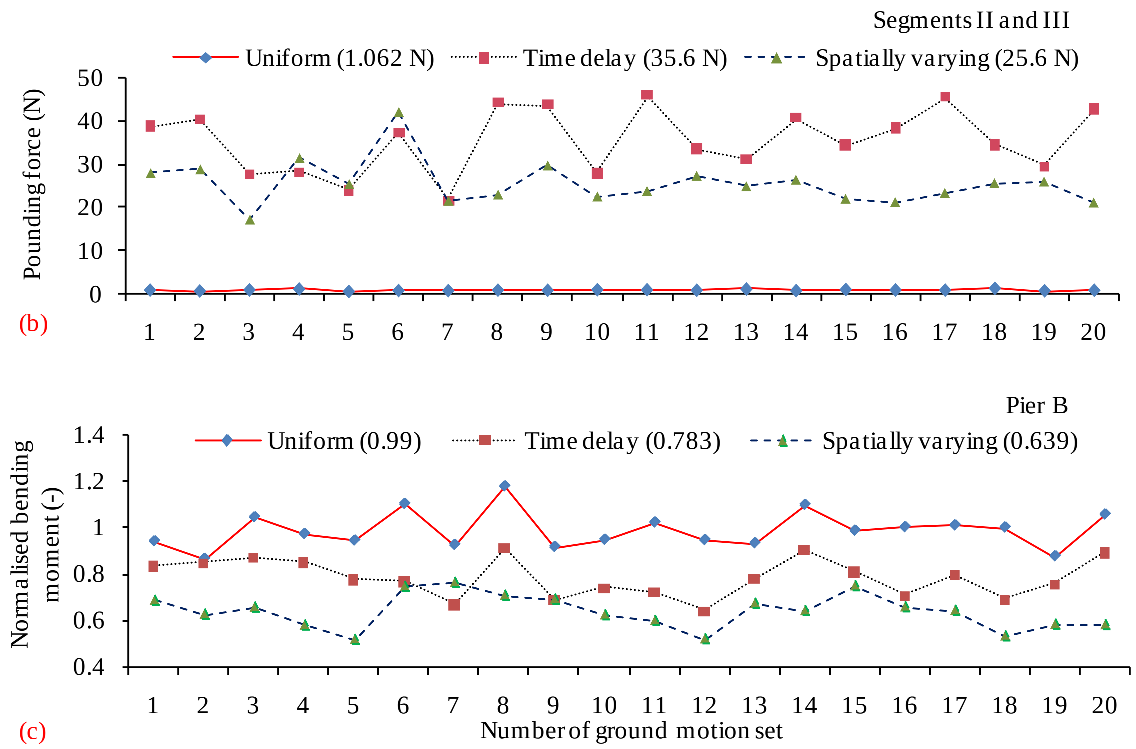


Figure 3.9 shows the influence of spatial variation of ground motions on the bridge response with pounding. The maximum opening relative displacements between adjacent bridge girders for all 20 sets of ground motions, depicted in figure 3.9(a), are normalised by the reference displacement, ie 4.15mm (as given in table 3.5). The number in the bracket is the average of the 20 maximum values of each model response. The reduction factors for the girder relative displacements due to time delayed and spatially varying ground motions are very similar, having averaged maximum reduction factors of 0.978 and 0.958, respectively. The extremely small values representing the normalised opening relative displacements resulting from uniform excitation are attributable to the in-phase movement of the bridge segments due to their equal fundamental frequencies. Figure 3.9(b) compares the maximum pounding forces corresponding to the 20 excitations of different cases, ie uniform, time delayed and spatially varying ground motions involving all three segments. From this figure, the excitations with 0.2s time delay resulted in greater pounding force development than spatially varying ground motions (comparing an averaged maximum pounding force of 35.6N with 25.6N). This revealed that spatially varying ground motions considering only time delay (the wave passage effect) can overestimate impacts compared with those considering both the wave passage effect and the coherency loss effect. According to figures 3.9(a) and (b), spatially varying ground motions can result in significant opening relative displacements and pounding forces despite having equal structural fundamental frequencies, whereas uniform ground motions, as expected, led to negligible relative movements and girder pounding due to in-phase movements. Consequently, even though the adjacent structures possess the same dynamic properties, assuming uniform ground motions will underestimate pounding potential and relative displacement between adjacent segments. This observation shows the inadequacy of most current design regulations which advocate matching natural frequencies but neglect the effect of the spatial variation of ground motions. The normalised maximum bending moments at the support of pier B, which are caused by the same set of uniform, time-delayed and spatially varying ground motions, are depicted in figure 3.9(c). These observations are the same as those found in figure 3.8, namely that at pier B, time delayed excitations overestimated the activated bending moments compared with those from spatially varying excitations, but both are less than those caused by uniform ground motions possibly due to poundings. Consequently, a conservative approach to selecting design bending moments based on the response to uniform ground motion could significantly overestimate the bending moments activated at the pier support. In the case considered, it is more than 35%.

Figure 3.9 Consequence of spatially varying ground motions: (a) normalised opening relative displacements between segments I and II, (b) pounding force between segments II and III, and (c) bending moment at pier B, due to highly correlated ground motions of soft soil condition considering three segment pounding; the average value of each case is given in the bracket

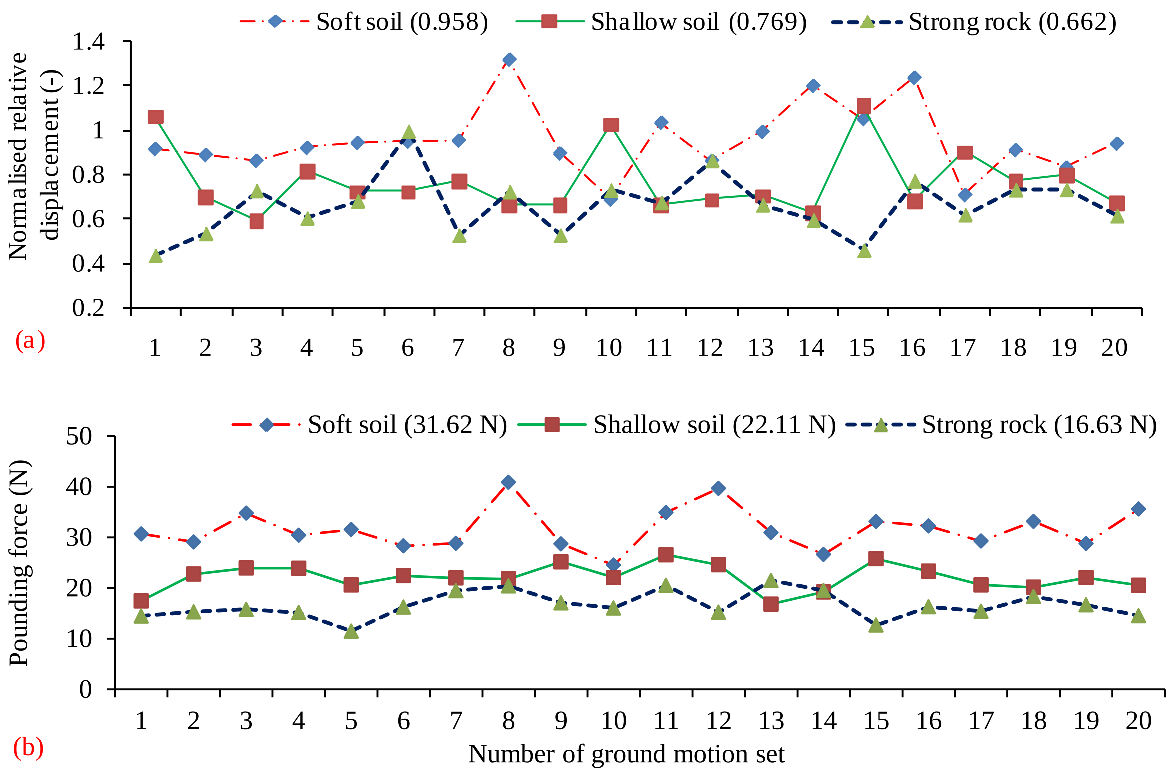




3.2.2 Effect of ground motion variations due to different soil conditions

Soil conditions can have a significant influence on the ground motions that bridge segments will experience in an earthquake. To understand the influence of different soil conditions, the three-segment bridge model was subjected to excitations corresponding to different site conditions. Figure 3.10 shows the normalised maximum opening relative displacements and pounding forces between segments I and II due to highly correlated ground motions of soft, shallow and strong rock conditions. Based on the reference response indicated in table 3.5, the averaged maximum normalised opening relative displacement of all 20 sets of ground motions of soft soil condition was found to be 0.958, which corresponds to 3.98mm being the maximum opening relative displacement between segments I and II. The considered ground motions corresponding to the shallow soil and strong rock conditions resulted in smaller separation distances with reduction factors of 0.769 and 0.662, respectively, which led to corresponding averaged maximum opening relative displacements of 2.32mm and 1.54mm. From figure 3.10(b), the averaged maximum pounding force between segments I and II when experiencing ground motions of soft soil condition was found to be 31.62N, which is 1.43 times that caused by ground motions of shallow soil conditions, and 1.9 times that due to motions of strong rock site. According to this study, the pounding response of segments I and II resulting from ground motions of soft soil condition was almost twice as large as that caused by ground motion of strong rock condition. The observation of the pounding force from figure 3.10(b) is assumed to follow the New Zealand design spectra reasonably well. Noting that the fundamental frequency of the bridge model, 1.97Hz, coincides with the highest values of the New Zealand soft soil design spectrum, shown in figure 3.5, the soft soil spectrum is, therefore, the most severe of the three soil spectra considered, over the frequency range of interest.

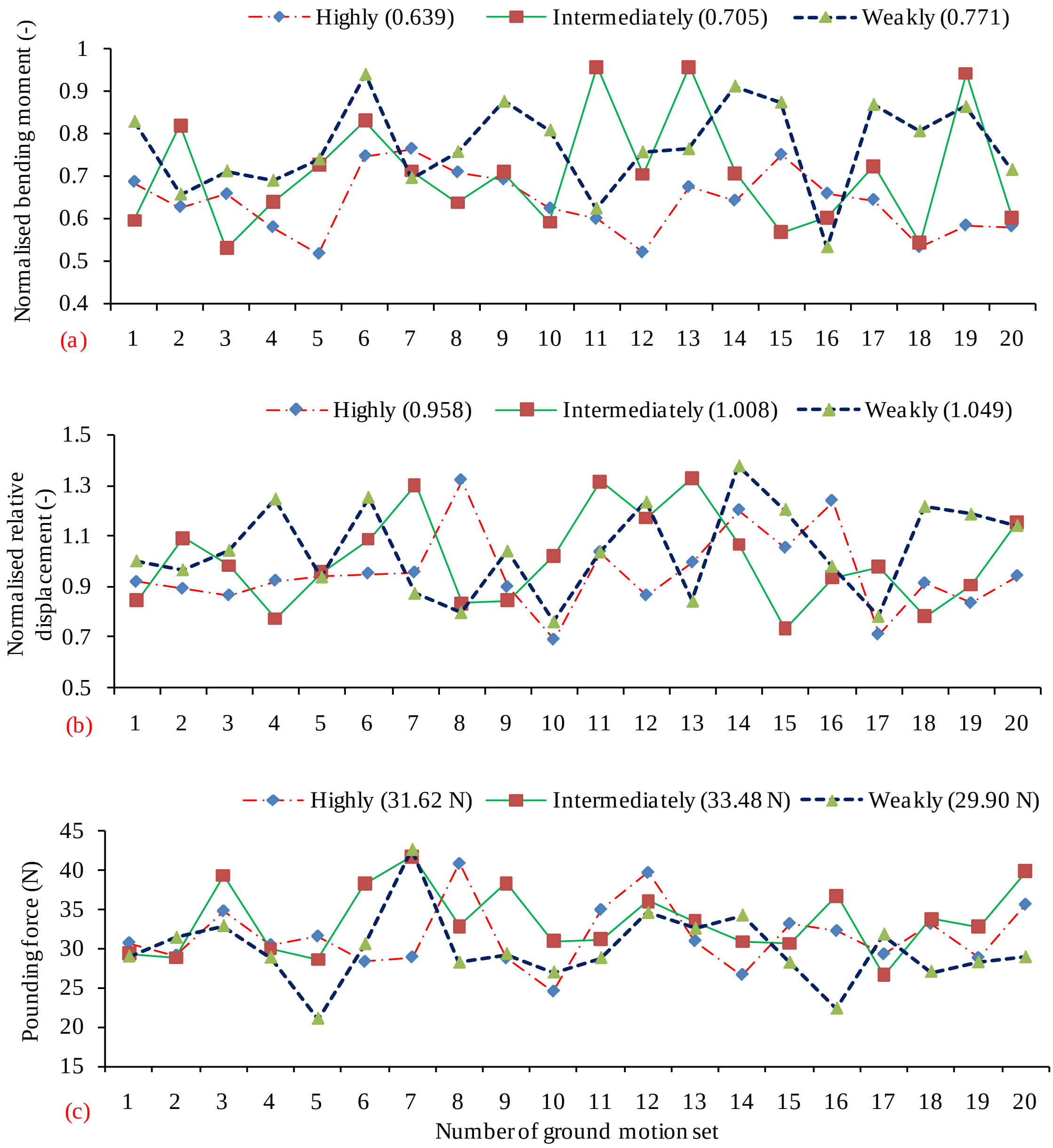
Figure 3.10 Consequences of site soil condition for the maximum values of (a) the normalised relative displacement and (b) the pounding force between bridge segments I and II assuming high correlation of the excitations



3.2.3 Effect of ground motion variations due to coherency loss

Figure 3.11 compares the response of segments I and II with pounding involving all three segments and considering highly, intermediately and weakly correlated excitations based on the soft soil spectrum. As shown in figure 3.11(a), high, intermediate and weak correlation of motions of soft soil condition resulted in smaller averaged maximum bending moment with the reduction factors of 0.639, 0.705 and 0.771, respectively. These correspond to actual pier bending moments of 0.51Nm, 0.563Nm and 0.615Nm, compared with the 0.798Nm reference bending moment. Figure 3.11(b) also shows that ground motions with weak correlation at subsequent supports can cause larger opening relative displacements than highly and intermediately correlated ground motions. According to figure 3.11(c), the maximum pounding forces due to weakly correlated ground motions were slightly lower than those due to the intermediately and highly correlated ground motions with the perverse exception that excitation set 7 caused the maximum pounding force of 42.7N, which is almost coincident with the intermediately correlated motions. The results show that highly and weakly correlated ground motions do not necessarily cause the smallest and largest pounding forces. In the case considered, the intermediately correlated ground excitation caused the largest pounding force.

Figure 3.11 Consequence of the coherency loss for the maximum values of: (a) normalised bending moment of pier B, (b) normalised relative displacement, and (c) pounding force between segments I and II under spatially varying excitations with three segment pounding



3.2.4 Effect of two-sided pounding

The time-histories shown in figure 3.12 depict the relative displacements between segments I and II due to spatially varying ground motions considering pounding involving two and three segments. Positive values represent opening relative displacements while negative values result from the occurrence of strong impacts causing the pounding head to induce inward deflection of the instrumented steel strip of the measuring head. Figure 3.12 indicates that pounding involving only two segments can, with few exceptions, develop larger opening relative displacements than pounding involving three segments when subjected to spatially varying ground motions.

Figure 3.12 Time histories of the relative displacement between segments I and II under intermediately correlated spatially varying ground motions of soft soil condition considering three or two segments with pounding effect

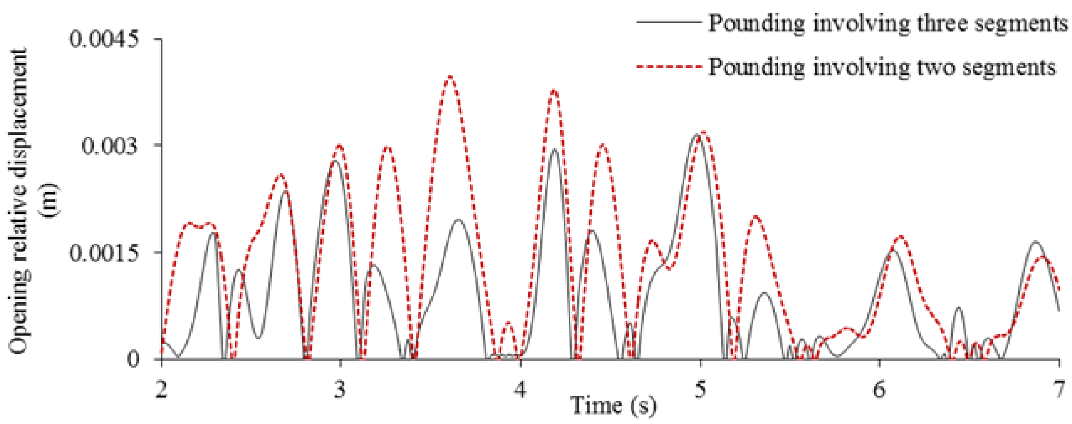


Figure 3.13 displays the pounding forces during impact of segments I and II considering two and three segments. Damage potential of bridge girders depends not only on the magnitude of the contact force, but also on the number of strong impacts. In contrast to the opening relative displacement response shown in figure 3.12, the impacts between segments I and II are slightly more significant when three segments are involved rather than two, implying that considering two bridge segments only results in large opening relative displacements but smaller strong pounding forces.

Figure 3.13 Time histories of the pounding force between segments I and II under highly correlated spatially varying ground motions of soft soil condition

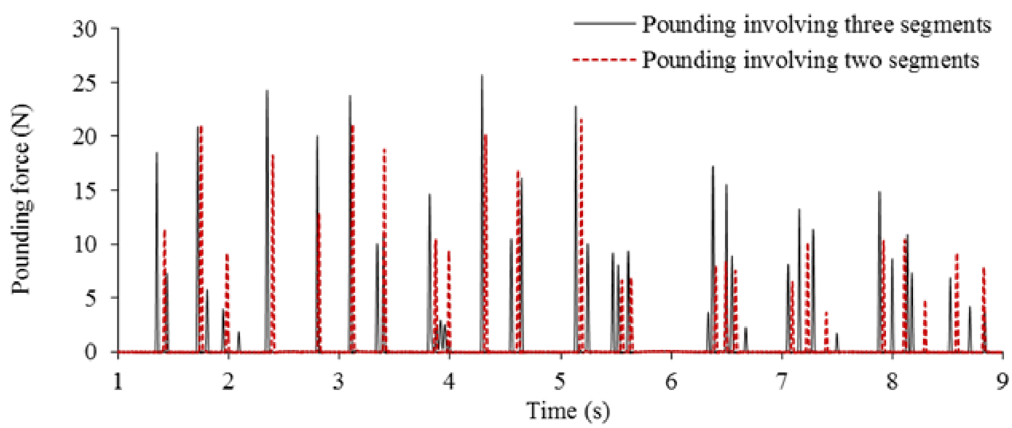
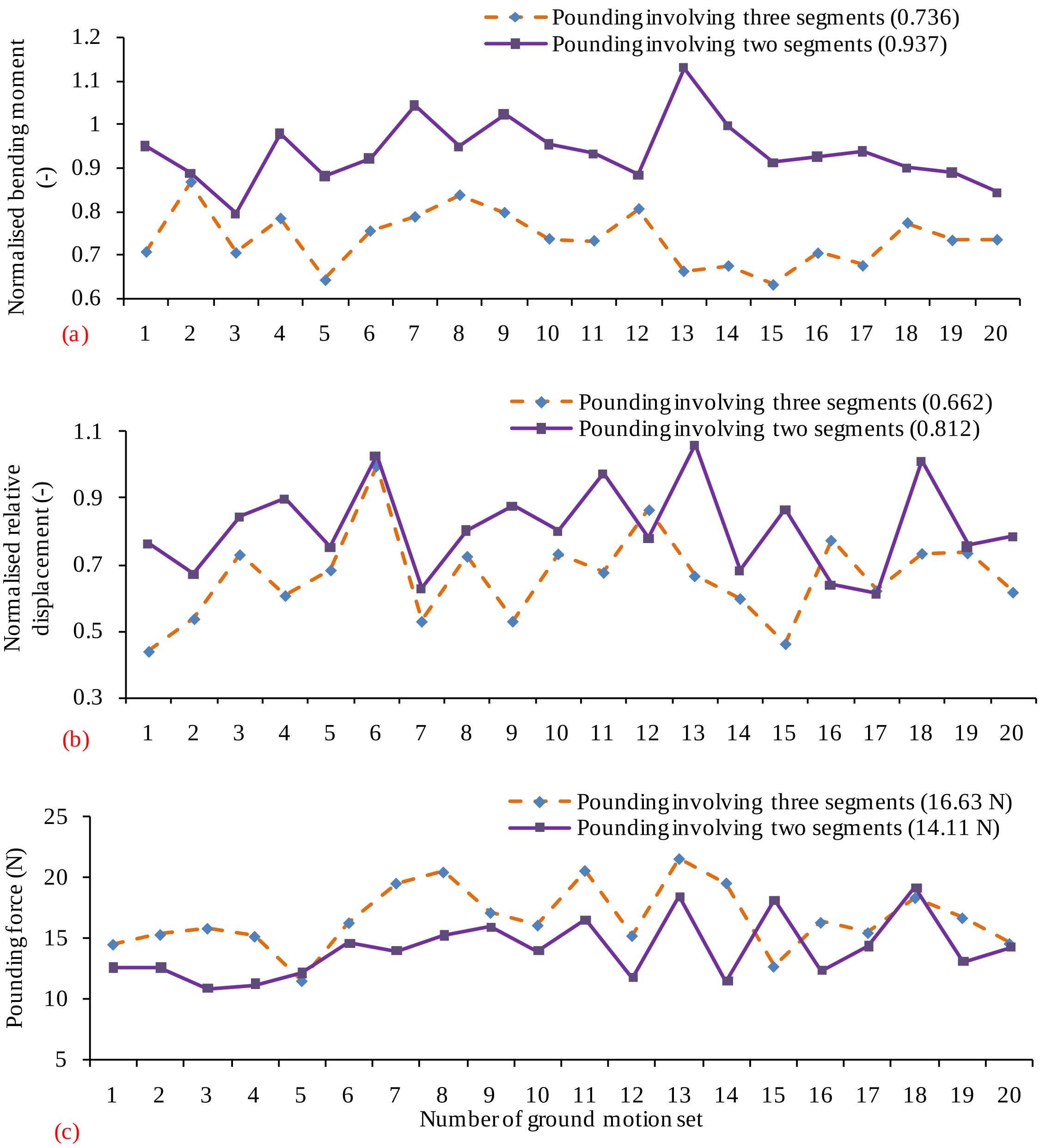


Figure 3.14 illustrates the effect of two-sided pounding (three-segment involvement) on the response of segment II, compared with pounding on only one side (two-segment involvement), under highly correlated ground motions of strong rock condition considering spatial variations. As shown in figure 3.14(a), smaller bending moments resulted at the base of pier C due to two-sided and not one-sided pounding (by comparing the reduction factor of 0.736 for three segments with 0.937 for two). The same observation can be found in figure 3.14(b) where pounding involving three segments led to a normalised maximum opening relative displacement of 0.662 between segments I and II, which was less than the response of 0.812 due to pounding on one side only. From the results shown in figures 3.14(a) and (b), the magnification of the bridge responses, ie the bending moment of pier C and the opening relative displacement of segments I and II, due to pounding from one side can be as significant as approximately 20% of the responses due to pounding on both sides. In contrast to the observations in figures 3.12 and 3.13, the pounding development was found to

be greater for two-sided pounding than one-sided, noticing that there was a 2.5N difference in the averaged maximum girder pounding forces between the two and three segment cases. These were the consequences of the additional impacts from segment III, which introduced more energy to segment II, but limited the movability of segment II, resulting in greater pounding force between segments I and II but smaller separation and smaller bending moment than those caused by single-sided pounding.

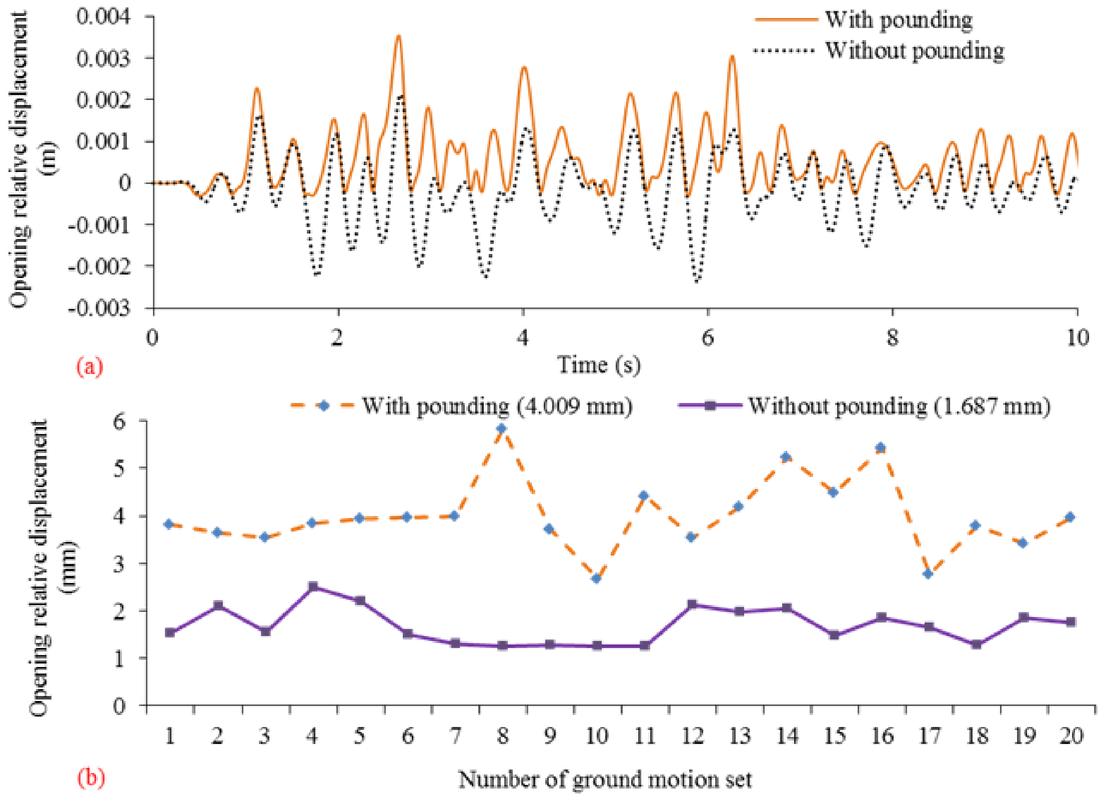
Figure 3.14 Consequences of one and two-sided pounding on: (a) bending moment at pier C, (b) the normalised relative displacement between segments I and II, and (c) the pounding force between bridge segments I and II under spatially varying excitations of strong rock condition



3.2.5 Effect of pounding on unseating potential

Figure 3.15(a) shows the time histories of the opening relative displacement of segments I and II when three segments under highly correlated ground motions of soft soil condition for set 12 (as an example) are considered with and without pounding. For the non-pounding cases, segments I and II were given an initial separation sufficient to avoid subsequent pounding. The negative values in this figure represent closing relative displacements. It is clear that impacts caused increases in the opening relative displacements (dotted line) of segments I and II, compared with the non-pounding case (solid line). Figure 3.15(b) summarises the peak opening normalised relative displacements of segments I and II for all 20 ground motion sets with and without pounding. It can be readily seen that pounding can potentially double the opening relative displacements, compared with the non-pounding case (increase from 1.687mm to 4.009mm), and hence increases the seat length required to prevent unseating. This finding has provided evidence for the suspicion (Raheem 2009) that pounding can contribute to the girder unseating. However, this observation is based on a model in which the influence of abutments was not considered.

Figure 3.15 Effect of pounding on unseating potential due to set 12 excitation: (a) Time histories of relative opening displacement, and (b) the normalised maximum opening relative displacement between segments I and II, with and without pounding, under highly correlated ground motions of soft soil condition



3.3 Analytical simulation of pounding

The development of the experimental pounding forces was analytically simulated using the Hertz contact model with a non-linear damper (the Hertz-damped model). From the simulation, it was found that the Hertz-damped model had the ability to predict the expected pounding force to a reasonable degree of accuracy.

The Hertz-damped model is defined as follow:

$$F = \begin{cases} k_h(u_1 - u_2 - g)^{3/2} + c_h(\dot{u}_1 - \dot{u}_2) & u_1 - u_2 - g > 0 \\ 0 & u_1 - u_2 - g \le 0 \end{cases} \qquad \text{(Equation 3.3)}$$

where k_h = nonlinear impact spring stiffness

c_h = impact damping coefficient

u_1 = displacement of bridge superstructure segment I

u_2 = displacement of bridge superstructure segment II

g = expansion joint gap length

$\dot{u}_1 - \dot{u}_2$ = relative approaching velocity of the bridge segments.

In order for the pounding force calculated from the impact model to be compared with the pounding force measured from the model bridge, the impact stiffness and the damping coefficient for the pounding force measuring device used on the model bridge were determined to be 244kN/m and 30.46N s/m respectively.

Based on the relative displacement and velocity obtained from the experiments, the selected impact model was utilised to calculate the pounding force. Figure 3.16 shows the calculated pounding force from the impact model and the measured pounding force from the bridge model for an earthquake event. Comparing the lower two plots in figure 3.16, there is reasonable correlation between the measured and calculated pounding force. The magnitude of force is mostly similar; however, between approximately six and nine seconds into the earthquake event, there were some noticeable differences between the calculated and measured force. Because the displacement transducer was not able to resolve very small changes (micro metres) in displacements during pounding, the computed pounding forces, which are heavily dependent on the relative displacements, did not reveal a time history that was highly consistent with the directly measured force values. However, the actual degree of accuracy achieved shows the potential of using an impact model to calculate the expected pounding force at a bridge expansion joint.

The difference between the calculated and measured pounding force was determined at each time step and the average of these differences was determined over the total duration of each simulated earthquake. This provided a more quantifiable difference between the calculated and measured pounding force. Table 3.6 shows the average difference for four earthquakes from each of the five ground motion cases considered. The highlighted value represents the average difference for the results displayed in figure 3.16. Table 3.6 indicates that the calculated pounding force is reasonably similar for most of the ground motions considered. Ground motion cases 4 and 5 were slightly more inaccurate than the others. This was attributed to the fact that the ground accelerations were lower for the harder soil types, and as a result the pounding force was smaller. This meant that the relative displacements were more subtle, which exacerbated the limitations in the relative displacements discussed previously.

Figure 3.16 Comparison of measured and calculated pounding force

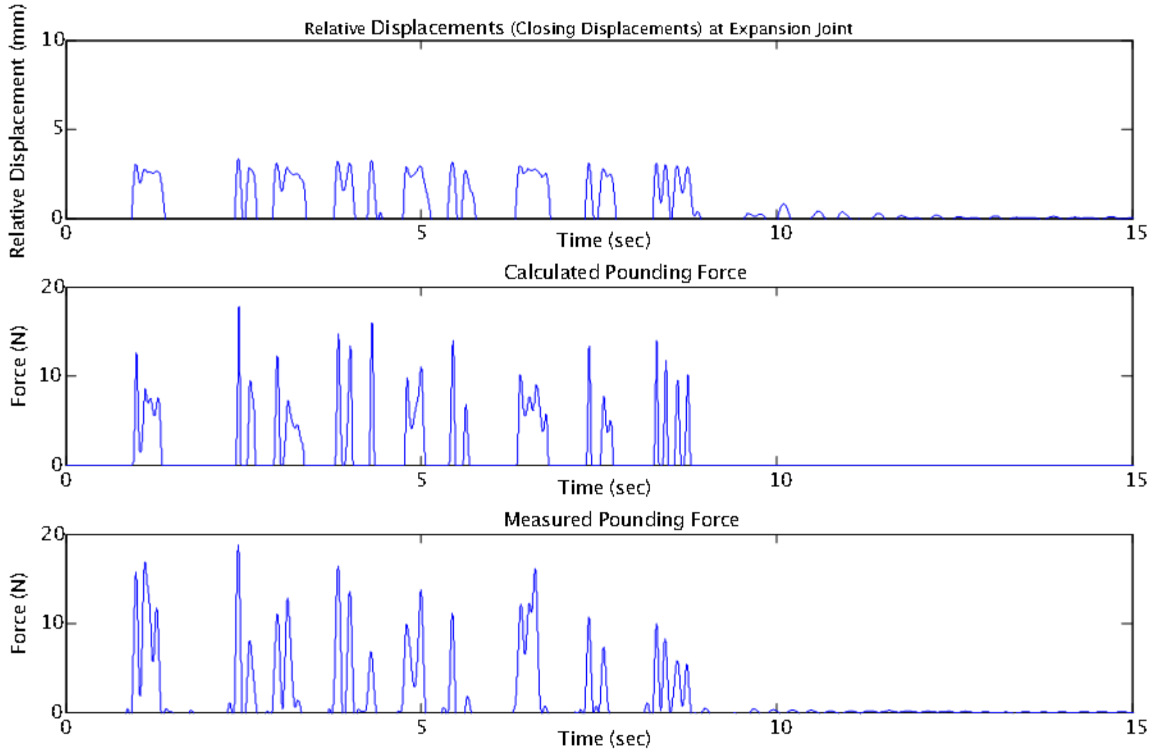


Table 3.6 Average difference between calculated and measured pounding forces

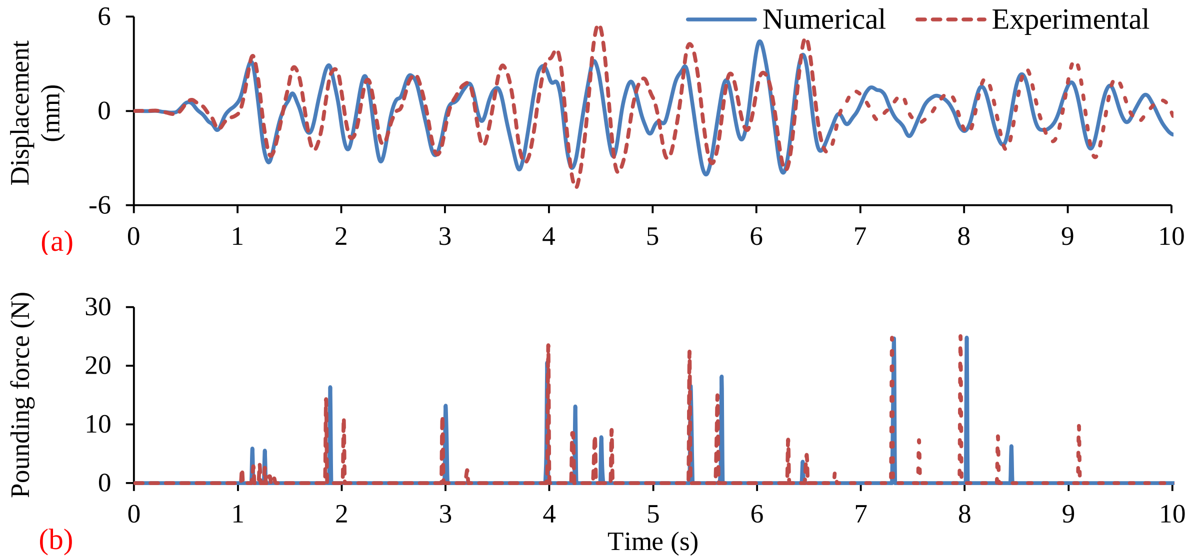
Case 1		Case 2		Case 3		Case 4		Case 5	
Earthquake record no.*	Average difference (N)	Earthquake record no.	Average difference (N)	Earthquake record no.	Average difference (N)	Earthquake record no.	Average difference (N)	Earthquake record no.	Average difference (N)
5	3.12	1	3.68	6	3.22	4	3.61	8	4.67
10	2.94	2	3.65	10	3.54	8	4.14	10	3.44
13	2.54	6	2.84	12	4.15	15	4.65	14	4.40
18	3.54	9	2.61	16	3.07	17	3.94	18	3.21

*The earthquake record number corresponds to the earthquake record from the 20 records generated for each ground motion case (see table 3.5).

3.4 Numerical simulation

To confirm the experimental results a finite element model of the experimental bridge model was constructed using ANSYS. Each of the bridge girders and piers was modelled by 100 and 33 beam elements, respectively. The impacts between the segments were modelled by a spring-dashpot element with spring stiffness and damping coefficient of 244 kN/m and 30.46 N s/m obtained from experiments. In total 500 elements were used. The initial gap between superstructure segments was modelled to be 0.1mm. Rayleigh damping was assumed for the first two natural modes of the bridge structures. The ground motion input for the finite element model was the same as the recorded shake table movements. Figures 3.17(a) and (b) compare the girder displacement of the bridge segment I relative to the ground without pounding effect and the pounding force development at the contact location of segments II and III under highly correlated ground motions of shallow soil condition. A reasonably good agreement between the numerical and experimental results is visible.

Figure 3.17 Comparison between numerical and experimental results: (a) Displacement of segment I relative to the ground and (b) pounding forces between segments II and III under highly correlated shallow soil excitation



3.5 Summary

The influences of spatial variation of ground motions and pounding on the responses of a three-segment scaled bridge model were investigated experimentally using three shake tables. The bridge segments were nominally identical and hence had the same dynamic properties and represented the most favourable bridge conditions with almost the same fundamental frequencies as recommended by most current design specifications. They were subjected to stochastically simulated ground motions based on the response spectra of the New Zealand earthquake loading standard for different soil types, ie soft, shallow and strong rock soil. Of the experiments, 280 were performed with and 160 without considering pounding. Each case was performed with 20 sets of ground motions. Abutments and soil-structure interaction effect were not considered.

The study revealed:

1 With pounding, spatially varying ground motions can increase the relative displacement at the inter-segment expansion joints by as much as 20% of the average maximum absolute displacement caused by uniform ground motion, and cause large pounding forces. Neglect of spatial variation of ground motions will underestimate the damage potential due to pounding between the girders.

2 Bending moment at the base of piers increases or decreases as a result of pounding depending on the ground motion considered. Overall, the pounding results in a decrease of the maximum bending moment value. Horizontal displacement of the girders is similarly affected. In the cases considered, the average reduction of the maximum moment and absolute displacement were 35% and 5%, respectively.

3 The scaled bridge model responded more strongly to ground motions of soft soil condition than to those of shallow soil or strong rock conditions. This is probably because the New Zealand design spectrum for soft soil condition has the highest spectral values in the vicinity of the fundamental frequency of the prototype bridge. Therefore, more damage to the bridge structure is anticipated under ground motions of soft soil than in shallow soil and strong rock conditions. According to this study, the maximum opening relative displacement and pounding force due to ground motions of soft soil

condition are nearly twice as large as those caused by ground motions derived from strong rock condition.

4 Two-sided pounding can result in smaller bending moments at pier supports and opening relative displacements than one-sided pounding. The impact on both ends of a girder confines its movement, which hinders the development of large opening relative displacements. However, the research observed that the pounding forces resulting from two-sided pounding may exceed those caused by one-sided pounding. When three segments are involved, the middle segment can gain more energy from the impact with the additional neighbouring girder, giving greater potential for large pounding forces.

5 Matching the frequency of adjacent segments, the only measure commonly suggested by most current design regulations to avoid pounding, is often inadequate when spatially varying ground motions are expected.

6 Pounding can contribute to girder unseating. From the cases considered, pounding can cause twice the opening relative displacement of non-pounding cases. However, further investigations including abutment effect are necessary to clarify the consequences of overall system performance.

7 Results obtained from one-sided pounding as performed in most research in the past may not lead to accurate predictions of response from two-sided pounding.

8 The Hertz-damped impact model has shown the ability to calculate the pounding force expected between bridge superstructures during earthquakes.

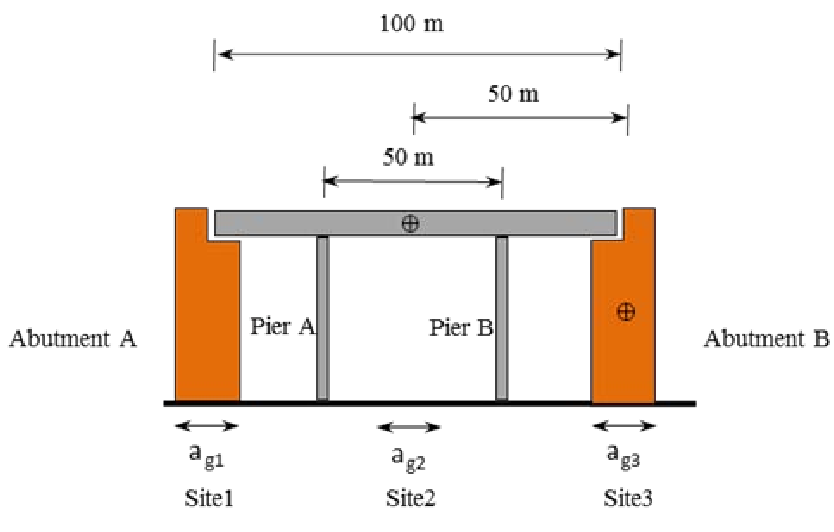
4 Pounding of an abutment-bridge system

Bridge damage due to pounding at joints of girders and abutments has been observed in many major earthquakes. This pounding phenomenon is common owing to the differences between the dynamic characteristics of a girder and the relatively rigid adjoining abutments. Hence, a good understanding of girder-abutment pounding is critical for bridge design. However, previous studies on this topic were performed mainly numerically and abutment excitation was hardly considered. Very limited experimental investigations were conducted and none of them considered the effect of abutment motion on bridge pounding response. The objective of this study was to experimentally evaluate the influence of abutment movement on the pounding behaviour of a system comprising a single segment bridge and its abutments using three shake tables. The spatially varying ground excitations were simulated based on the New Zealand design spectra for soft soil, shallow soil and strong rock conditions using an empirical coherency loss function as described in chapter 3. Results showed that with the spatial variation effect of ground motions, a consideration of movable abutments led to greater opening relative displacement with the adjacent structure and different pounding potential than with fixed abutments.

4.1 Prototype structure and model

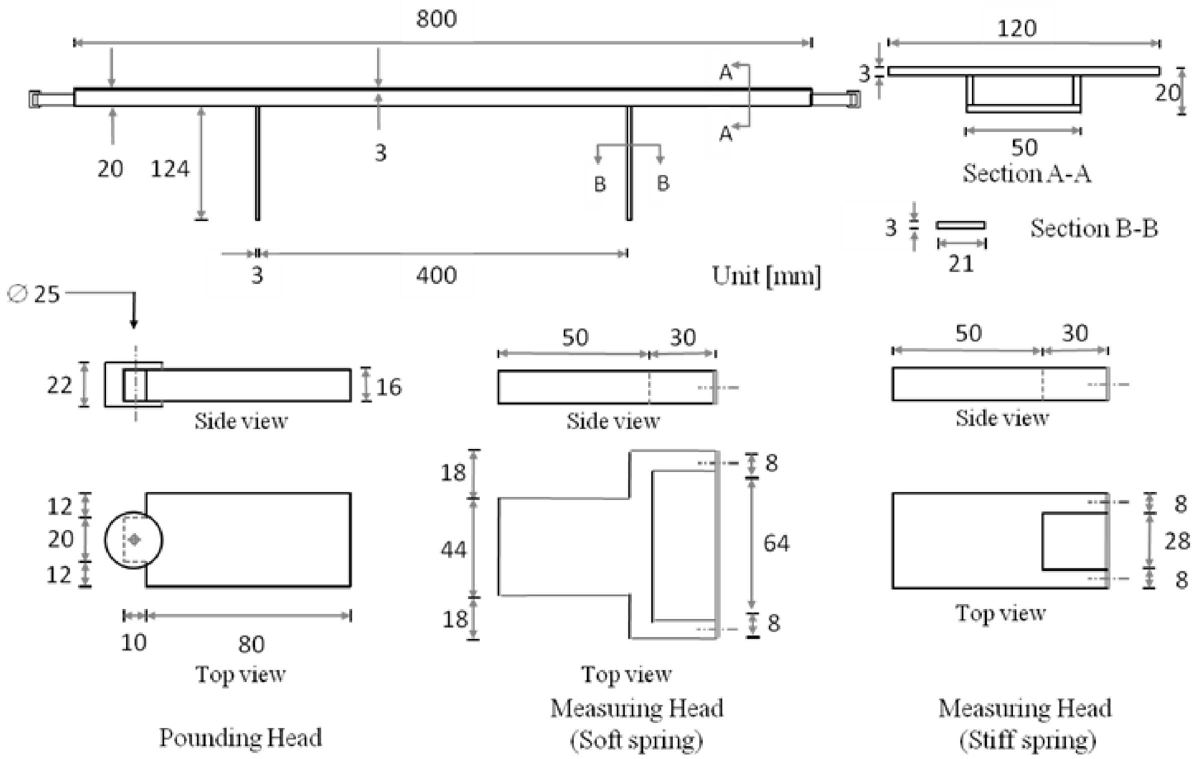
In order to study the response of a bridge structure to the effect of pounding from adjacent abutments, a system consisting of a one-segment bridge and abutments was considered (figure 4.1). To limit the influence factors, it was assumed that the system experienced ground excitation a_{gn} along the span direction only, where n was the number of the considered sites (n = 1, 2 and 3). The distance from the centre of the girder to the centre of each abutment was assumed to be 50m. The bridge model used in this study was based on the same prototype structure as in chapter 3. For more details about the scaling and construction of the model, please refer to section 3.1.

Figure 4.1 Considered prototype system



To measure pounding forces generated between the abutments and the girder, pounding heads and measuring heads were utilised (see section 3.1.2 for more details). Two types of measuring head, ie stiff and soft, with different spring steel strip lengths were used for considering a different contact stiffness of the contact surface between the abutment and the girder. The contact stiffnesses for the stiff and soft measuring heads were found to be 244N/mm and 30.5N/mm, respectively. Detailed dimensions of the heads, together with the model's dimensions are shown in figure 4.2.

Figure 4.2 Schematic drawing of the model with details of the pounding and measuring heads



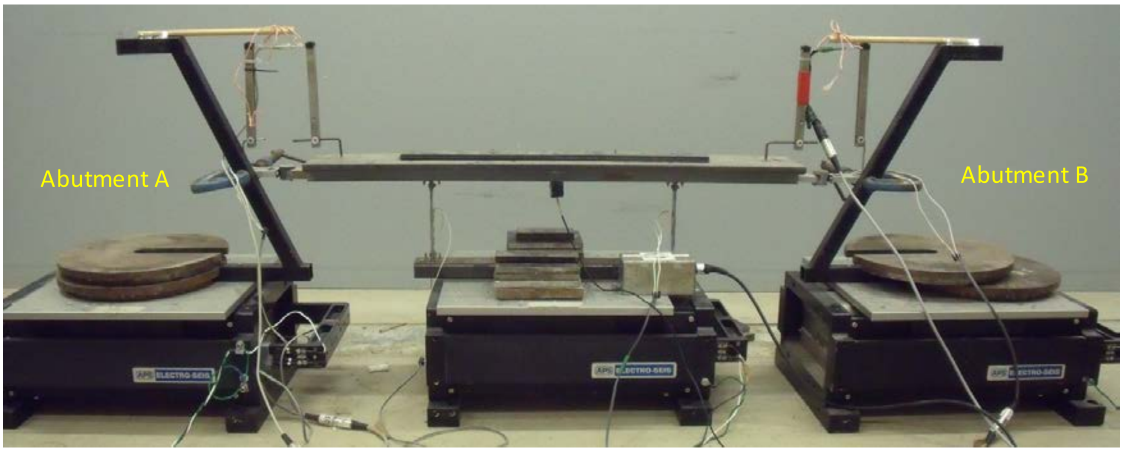
4.2 Equipment and testing

In order to provide varying ground motions to the bridge-abutment system, three APS Model 400 Electro-Seis shake tables were used. A strain gauge was bonded to each pier base. Relative displacement transducers were installed between the girder end and the abutment. Unidirectional accelerometers with a measuring capacity of ± 2g were fixed to the girder and the shake table platforms.

In this research, only fixed base supports to the piers were considered, and the SSI effect was excluded. Fundamental frequencies ranging from 1.4Hz to 2.6Hz with an increment of 0.2Hz were achieved by placing different weights on the bridge deck allowing consideration of bridges with different dynamic properties. In order for the shake tables to apply the same accelerations to models of different weight, additional weights were attached to maintain the same total weight on each shake table. The damping ratios of the bridge segment for different fundamental frequencies were determined by performing snap-back tests and the values were found to increase from 1.6% to 3% as the fundamental frequency of the bridge increased from 1.4Hz to 2.6Hz.

In order to investigate the effect of abutment movement and spatial variation of excitations on the pounding response of the bridge model, both fixed and movable abutments were tested. The fixed abutments were kept in place at all times. The movable abutments were subjected to spatially varying ground motions simulating near-fault earthquakes based on New Zealand design spectra (NZS 1170.5 2004). The soil conditions tested were Class A (strong rock), Class C (shallow soil site) and Class D (soft soil site). The incorporation of coherency loss effect was the same as in section 3.1.4. The computer-based operation of the shake tables and the data logging system were controlled by a MATLAB program. The final setup of the shake tables, the abutments and the bridge model is shown in figure 4.3.

Figure 4.3 Setup of the test system



4.3 Spatial variation of ground motions

The simulation of spatially varying ground motions was achieved using the same method described in section 3.1.4, except d_{ij} is 50m. Figure 4.4 shows one set of the simulated ground motion time histories with peak ground acceleration of 1.2g. Figure 4.5 illustrates the difference in the coherency loss of the simulated ground motions and the empirical coherency loss function.

Figure 4.4 Simulated ground: (a) accelerations and (b) displacements for site 1 to 3 with a 50m site distance

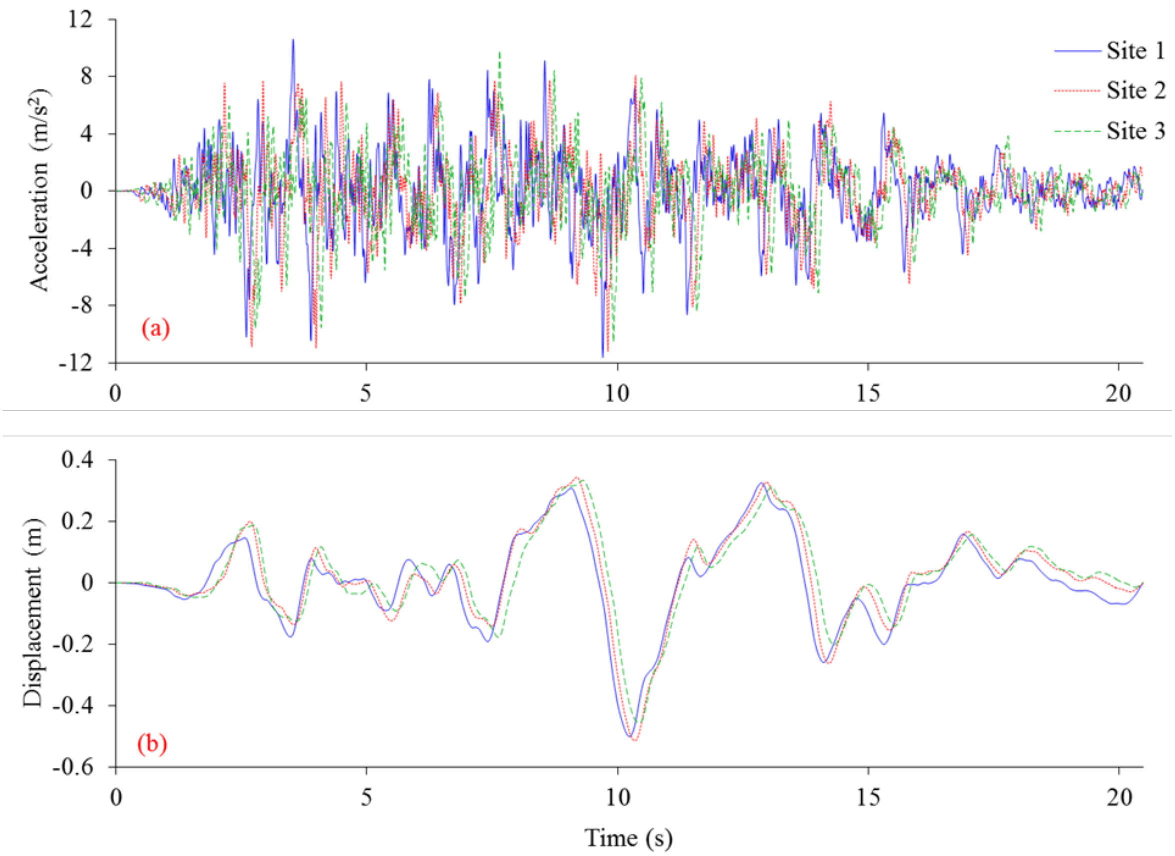
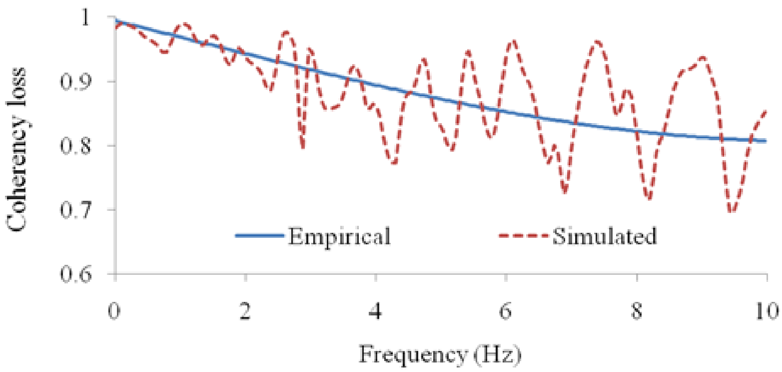


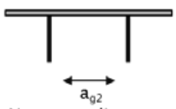
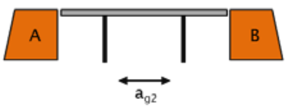
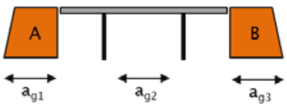
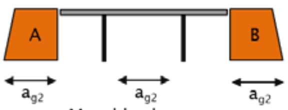
Figure 4.5 Comparison between coherency loss of simulated highly correlated ground motions with a distance of 50m and model coherency loss function



4.4 Results and discussion

The results were obtained from tests involving a bridge-abutment system with and without pounding. The bridge model had different fundamental frequencies as given in table 4.1. The abutments were subjected to uniform and spatially varying excitations, and the bridge-abutment responses resulting from the moving abutments were compared with those obtained from the fixed abutments. Abutments with two kinds of contact stiffness were considered, ie the ‘stiff spring’ and the ‘soft spring’. The discussion focuses mainly on the abutments with stiff spring as their stiffness after back-scaling, ie 2.93MN/mm, was more realistic for a real bridge compared with the soft spring. The cases are summarised in table 4.1. To obtain general conclusions, 20 sets of ground motions were used for each soil condition, resulting in the consideration of 100 sets of stochastically independent, simulated spatially varying ground motions.

Table 4.1 Summary of the parameters considered for the tests

Tests	Fundamental frequencies	Contact stiffness	Ground motions	
			Soil conditions	Coherency
 Non-pounding test	1.4 ~ 2.6 Hz (Corresponding to prototype frequencies of 0.7 ~1.3 Hz)	N/A*	Soft soil Shallow soil Strong rock	N/A N/A N/A
 Fixed abutment	1.4 ~ 2.6 Hz	Stiff & Soft Stiff spring: 244 N/mm Soft spring: 30.5 N/mm	Soft soil Shallow soil Strong rock	N/A N/A N/A
 Movable abutment Non-uniform excitations	1.4 ~ 2.6 Hz	Stiff & Soft	Soft soil Soft soil Soft soil Shallow soil Strong rock	Highly Intermediately Weakly Highly Highly
 Movable abutment Uniform excitations	1.4 ~ 2.6 Hz	Stiff & Soft	Soft soil Shallow soil Strong rock	N/A N/A N/A

*N/A: Not applicable

To generalise the interpretation of the results, some of the structural responses, ie bending moment at pier support and opening relative displacement at girder ends, were normalised by the reference values (shown in table 4.2), which were the averages of the 20 maximum bending moments and the maximum displacements of the bridge girder due to the 20 corresponding ground motions without pounding. For each soil condition, an average of the 20 normalised responses for the same structural frequency was used for comparison. Owing to the asymmetry of earthquakes, the pounding forces and relative displacements at different girder ends are not the same even though the structure is symmetric. Moreover, once the model has displaced, the system is no longer symmetric. Therefore, for comparison of the average maximum response, the greater value at either side is always chosen in order to obtain the most conservative conclusions. The ratios of the normalised results are only valid for the cases considered or for very similar cases.

Table 4.2 Summary of the reference values used for normalising the response

Soil condition	**Reference response due to ground motions of different soil conditions Absolute displacement (mm)/bending moment (Nm)**						
	1.4Hz	1.6Hz	1.8Hz	2Hz	2.2Hz	2.4Hz	2.6Hz
Soft soil	4.11/0.798	4.15/0.788	4.17/0.803	4.18/0.800	4.20/0.784	4.30/0.785	4.30/0.783
Shallow soil	2.91/0.555	2.93/0.540	2.99/0.538	3.03/0.531	3.09/0.532	3.31/0.568	3.33/0.572
Strong rock	2.13/0.412	2.22/0.402	2.27/0.416	2.30/0.417	2.29/0.419	2.48/0.417	2.50/0.414

4.4.1 Effect of abutment movement

During an earthquake, abutments will inevitably move when their surrounding soil mass moves, meaning that the 'fixed abutment' assumption considered by earlier researchers does not normally arise. Owing to the complexity in incorporating all the influence factors, previous studies, for simplicity, often assumed an irrational fixed abutments condition and neglected the abutment excitation by the ground movements during an earthquake. Only the effect on bridge structures was considered. In contrast, this study used the fixed abutment case for comparison purposes only. To investigate the consequences of abutment movements for pounding response development in conjunction with the effect of spatially varying ground motions, the results obtained from fixed and movable abutment pounding experiments under spatially varying and uniform excitations were compared.

Figure 4.6 Consequence of movable abutments for the average normalised: (a) maximum bending moment, (b) maximum opening relative displacement, and (c) the average pounding force due to highly correlated excitations of soft soil condition with stiff contact

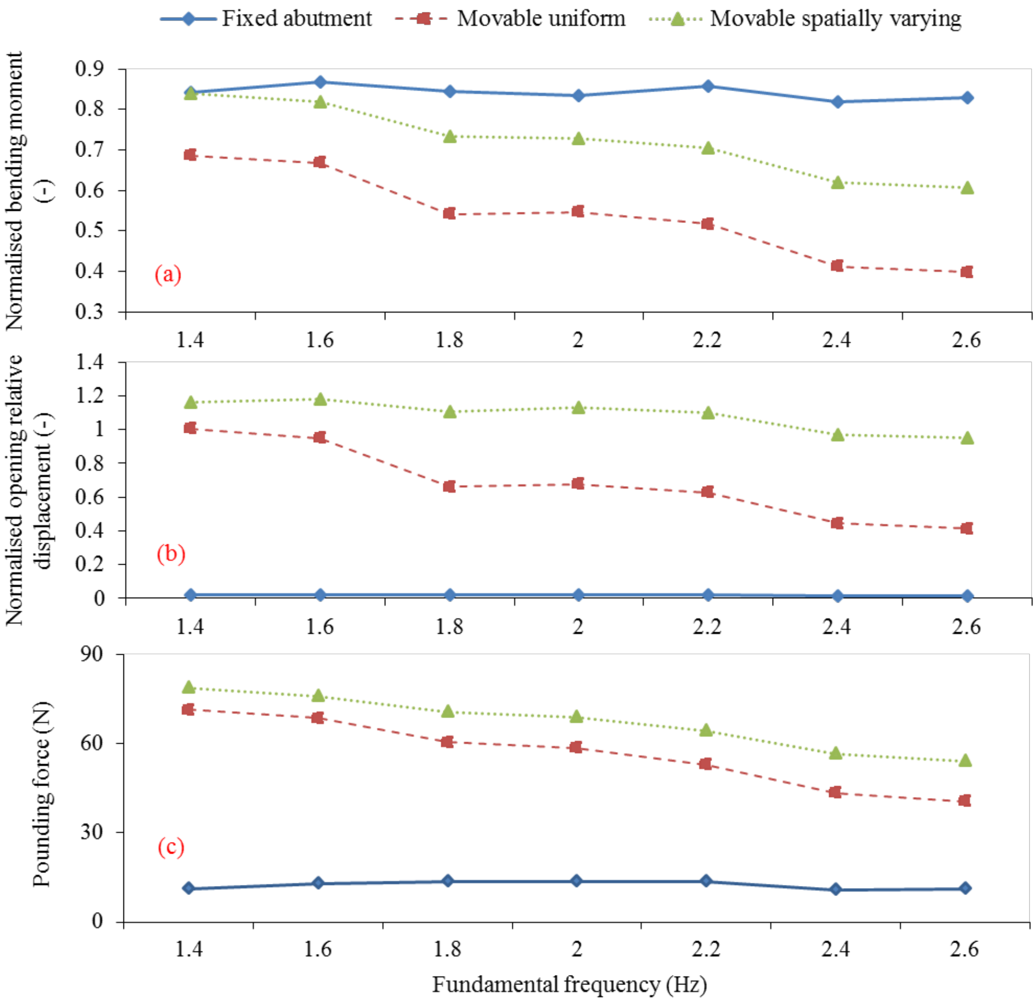


Figure 4.6 compares the average maximum structural responses with fixed and movable abutments under uniform and spatially varying ground motions with zero gaps. Figure 4.6(a) shows the maximum bending moment at pier A support normalised by the reference values given in table 4.2. As shown under the soft soil highly correlated ground motions, in all cases the greatest support bending moments occur when fixed abutments are assumed, and the average normalised maximum bending moments at pier supports appear to have very similar values, ie 0.85 on average. This is because the bending moment is strongly influenced by the pier sway. Since the initial gaps between the fixed abutments and the girder ends have a zero value which disables the girder movement (apart from small deformations of the stiff pounding heads), the bending of the bridge is therefore determined only by the site 2 ground movement, which is independent of the fundamental frequency of the bridge structure. Consequently, the support bending moments with fixed abutments should theoretically be almost constant regardless of the bridge frequency. The difference is caused solely by the small movements of the bridge deck under the varying site 2 ground motions that are stochastically independent and simulated for each case. With all the normalised bending moments less than 1, figure 4.6(a) also implies that if the responses of abutments are considered, the bending moment is always less than with fixed abutments. This is because with abutments responding to ground movements, the relative movement between the girder and the pier support becomes less because of abutment constraints. When abutments are not considered (apart from their gravity support role), the pier sway is

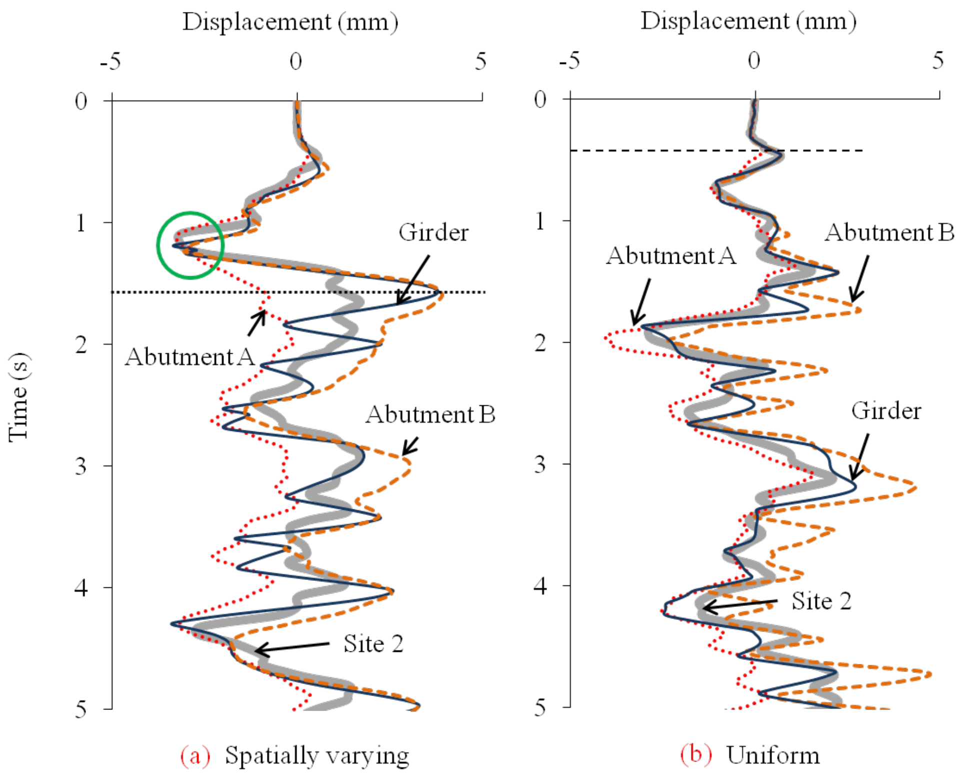
determined mainly by the structural characteristics in relation to the ground motion characteristics. In the case of movable abutments, although the bridge girder is not rigidly blocked as in the case of non-moving abutments, the pier sway is still impeded by the moving abutments. Consequently, the bending moments obtained without an abutment constraint are generally larger than when abutments are present.

As shown in figure 4.6(b), when the two abutments are fixed, the relative displacement between abutment and bridge segment is very small, equal to the compressive deformation of the pounding head. When the response of the abutments is considered, between the girder and the abutment A, the uniform ground motions result in a normalised average maximum opening relative displacement of 1.0 at 1.4Hz, corresponding to 4.11mm, which indicates there is still some significant out-of-phase movements despite the same ground motions being applied to all the shake tables. This is because of the different dynamic behaviour of the abutments from that of the relatively more flexible bridge structure. Another important insight obtained from figure 4.6(b) is that under spatially varying ground excitations the normalised opening relative displacements are 1.16 and 0.95 for the bridge structure with the fundamental frequencies of 1.4Hz and 2.6Hz, which correspond to 4.68mm and 4.08mm respectively. However, uniform excitations result in 4.11mm and 1.76mm opening relative displacement for a structure with the corresponding frequencies. Therefore, considering spatially varying ground motions leads to greater separation between the segment and the abutment (unseating potential of the bridge girder) than spatially uniform ground motions. Figure 4.6(c) shows that the spatial variation of ground motion also causes larger impact forces compared with uniform ground motions for all the cases considered. The average maximum pounding forces due to both uniform and spatially varying ground motions decrease with an increase in the bridge fundamental frequency. Further explanation of this observation is given in figure 4.8.

Another notable observation from figure 4.6 is that the consequence of assuming uniform ground motions is more serious (the difference between the green dotted line and the red dashed line is larger) as the bridge becomes stiffer for all the responses considered. The reason is that as the stiffness of the bridge gets closer to that of the abutments, the adverse effect of dissimilar fundamental frequencies on the structure response is reduced. Moreover, the reduction in bridge mass used to increase its fundamental frequency results in smaller pounding (refer to figure 4.8 for more details), mitigating the influence towards spatially varying motions. Consequently, for more flexible bridges with abutments, spatial variation of ground motions should always be considered. It should be noted that this conclusion only applies to the cases considered, since the fundamental frequencies can also be altered when the mass is constant while the bending stiffness of the piers varies.

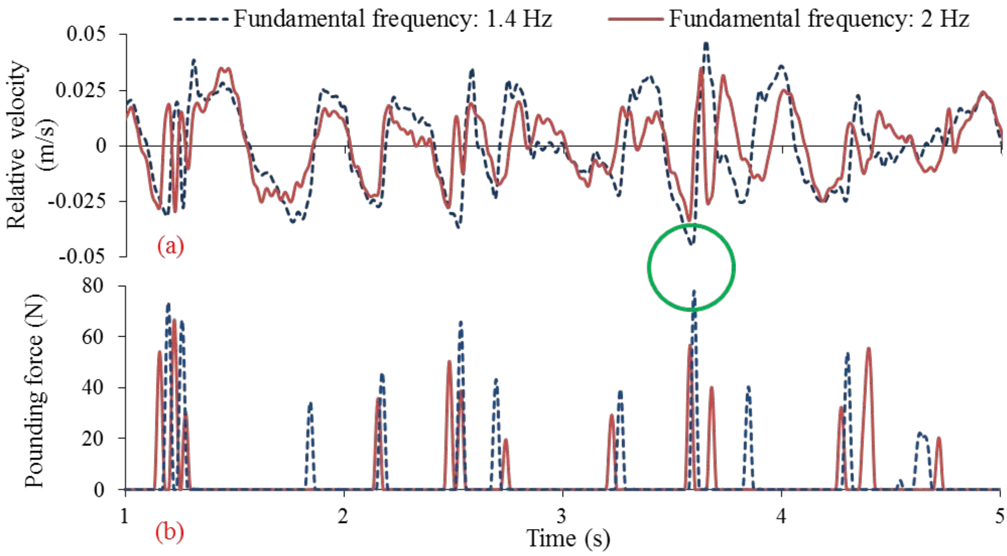
Figure 4.7 demonstrates the reasons for the large separations between the structures shown in figure 4.6(b). The displacements of the girder, the abutments and site 2 due to spatially varying and uniform ground motions for the bridge model with a fundamental frequency of 1.4Hz are presented. The crossover of the displacement curves of the abutments and the girder indicates the occurrence of pounding and the more the structure displacement time histories cross each other, the stronger the pounding (due to the small inward bending of the steel strip during contact). In figure 4.7a the opening relative displacement between abutment A and the girder reaches its maximum at 1.55s (indicated by the dotted horizontal line). This is due to the combined effect of the strong pounding occurring at 1.23s (circled time instant) and the ground motions of the structures. The bridge girder and abutment A separate immediately after pounding, slowing down the movement of abutment A (indicated by the slope of the dotted curve after the circled pounding instant). Meanwhile, the ground (the thick grey solid curve) which the bridge is based on is moving in the same direction as the girder (the thin dark blue solid curve), and hence provides acceleration to the girder. As a result, a large opening relative displacement of 4.68mm between the girder and abutment A is formed at 1.55s. Figure 4.7(b) explains the observation of the notable opening relative displacement between the structures despite uniform ground motion. Owing to the distinctly different dynamic characteristics of the abutment and bridge, pounding is inevitable. With zero gaps, initially all the structures move in phase for a short while under uniform ground motions. As abutment A moves to the right, it pushes the girder to move as the bridge support excitation (due to uniform ground motions). After 0.4s (indicated by the dashed horizontal line), a separation occurs between abutment A and the girder, initiated by the activated inertia of the bridge structure. At the instant of separation, the direction of abutment A motion is reversed as a rigid body (indicated by the slope change of the dotted curve) while the response of the bridge is heightened by its elastic response to the ground motion. It continues its movement in the previous direction, pushing the abutment B further to the right. Meanwhile, abutment A moves with the ground to the left. As a result, a gap develops and out-of-phase movement commences, initiating pounding which affects the development of subsequent relative movements. For the rest of figure 4.7(b) after 0.4s, the in-phase movement of the structures is no longer observed. However, the opening relative displacements between the girder and the abutments are less significant than those under spatially varying ground motions (compare with figure 4.7(a)).The maximum separation is 3.2mm in the case of uniform ground motions compared with 4.68mm resulting from spatially varying excitations. Therefore, assuming uniform ground motions will underestimate the unseating potential even for a short site distance of 50m.

Figure 4.7 Consequence of spatial variation of ground motions for relative abutment-girder displacements: (a) Spatially varying motions with high correlation and (b) uniform ground motions of soft soil site conditions including pounding effect with stiff contact stiffness for a model of 1.4Hz



By comparing the relative velocity and the pounding force time histories at abutment A girder end interface between the bridge structure with the fundamental frequencies of 1.4 and 2Hz, figures 4.8(a) and (b) illustrate why the bridge with lower fundamental frequency has larger pounding forces than the one with higher fundamental frequency (see also figure 4.6(c)). For the relative velocity time histories, positive values indicate that the abutment and the bridge structure are separating while negative values indicate that they are approaching. As can be seen at 3.6s (the circled instant), the maximum approach velocity just before pounding in the case of bridge with a fundamental frequency of 1.4Hz is 0.042m/s, while the bridge with a fundamental frequency of 2Hz moves at only 0.03m/s. The separation velocity in the case of the 1.4Hz bridge structure reaches 0.046m/s immediately after the pounding while for 2Hz, it moves away from abutment A at only 0.032m/s. The pounding force magnitude depends primarily on the relative approach velocity of the two impacting objects and their masses (assuming that the contact surface remains elastic after pounding) (Goldsmith 1960). Therefore, with larger structural mass and higher relative approach velocity, the 1.4Hz system results in larger impact forces than the 2Hz system. Similar observations can be found at 1.3s, 2.1s, 2.5s, 3.25s and 4.4s.

Figure 4.8 Relationship between: (a) activated relative velocity and (b) pounding force due to highly correlated soft soil excitation and movable abutments

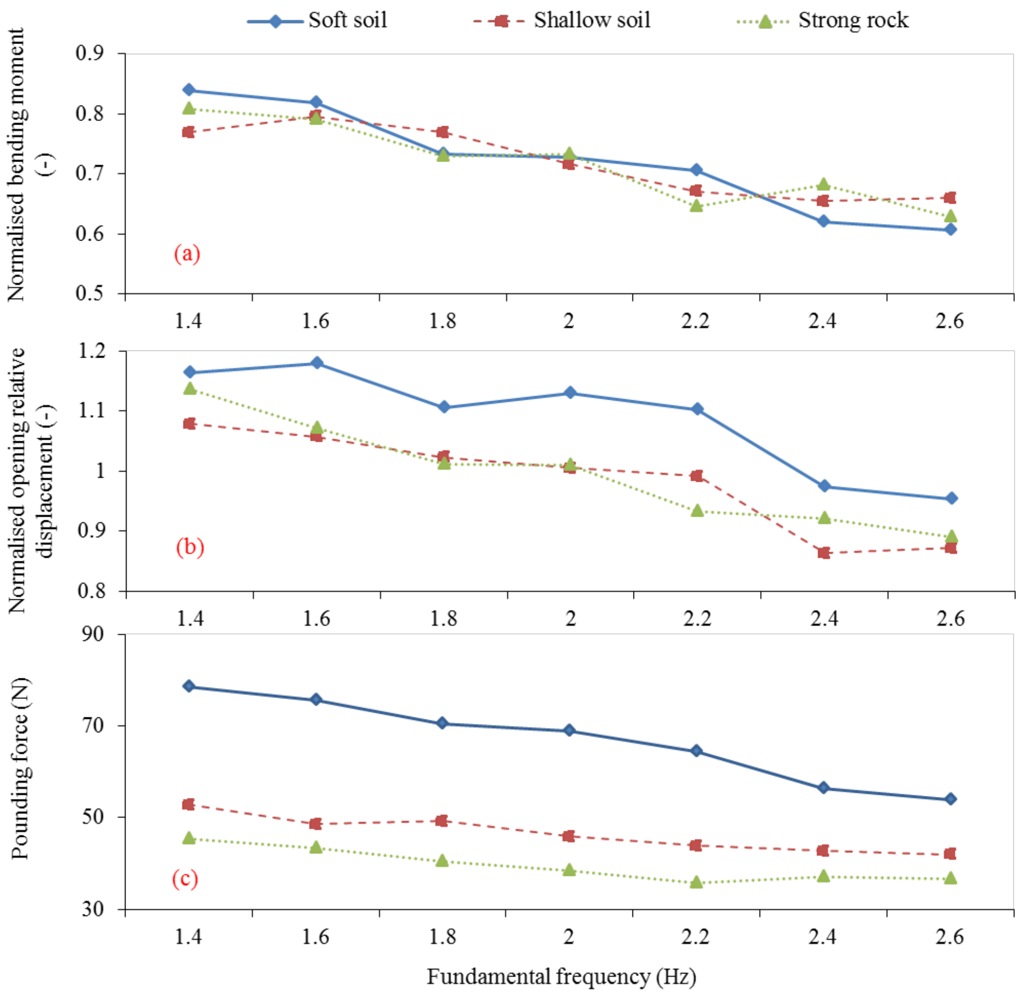


Designers should consider the mobility of abutments and spatial variation of ground motions when designing bridges for pounding. When designing for the bending moment, not considering pounding between a bridge segment and its abutment will usually lead to the most conservative design. It should be noted that the results presented are only valid for the same or similar conditions that this study considered. Since considering movable abutments under spatially varying ground motions is vital in bridge integrity design, the following discussion focuses on the effect of abutments engaging in spatially varying ground motions.

4.4.2 Effect of ground motion variation due to different soil conditions

Site soil properties can have a huge impact on the excitation that a bridge structure will experience during an earthquake. To investigate the bridge response due to ground motions of different site conditions, highly correlated spatially varying ground motions simulating soft soil, shallow soil and strong rock conditions are applied to the abutment-bridge system with zero initial gaps. Figure 4.9 shows the influence of different soil condition ground motions on structural responses normalised by the reference values given in table 4.2. A stiff contact stiffness and highly correlated ground motions are considered. Almost all the normalised responses decrease with an increase in the bridge fundamental frequency. The greatest normalised bending moment occurs at the fundamental frequency of 1.4Hz due to soft soil condition excitation, ie 0.839, corresponding to 670Nmm. For shallow soil and strong rock site conditions, the maximum normalised bending moments are found to be 0.770 and 0.809, corresponding to 427Nmm and 333Nmm, respectively. Soft soil condition also causes greater average maximum opening relative displacement and pounding force for all the systems considered. For the bridge with a fundamental frequency of 1.4Hz, the opening relative displacement and the pounding force resulting from soft soil excitation are respectively 4.72mm and 78.6N, approximately double those resulting from strong rock conditions, ie 2.3mm and 45.3N, respectively. When pounding is considered, soft soil site excitations generally lead to the largest structural responses over shallow soil and strong rock.

Figure 4.9 Consequence of different soil conditions for the average maximum normalised: (a) bending moment of pier A, (b) opening relative displacement, and (c) actual pounding force between bridge abutment A and girder

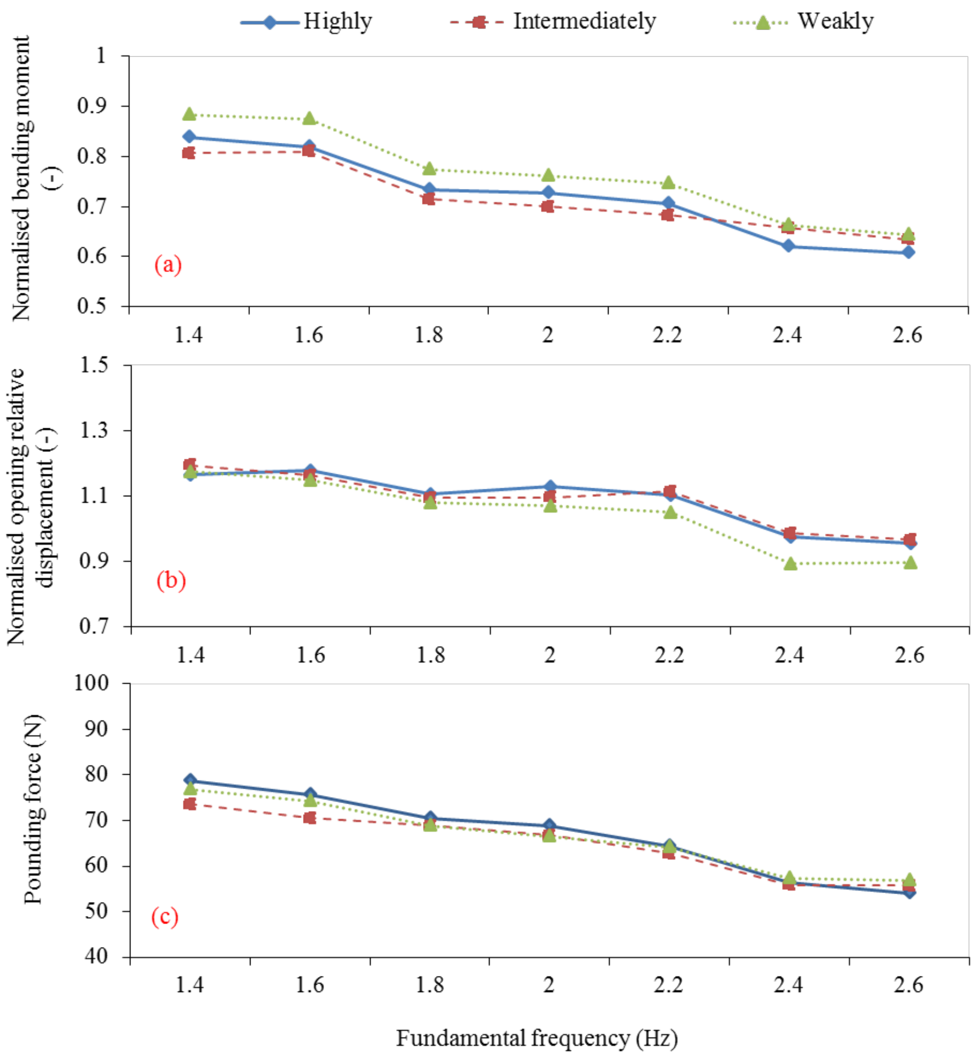


4.4.3 Effect of ground motion variation due to coherency loss

Figure 4.10 shows the influence of coherency loss on the response. The excitation considered is the soft soil condition ground motions and stiff contact stiffness at the abutments is assumed. From figure 4.10(a), ground motions with weak correlation result in a slightly larger normalised bending moment at the support of bridge pier A than intermediately and highly correlated excitations. However, the differences tend to decrease as the fundamental frequency of the model increases. The largest difference between the normalised bending moment among different correlations of ground motion occurs at 1.4Hz (between weakly correlated and intermediately correlated excitations), being only 10%. The differences in all the structural responses due to the excitation incoherency effect become less obvious, except in figure 4.10(b) for 2.2Hz, 2.4Hz and 2.6Hz, where the normalised maximum opening relative displacements between abutment A and the girder due to weakly correlated excitations are slightly smaller than those due to highly and intermediately correlated excitations. Figure 4.10(c) compares the pounding forces between abutment

A and the girder due to different coherency loss. The results show that the pounding forces resulting from different coherence of motions are very similar. Therefore, when the site distance is 50m or less, for simplicity the coherency loss effect of spatially varying excitation can be neglected and for a conservative consideration of pounding damage, highly correlated spatially varying ground motions can be assumed. However, further investigations are recommended, since the results obtained are valid only for the cases considered.

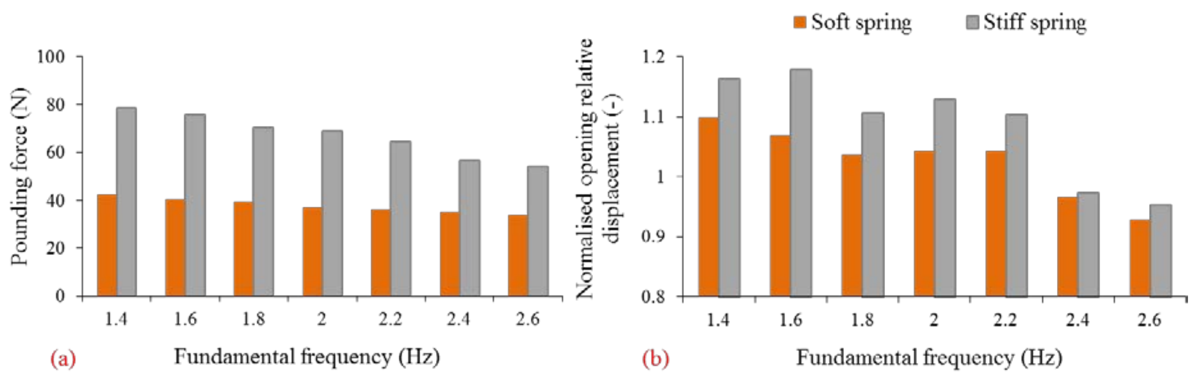
Figure 4.10 Consequence of different coherency loss of soft soil condition ground motions for the average maximum normalised: (a) bending moment of pier A, (b) opening relative displacement, and (c) actual pounding force between abutment A and the bridge girder



4.4.4 Effect of contact stiffness

In this research, different contact stiffness at the abutments-girder interface was considered by using two measuring heads with stiff and soft springs, respectively. The considered contact stiffnesses are given in table 4.1. Figure 4.11 illustrates the effect of the contact stiffness on structural response. The pounding force obtained with a stiff spring in figure 4.11(a) exceeds that resulting from soft spring by a significant amount for all the fundamental frequencies, especially for 1.4Hz and 1.6Hz, where the differences almost double. Larger normalised opening relative displacements are also obtained when stiff spring is considered as shown in figure 4.11(b).

Figure 4.11 Consequence of contact stiffness for the average maximum: (a) pounding force, and b) normalised opening relative displacement between the abutment A and the bridge girder under highly correlated excitations of soft soil condition



The reason for the observation in figure 4.11 can be found in figures 4.12(a) and (b), which show the development of the relative velocity and the pounding force within the time window between 4.2s and 5.7s due to soft and stiff springs. Figures 4.12(c) and (d) are zoomed-in plots of the portion surrounded by the dashed rectangle in figures 4.12(a) and (b) for a clearer presentation of the relations between the relative velocity and the pounding forces developed at the abutment A – girder interface. In the relative velocity time histories, positive values represent a separation velocity while negative values an approach velocity. The pounding commences when the negative relative velocity reaches its negative maximum due to contact of the structures (indicated by the dash-dotted line as shown in figures 4.12(c) and (d), and finishes when the relative velocity reaches its positive maximum. As the contact surfaces deform, kinetic energy is transferred into elastic strain energy. Soon after the pounding force reaches a maximum, the abutment and the girder move away from each other with an increasing relative velocity due to the reverse transfer of elastic to kinetic energy. By referring to the 'impulse-momentum law' (equation 4.1), which is valid for pounding with relatively short contact duration, the magnitude of pounding force depends upon the rate of momentum change. Since the girder mass is constant in the cases considered, the pounding force is now directly related to the rate of change of the relative velocity, which is indicated by the slope of the relative velocity curves.

$$\int F(t)dt = \Delta mV \qquad \text{(Equation 4.1)}$$

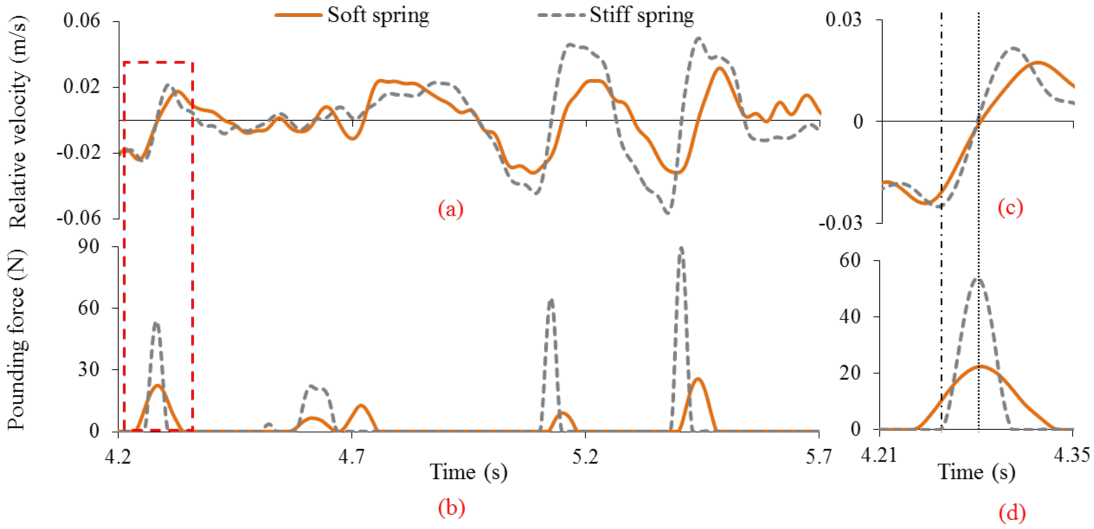
where $\int F(t)dt$ and V are the impulse and the relative velocity of the colliding bodies, respectively, and m is the effective body mass defined in equation 4.2:

$$m = \frac{m_1 + m_2}{m_1 m_2} \qquad \text{(Equation 4.2)}$$

where m_1, m_2 are the masses of the two impacting bodies, which in this study, are the masses of abutment A and the bridge girder, respectively.

Figure 4.12(c) shows that the slope of the relative velocity resulting from pounding with the stiff springs is steeper than that with the soft springs. Hence, a larger pounding force is observed in figure 4.12(d). Compared with the stiff spring, the soft spring is associated with longer contact duration, resulting in a smaller rate of change of the relative velocity. Therefore, larger contact stiffness can cause a more significant pounding force than that of smaller stiffness. As a result, minimising the contact stiffness can reduce the pounding force between an abutment and a bridge girder. A possible mitigation measure might be developed in the future by considering softer contact interface, eg by installing a rubber buffer, strong enough to carry heavy traffic loading and flexible enough to deform when the girder impacts.

Figure 4.12 Consequence of contact stiffness for the relative velocity and the pounding force at the abutment-girder interface due to highly correlated soft soil site excitation of the structure with the fundamental frequency of 1.4Hz between the time windows (a) and (b) of 4.2s and 5.7s; (c) and (d) of 4.21s and 4.35s



4.5 Summary

The effect of considering abutment excitation with spatial variation on the pounding responses of an abutment-segment-abutment system with an abutment-bridge site distance of 50m was experimentally investigated using three shake tables. The bridge model was closely scaled from the prototype structure based on the similitude laws. A bridge with various fundamental frequencies ranging from 1.4Hz to 2.6Hz with an increment of 0.2Hz was considered by varying the mass of the bridge deck. The abutment-bridge-abutment system was subjected to uniform and spatially varying ground motions based on the New Zealand design spectra. Simulations of ground motions with three soil conditions, ie soft soil, shallow soil and strong rock, defined in the target design spectra were achieved by using an improved ground motion simulation technique. Soft soil ground motions considering spatial variation effect were further categorised as having high, intermediate and weak coherency while for shallow soil and strong rock ground motions, only high coherency was considered. The effect of contact stiffness on the structural response was also investigated by using measuring heads with different spring stiffnesses. A total of 3080 pounding experiments and 420 non-pounding tests were conducted for this study.

The study revealed:

1 Considering movable abutments subjected to spatially varying ground motions generally leads to increases in opening relative displacement and pounding force between the girder and abutment over the cases of uniform ground motions and fixed abutment. Neglect of spatial variation of ground motions and abutment excitation will significantly underestimate the unseating and damage potential due to pounding between a girder end and adjacent abutment.

2 When abutment motion is considered, pounding will always occur even if uniform ground motions can reasonably be assumed.

3 In the cases with the same pier stiffness, bridges with lower fundamental frequency will suffer more damage from pounding than those with higher frequency. The more flexible the adjacent bridge structure is, the more the pounding and unseating potential will be underestimated.

4 The pounding responses of the system due to soft soil condition ground motions are more severe than those of shallow soil or strong rock conditions.

5 Pounding with movable abutments will generally reduce the support bending moment. Abutments with a stiff contact interface lead to greater pounding forces and relative displacements than a softer contact interface. Consideration of reducing the stiffness at the possible pounding locations, for instance, installing a rubber buffer, may be a cost-effective solution to mitigate the pounding effect in the future.

5 Field test of a 1:22 scale bridge

In order to investigate the effect of actual soil-bridge structure interaction on the relative response of adjacent bridge structure due to spatially varying ground excitations, a three-segment bridge was considered with its surrounding soil. A 1:22 scale bridge model was constructed and tested on the beach in Auckland, New Zealand. Fixed base and field tests were carried out to reveal the influence of SSI. The bridge was subjected to simulated excitations based on the New Zealand design spectra for soft, shallow and strong rock conditions (see section 3.1.4). The spatially varying excitations were generated with low, intermediate and high correlations using a coherency loss function.

5.1 Prototype structure and model

The bridge model was constructed according to the prototype structure described in section 3.1. A scale ratio of 22 was selected, resulting in a deck length of 4.55m and pier height of 0.705m. PVC was chosen as the construction material due to the same reason for the construction of the 1:125 scale models (see section 3.1.1 for more details). A time scale factor of 2 was adopted to ensure the excitations were strong enough to obtain measurable results. The scaling factors were chosen according to the method described in section 1.7 and these are listed in table 5.1.

Table 5.1 Scaling ratios for all significant physical quantities

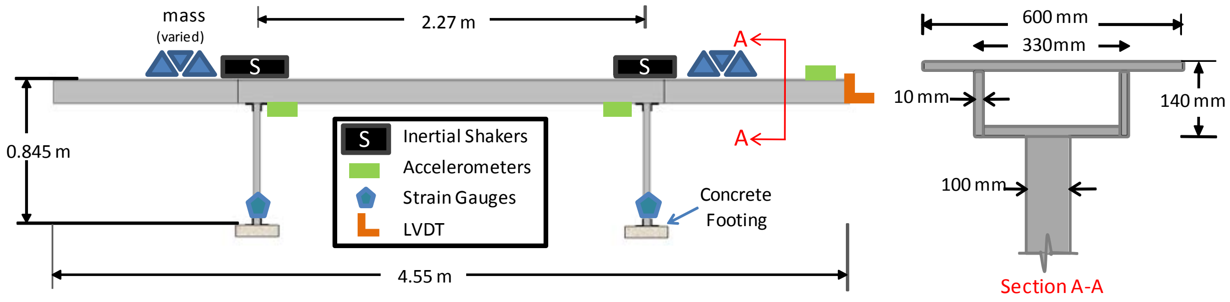
Physical quantity	Scaling ratio
Modulus of elasticity	12
Length	22
Time	2
Acceleration	5.5
Mass	5800
Force	32000

Table 5.2 Key properties, dimensions of the prototype and model bridge segments

Property	Prototype	Model
Material	Concrete	PVC
Segment length	100m	4.55m
Column height	15.5m	0.705m
Mass	1,895,405kg	326kg
Deck moment of area	$9.337\text{x}10^{12}\text{mm}^4$	$39{,}858{,}104\text{mm}^4$
Column moment of area	$3.891\text{x}10^{11}\text{mm}^4$	$287{,}424\text{mm}^4$
Stiffness	60,077,000N/m	48,865N/m
Fundamental frequency	0.98Hz	1.95Hz

The model columns were designed as 100mm by 36mm box sections with a 6mm wall thickness. The deck was a 330mm by 140mm box section with 10mm wall thickness. The top flange of the deck overhung the box by 135mm on each side as shown in figure 5.1. Full model and prototype properties are shown in table 5.2. The model also required footings for use in the field test. In both the fixed base and field tests, the bridge columns were bolted onto 340 x 340 x 70mm concrete footings. The concrete footing was designed to have a compressive strength of 30MPa.

Figure 5.1 Considered system



The three-segment bridge model was subjected to spatially varying ground motions on three separate sites. The absolute displacements obtained from each site excitation were then used to calculate the relative displacement between any adjacent segments. The opening and closing relative displacements were analysed to reveal the effect of spatially varying ground motion and SSI.

5.2 Equipment and testing

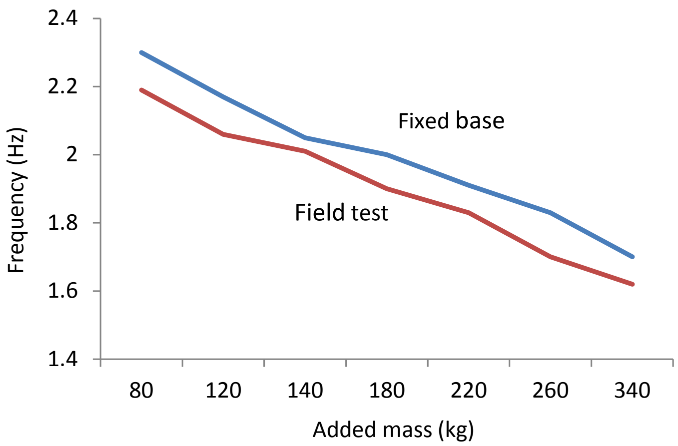
The test was conducted with fixed base and with subsoil and the columns were bolted to the concrete footings. The field test was conducted on Auckland's Kohimarama beach with the concrete footings resting on a compacted sand layer 200mm in depth. This ensured the depth of sand under the applied loads was compacted as uniformly as possible. The concrete footings were carefully placed on the sand bed to ensure they were level with the surrounding ground and the edges of the footings were enclosed by sand. The bi-axial levels of the footings and the bridge were checked to ensure absolute verticality in the longitudinal and transverse directions. The foundations are shown in figure 5.2.

Figure 5.2 Foundation conditions: (a) Fixed base and (b) field test foundation (concrete footing on sand)

In the field tests (see also figure 5.4), the soil was not scaled as the complexity of doing this was outside the scope of the research. The use of a soil box was not practical and feasible for the large size of the model bridge. The lack of soil scaling meant the field tests simulated a bridge and foundation on stiffer soil than was desirable for this test.

To consider the fundamental frequency of the different bridge structures, the model was provided with different mass blocks together with the mass of two inertia actuators to achieve a variety of fundamental frequencies between 1.7Hz and 2.3Hz. Because of the bridge scaling, the above measured frequencies corresponded to equivalent fundamental frequencies of 0.85Hz – 1.15Hz for the prototype. This range of frequencies is common for most large bridge structures. The natural frequencies were derived by conducting snap back tests. The resulting fundamental frequencies of fixed base and field test are depicted in figure 5.3 (the added mass does not include the mass of the inertia actuators).

Figure 5.3 Comparison of the fundamental frequencies of the model obtained from fixed base and field tests



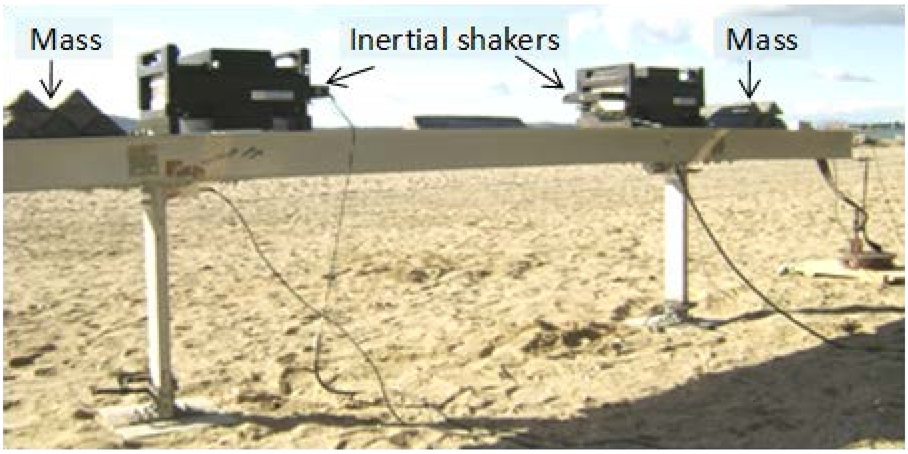
The model will behave as a SDOF systemdirect ground excitation, if:

- the bases of each pair of piers as shown in figure 5.1 are subject to the same ground motion
- the effective longitudinal structural mass is concentrated at deck level, equally at the top of each pier
- there is no longitudinal structural interaction with adjacent spans at either end of the girder
- half the equivalent inertia forces are applied simultaneously to the deck at the tops of each pier (this was due to the limited capacity of the inertial actuators).

The model was tested by mounting two identical Electro-Seis shakers (the same as used in chapters 3 and 4 but in the actuator mode) on the bridge model as shown in figure 5.1. Owing to the difficulties of exciting the soil bed, excitation was applied from the top of the structure above (slightly off) the column-deck joint with negative amplitude equal to the required ground acceleration. This kept the ground stationary while applying the true ground acceleration to the structural mass. The resulting inertia forces were applied by means of inertial force actuators.

Each actuator excited 18kg of attached mass and the inertia force produced by the applied motion excited the structure. A number of measuring devices, shown in figure 5.1, were used to record the responses of the model at a high sample rate (1000Hz) using various pieces of equipment as listed below for the duration of each of the excitations. The bridge deck displacements were recorded using a linear variable differential transformer. The accelerations of the columns and the bridge deck were recorded by attaching accelerometers to the top of each column and the bridge deck. Strain gauges were glued at the bottom of each of the columns to record the flexural strain experienced by the columns. This equipment setup is shown in figure 5.4.

Figure 5.4 Setup of the model in field experiments



The time history data was used to calculate the relative displacement between adjacent bridge girders. The recorded strain, displacement and acceleration data was processed and analysed using MATLAB programs. The same ground motions used for the study in chapter 3 but with a different scale of their magnitudes were applied to this model. The average maximum relative displacement between adjacent structures for each case was determined. The use of average maximum relative displacements minimised the effects of any bias in the response of the structure for any given set of excitations. The maximum relative displacements calculated can be used to identify the minimum total gap and seating length required for an MEJ system to avoid pounding and unseating.

5.3 Results and discussion

The study investigated three factors that influence the bridge response, ie the effect of fundamental frequency, SSI and spatially varying excitation. To address the effect of spatial variation of support excitations, different ground motions on different sites were applied to the model bridge. Different fundamental frequencies of the bridge were considered to evaluate the recommendation of most design regulations, which suggested the frequency of the adjacent structures should be equalised to avoid pounding. In total, 4200 tests were performed.

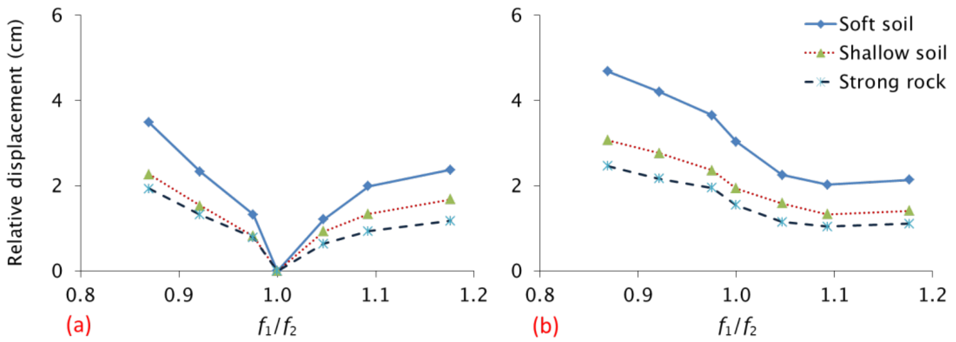
To encourage in-phase movement of adjacent segments, some bridge design manuals recommend that identical or similar fundamental frequency between neighbouring structures should be maintained. In this study, seven fundamental frequencies of the model were used to evaluate this recommendation under the effect of spatially varying ground motions. The frequency ratio is defined by the ratio of the fundamental frequency of the variable mass model (segment 1) to that of the reference model (segment 2) with an additional mass of 180kg, ie f_1 changes from 1.7Hz to 2.3Hz, and f_2 is 2.0Hz throughout section 5.3. The model is first loaded with uniform ground motions, followed by spatially varying ground motions with an assumption of a fixed-base foundation.

5.3.1 Effect of fundamental frequency ratios and spatial variation

Figure 5.5(a) shows the effect of fundamental frequency ratios on the average maximum opening relative displacement between segments 1 and 2 under a fixed-base foundation. Different soil conditions of uniform ground motion were considered. The relative displacements were very small when the frequency of segment 2 was the same as that of segment 1 (f_1/f_2 = 1). As the frequency ratio moved away from 1, there was an increase in relative displacements. Figure 5.5(a) also shows the effect of different soil condition ground motions on the response. Soft soil excitations led to the largest displacements, followed by excitations due to shallow and strong rock soils. This is in agreement with the soil spectra from NZS1170.5, as shown in figure 3.5.

When assuming uniform excitations, the observations discussed above prove that current design recommendations to ensure similar fundamental frequencies for adjacent structures indeed confirm no relative movement between the adjacent bridge structures and thus no pounding and unseating will take place. However, benefits brought by the recommendations diminish very quickly as the ratio moves away from 1. Design codes such as CALTRANS (2010) specify that the fundamental frequency of a flexible bridge frame should be more than 70% of that of an adjacent stiff bridge frame. Therefore when uniform excitation can be verified, this recommendation is surely correct. However, when spatially varying ground motions are anticipated, the advantage of having the same frequency no longer exists. Figure 5.5(b) reveals the opening relative displacement resulting from spatially varying ground motion with a fixed-base foundation. It shows that with consideration of spatial variation of the excitations, relative movement cannot be avoided despite the same fundamental frequency between the adjacent structures. The smallest opening relative displacement does not occur at the frequency ratio of 1. Another notable observation is that the soft soil still results in the largest separation while strong rock soil causes the smallest. It implies that soft soil normally leads to the largest relative response even under spatially varying ground motions. Bridge designers should consider a soft site condition to ensure a safer design of MEJs.

Figure 5.5 Consequence of frequency ratio and soil conditions on the average maximum opening relative displacement between segments 1 and 2 under (a) uniform and (b) spatially varying ground motions with fixed-base foundations

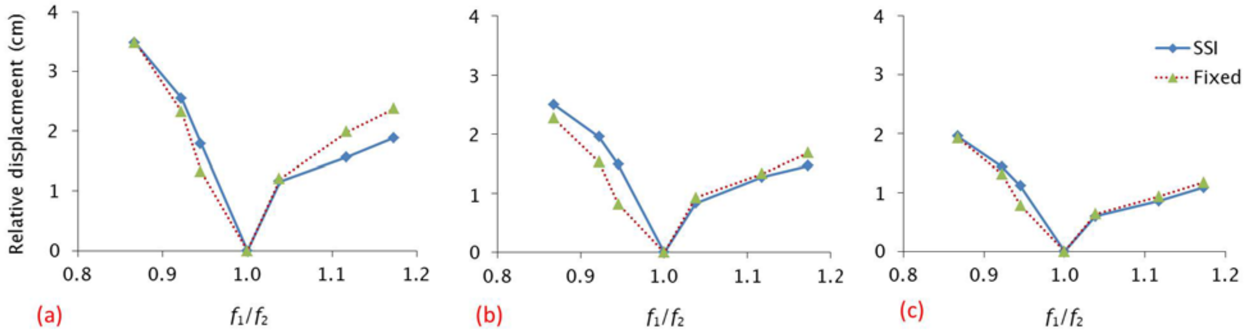


The opening relative displacements are derived between segments 1 and 2, where the fundamental frequency of segment 2 is kept as 2Hz while segment 1 has a variable frequency. In addition to the observation that excitations of soft soil condition cause the maximum opening movements, it is also found that if the adjacent segment is more flexible, the relative response between them will be more significant than if it is stiffer. This is evident from figure 5.5(b) where the separations between the segments with low frequency ratios are greater than those with high frequency ratios through all the soil types. However, a slight increase of the relative response is observed at the largest frequency ratio, ie 1.18 for all the soil conditions. This observation implies that the difference in the frequencies of adjacent segments is still influential although the first segment is stiffer than the reference segment. By comparing figures 5.5(a) and 5.5(b), for the frequency ratio larger than 1, the relative response between the considered segments is more significant for uniform ground motions than for spatially varying ground motions. Therefore, when designing bridges, designers should consider spatial variation of excitations.

5.3.2 Effects of spatially varying excitation and SSI

As shown in figure 5.6, the bridge response between segments 1 and 2 across all considered frequency ratios is compared in the case of uniform excitations with and without SSI. As the soil becomes stiffer, the opening relative response between structures decreases as expected. The peak relative response between the segments with a frequency ratio of 0.87 for a fixed-base foundation subject to excitation of a soft soil condition is 3.48cm and decreases to 1.93cm in the case of strong rock condition.

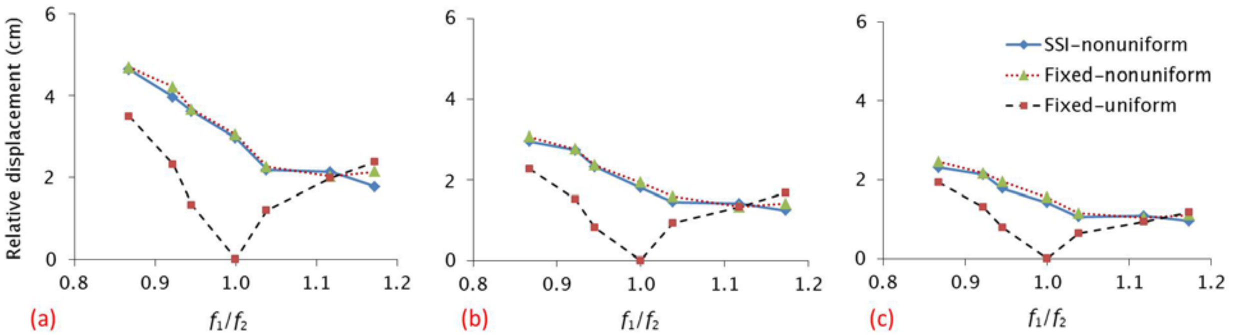
Figure 5.6 Consequence of frequency ratio and SSI on the opening relative movement between segments 1 and 2 subject to uniform excitations of (a) soft soil (b) shallow soil and (c) strong rock conditions



If the frequency ratio is larger than 1 for all soil conditions, SSI has a beneficial effect as it decreases the average maximum relative displacement between the structures when subject to uniform excitations. For frequency ratios between 1 and 1.08, SSI results in the same relative response as with a fixed foundation for all soil conditions. For frequency ratios between 1.08 and 1.20, SSI decreases the relative response, especially in the case of soft soils.

Figure 5.7 illustrates the combined effect of spatially varying ground motions and frequency discrepancy on the relative response between segments 1 and 2. The figures show that for all frequency ratios and all soil conditions, SSI gives a slightly lower relative response than a fixed base when subjected to spatially varying excitations. It also shows that for frequency ratios less than 1.13, uniform excitations with fixed foundations will significantly underestimate the gap required. As the frequency ratio increases, the gap required to avoid pounding when subjected to spatially varying excitations (with or without SSI) will generally decrease until the discrepancy in the fundamental frequencies is significant enough to cause the relative movement to be regained, which is what happened in this study when the frequency ratio exceeded around 1.13.

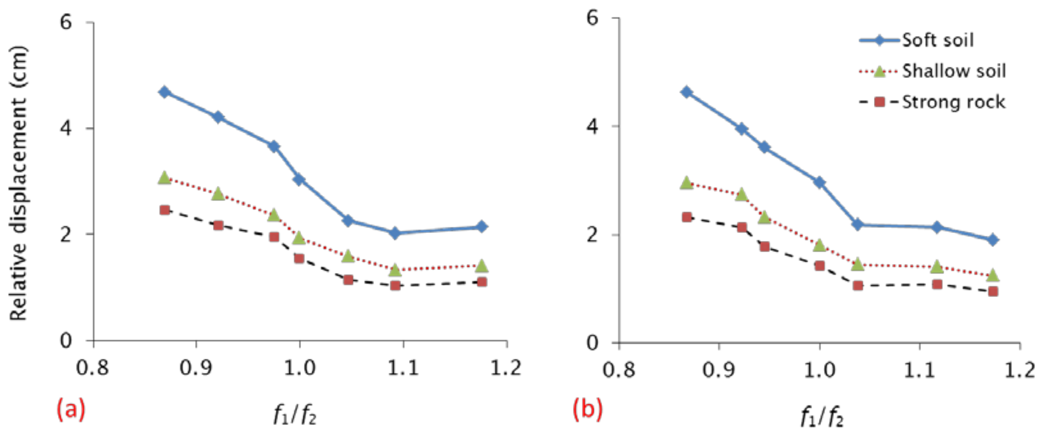
Figure 5.7 Consequence of frequency ratio, spatially varying excitation and SSI on the closing gap between segments 1 and 2 subject to highly correlated excitations of (a) soft soil (b) shallow soil and (c) strong rock conditions



For both uniform and spatially varying ground motions, the decrease in relative displacements due to SSI can be explained by the loss of rigidity at the base of a structure and an increase in system damping due to SSI. The decrease in the rigidity of the whole bridge structure system including subsoil leads to a smaller fundamental frequency of the system. The decrease in fundamental frequency leads to a decrease in the response (the dotted lines in figures 5.8 are below the solid lines) for soft soil, shallow soil and strong rock conditions as shown in the New Zealand design spectra (figure 3.5).

The responses of the model bridge across all the frequency ratios are compared with and without SSI when subjected to highly correlated spatially varying excitations of soft soil, shallow soil or strong rock conditions in figure 5.8. It shows the influence of excitations of different soil conditions coupled with the effect of SSI and how this affects the average maximum closing relative displacement between the girders. It can be observed that ground motions of soft soil conditions cause the largest closing relative movement between adjacent structures across all frequency ratios when subjected to spatially varying excitations. The peak relative response for fixed-base structures due to the excitation of soft soil condition is found to be 4.69cm and decreases to 2.41cm for the strong rock condition.

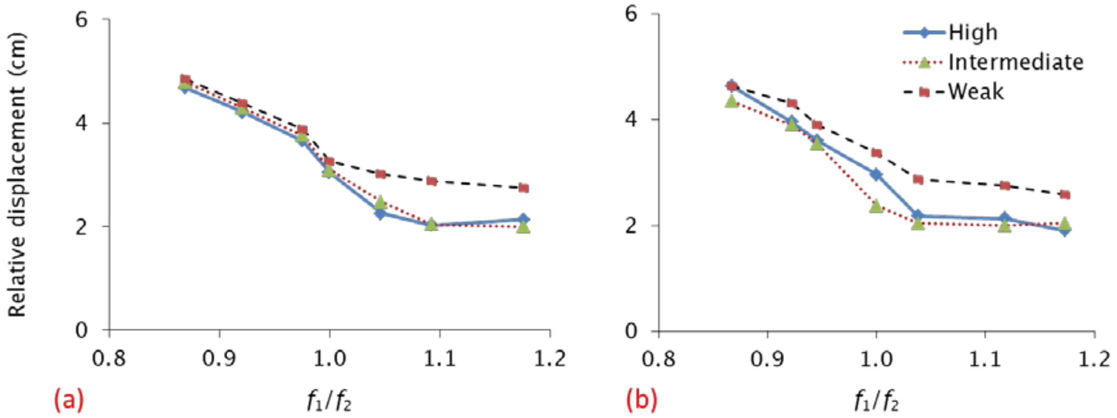
Figure 5.8 Consequence of different soil condition ground motions on the closing relative displacement between segments 1 and 2 subject to spatially varying excitations for (a) fixed base (b) field test



5.3.3 Effect of varying coherency of excitations

Figure 5.9 compares the influence of coherency loss on average maximum relative displacements with and without SSI. The results show that as expected with a decrease of coherency the relative response between adjacent structures increases. This is indicated by the relative response for weak correlation excitations being the highest across all frequency ratios considered. However, there is no clear relationship between the relative displacement and the coherency loss for the cases of high and intermediate coherency of ground motions as an opposite relation is observed between the magnitude of relative response due to high and intermediate correlation of ground motions for fixed base and field tests, even if the difference is not significant. In figure 5.9(a), for the frequency ratio up to 1, there is no noticeable difference between the responses caused by different coherency losses. The difference between the required closing gap resulting from weak correlation of ground motion and from other correlations becomes significant when the first segment is stiffer than its neighbour. Hence, in the design of a minimum required gap to avoid pounding, if spatially varying ground motion can be anticipated, weak correlation of the support excitations should be taken into account.

Figure 5.9 Consequence of coherency loss on total closing relative displacement between segments 1 and 2 subject to spatially varying excitations with (a) fixed base and (b) foundation on sand



5.4 Summary

In this study, a larger-scale bridge built from PVC was tested to investigate the effects of SSI, spatially varying excitations and different fundamental frequencies. While the fixed-base tests were performed in the laboratory with its foundations fixed on a strong floor, the influence of SSI was investigated by performing experiments on beach sand. A total of 4200 tests were performed. It is assumed that structures and subsoil remained linear during the experiments.

This study revealed:

1 The recommendation of most current design standards about equalising the fundamental frequencies of adjacent bridge structures to minimise girder pounding and unseating is only valid when the assumption of uniform ground motions can be justified. However, the benefits of this recommendation are no longer observed when spatial variation of excitation is inevitable. Therefore current design recommendations of adjacent structures having a similar fundamental frequency can underestimate the required gap at an expansion joint and give a false sense of safety.

2 In most cases, SSI has a beneficial effect on the bridge response when involving spatially varying ground motions. However, the advantage, according to this study, is not significant.

3 Spatially varying excitations almost always lead to a larger relative response from structures with a fixed base compared to the effect of uniform excitations. If considering the effect of SSI, the relative displacements due to spatially varying ground motions dominate over uniform ground motions up to a frequency ratio of 1.13.

4 Excitations of soft soil conditions always cause more significant relative movement between adjacent segments than those of shallow soil and strong rock conditions, regardless of how solidly fixed the foundations are.

5 The relative movement between bridge structures is most severe when subjected to low coherency spatially varying excitations.

6 Recommendations for the NZTA Bridge manual

This section reviews the current NZTA (2005) *Bridge manual* for the adequacy of predicting the minimum seating length required to avoid unseating of spans under strong earthquakes. The required seating length was obtained from the NZTA *Bridge manual* and Japan Road Association codes (JRA). In this study, spatially varying excitations were simulated corresponding to the New Zealand design spectra and modified with an empirical coherency loss function. An abutment-bridge-abutment system, a two-segment bridge system, a three-segment bridge system and a large scale single segment bridge were studied. The effects of pounding and SSI were also considered. The study evaluated the adequacy of the NZTA *Bridge manual* for suggesting the necessary seating length and clearance for bridge structures under spatially varying ground motions, especially when the adjacent bridge structure had a similar fundamental frequency.

6.1 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 AASHTO and the Japanese codes address this problem by suggesting sufficient seating length. Other bridge design codes, eg CALTRANS (2010) and NZTA (2005) do not have specific guidelines to address this problem even if the NZTA *Bridge manual* suggests the minimum seating length to account for out-of-phase response. An important reason for very limited implementation of spatially varying excitation is the highly complex combined effects between earthquake input, 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 (1999) bridge design specification prescribes a minimum seat length $S_{E,min}$ (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:

$$S_{E,\min} = (0.203 + 0.00167L_s + 0.00666H)(1 + 0.000125\alpha^2) \quad \text{(Equation 6.1)}$$

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

$$S_E = u_{rel} + u_G \geq S_{E,\min} \quad \text{(Equation 6.2)}$$

$$S_{E,\min} = 0.7 + 0.005l \quad \text{(Equation 6.3)}$$

$$u_G = \varepsilon_G L \quad \text{(Equation 6.4)}$$

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:

$$S_{E,\min} = l_m + d_{eg} + d_{es} \quad \text{(Equation 6.5)}$$

$$d_{eg} = \varepsilon_s L_{eff} \leq 2d_g \quad \text{(Equation 6.6)}$$

$$\varepsilon_s = \frac{2d_g}{L_g} \quad \text{(Equation 6.7)}$$

$$d_g = 0.025 a_g S T_C T_D \quad \text{(Equation 6.8)}$$

where l_m is the minimum support length securing the safe transmission of the vertical reaction $\geq$ 40 cm; 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 6.1, 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 6.1; 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 6.1 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 & 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* also specifies the provisions for the minimum seating length to prevent span loss. The provision for no linkage system is as follow:

$$S_{E,\min} = 2.0\ E + 0.1 \geq 0.4\,\text{m} \quad \text{(Equation 6.9)}$$

where 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 6.1, the AASHTO code suggests the minimum seating length by a function of only the span length and the pier height for a straight bridge. 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 study by Chouw and Hao (2006), it still underestimates 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 6.5.

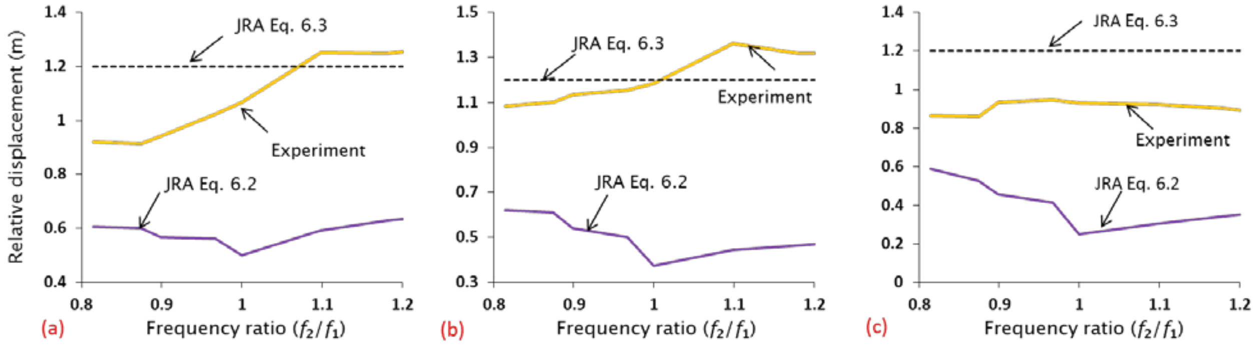
6.2 Methodology

The required seating lengths (overlaps) were calculated for the models presented in chapters 2 to 5 based on the suggestions from the NZTA *Bridge manual* and the Japan design specification, and the predicted values were compared with the results from the experiments of an abutment-bridge system, two-segment bridge structure, three-segment bridge structure and the large-scale model bridge. The experimental results are the average of the maximum opening relative displacements under 20 sets of stochastically independently simulated ground motions of the same characteristics based on New Zealand design spectra. Therefore, the results are relevant to New Zealand bridge design. The experimental results were back-scaled to the case of the prototype bridge in order to make suitable comparisons. From the comparisons, we were able to review the adequacy of the NZTA *Bridge manual* for suggesting minimum seating length when spatially varying ground motions are anticipated.

6.3 Evaluation of JRA standard

The Japan design specification is considered as one of the most advanced design standards in the world as it addresses the effect of spatially varying ground motions when considering minimum seating length for avoiding span unseating, although only empirically. However, its recommendations for these situations have proved to be inadequate. The predictions of necessary overlaps based on the JRA code are compared with the experimental results of the three-segment bridge non-pounding tests described in chapter 5. Segments 1 and 2 were considered in this evaluation. Figure 6.1 shows the results due to different soil conditions, where segment 2 (f_2 = 1.95 Hz) was kept constant while segment 1 was loaded with different masses to obtain a range of frequency ratios. According to figure 6.1, the JRA (2004) bridge manual underestimates the required seating length for excitations of soft and shallow soil conditions when the frequency ratio is small and the adjacent structure is relatively flexible. The results show that even the most advanced bridge code could underestimate the required seating length under spatially varying ground motions.

Figure 6.1 Comparisons between the recommended minimum seating length and the experimental maximum opening relative displacements between two segments due to highly correlated spatially varying ground motions of (a) soft soil, (b) shallow soil and (c) strong rock conditions according to New Zealand design spectra

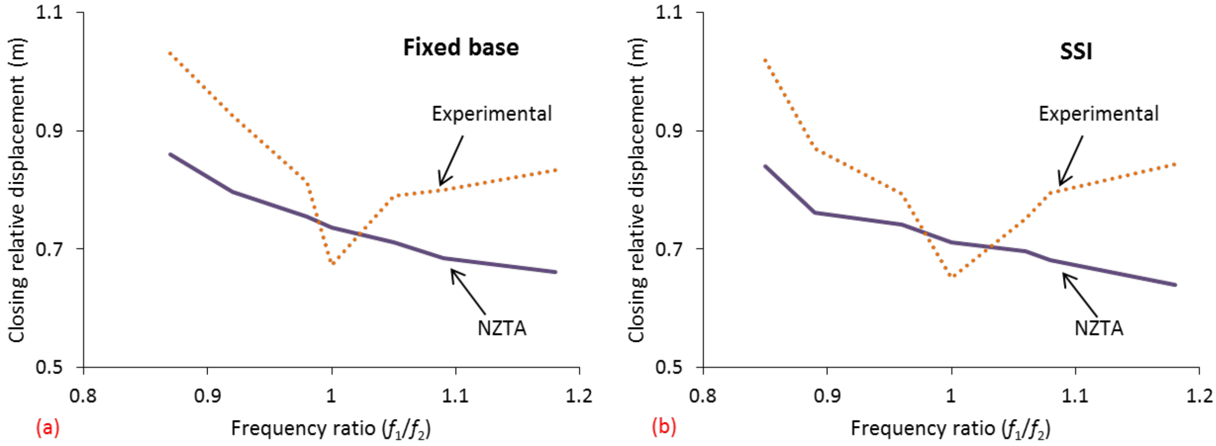


6.4 Evaluation of the NZTA Bridge manual

The NZTA (2005) *Bridge manual*, in clause 5.2.7(iii), states that contact between parts of the bridge shall not occur if the pounding can cause damage to the structure to the extent that persons would be endangered, or reduce the strength of structural elements below the required strength. In extreme cases, pounding is excluded to secure the bridge. To evaluate this provision, the closing relative displacements obtained from non-pounding testing of the 1:22 scale model with both fixed foundation and foundation on sand are compared with the recommended clearance, based on clause 5.6.1(a), calculated by SRSS of the recorded maximum displacements (clause 5.4.5(c)).

Figure 6.2 evaluates the NZTA code recommendation for the clearance that a deck joint must have in order to preclude pounding by comparing the values derived from clause 5.6.1(a) with the average maximum closing relative displacements for a two-segment bridge due to highly correlated excitations of soft soil condition with and without considering SSI effect. In these comparisons, the maximum relative displacements are obtained by considering a reference bridge with 180kg and the other with varying masses from 80kg to 340kg subjected to spatially varying ground motions. The frequency ratios are derived by dividing the natural frequency of the reference segment (f_2) by that of the varying segment (f_1). As shown in figure 6.2, almost for all the considered frequency ratios, the NZTA recommendations calculated based on the SRSS of the maximum displacements according to clause 5.6.1(a) of the manual still underestimate the closing relative displacements from the experiments except for a frequency ratio of 1. The maximum displacements are obtained from experiments described in section 5. The results show that the recommended gap by the NZTA manual may not be sufficient to preclude pounding. Even if pounding is not allowed by the NZTA *Bridge manual*, in a real-life practice, it is still likely to occur. Hence, in the discussion on the NZTA requirement for a span/support overlap, pounding should be included to avoid underestimation of the relative response.

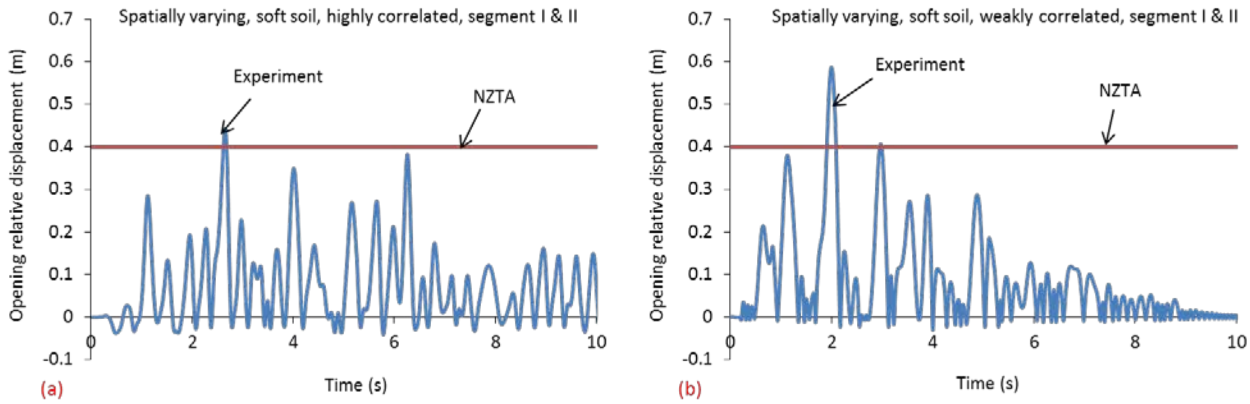
Figure 6.2 Comparison of the experimentally obtained average maximum closing relative displacements with the recommended clearances by the NZTA *Bridge manual* with (a) fixed base foundation and (b) foundation on sand under highly correlated excitation of soft soil condition.



Based on the experimental results, the conservatism of the approach proposed by the NZTA *Bridge manual* is illustrated through calculations of the seating length using equation 6.9. To enhance in-phase movement of adjacent structures, equal or similar fundamental frequency is recommended by many current design specifications, eg CALTRANS manual. In the current design regulation of the NZTA for bridges, uniform ground excitation is assumed. In this section, the bridge seismic performance is examined under the consideration of spatially varying ground motions to evaluate the current NZTA recommendation for the span/support overlap.

Figure 6.3 compares the measured opening relative displacements between identical segments due to different excitations (from chapter 3) with those according NZTA recommendation for the minimum required seating lengths. In the situation of two identical segments subject 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 6.9. All the figures 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 overlaps between bridge structures that experience earthquakes of soft soil condition regardless coherency and number of span involved. According to figure 6.3(d), the underestimation could be more than 50% for the case of a two-segment bridge subject to spatially highly correlated excitation of soft soil condition.

Figure 6.3 Minimum overlaps according to the NZTA recommendation and experimental results. (a)–(c) three-segment bridge and (d) two-segment bridge under different ground motions with pounding effect



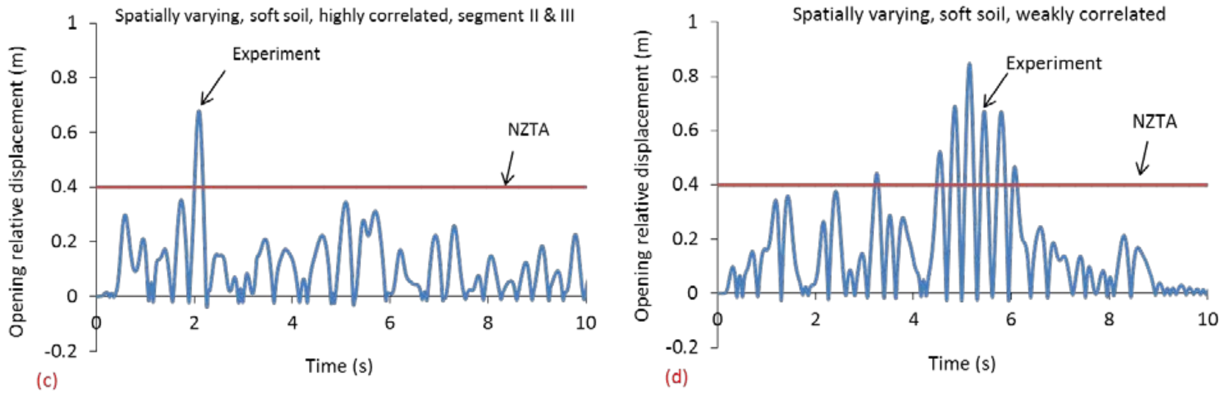


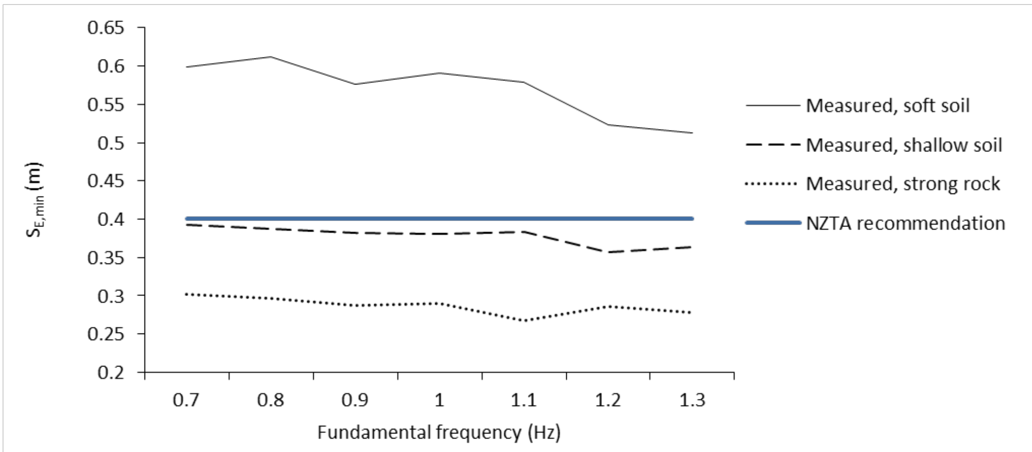
Table 6.2 Comparison between seating length suggested by the NZTA *Bridge manual* and the recorded average maximum relative displacements of two and three identical segments (from chapter 3) considering spatially varying and time-delayed ground motions of different soil conditions with pounding effect

Opening relative displacement (cm)	Soft soil (Class D)	Shallow soil (Class C)	Strong rock (Class A)
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 overlap from NZTA		40	

Table 6.2 shows that the NZTA *Bridge manual* underestimates the necessary seating length between adjacent spans in the situation of spatially varying excitation of soft soil condition. This is because the NZTA provisions assume structures to undergo uniform ground motions, which leads to zero relative displacement in the case of identical adjacent structures. By following equation 6.9, the minimum overlap is found to be 40cm. The assumption of uniform ground excitation along the bridge makes the current NZTA *Bridge manual* unable to provide sufficient seating length for adjacent spans, and thus amendment is required.

Figure 6.4 compares the estimate of the minimum span-abutment overlap from equation 6.9 and the recorded separations (figure 4.6(b) from chapter 4) with pounding. For this experiment, the abutments are excited by spatially varying ground motions, which is the case that causes the largest separations (more details are described in chapter 4). As shown in figure 6.4, the measured opening relative displacements due to excitations of soft soil condition are larger than the value according to the NZTA recommendation, which is 40cm. In the case of soft soil condition, the recommended minimum overlaps between the span and its abutment are only 70% of the seating length required.

Figure 6.4 Comparison between the suggested minimum abutment-girder overlap and measured opening relative displacement considering abutment movement and pounding following spatially varying ground motions with pounding



The minimum seating length of 40cm appears to be insufficient to accommodate the girder/abutment relative opening displacements due to excitations of the soft soil condition. This shows that when the bridge is funded on the soft soil, the underestimation of the minimum seating length by the NZTA *Bridge manual* is more than 25% for the bridges considered. The manual can, at most, underestimate the necessary seating length by 33% when the bridge has a fundamental frequency of 0.8Hz. Although the predicted minimum seating length is greater than the average maximum displacement of the girder with respect to the abutment support for the shallow soil condition, it is still possible that the NZTA *Bridge manual* may underestimate the seating length required as the measured relative displacement almost reaches the provided seating length for the bridge with a fundamental frequency of 0.7Hz.

6.5 Recommendations for the NZTA Bridge manual

This experimental and theoretical investigation into the effect of spatially varying ground motions on bridge structures concludes that superstructure displacements can be larger than the predicted seating length due to uniform ground motions especially when pounding occurs. The current NZTA *Bridge manual* provisions for minimum span overlap, bearing overlap and deck joint movement range may be insufficient in the case of spatially varying ground excitations.

Chapter 3 of this report demonstrates that numerical modelling of bridge behaviour to spatially varying ground motions accurately models experimental behaviour. It is concluded that this type of modelling is more holistic on predicting the expected displacements of bridge structures due to spatially varying ground motion. Modelling involves:

- developing a suite of acceleration time history records relevant to the site of the bridge
- determining the appropriate time shift and coherency loss to be applied to each record for each bridge support
- running the minimum number of time history analyses (typically three)
- determining the maximum displacements from the multiple runs.

It is recognised that this type of modelling is highly specialised and labour intensive and the investment may not be justified for low importance bridges. Indeed this analysis is irrelevant for integral abutment bridges, or bridges with tight linkage between spans and abutments. For low importance bridges a suitably conservative seating length and deck joint gap may suffice. Some recommendations in this regard are given later in this section. For high importance bridges, the investment in modelling spatially varying ground motions is in our opinion, justified. This recommendation is consistent with recent CALTRANS practice on long bridges such as the San Francisco Bay Bridge, the Coronado Bay Bridge in San Diego and the Vincent Thomas Bridge in the Port of Los Angeles. For the San Francisco Bay Bridge, spatially varying ground motions were used both in retrofit and new bridge design, and for the other two they were used in retrofit design.

According to section 6.4, the current NZTA *Bridge manual* may be at a risk of underestimating the relative response when spatially varying ground motions are involved. It is, therefore, necessary to propose a new approach to predict the possible seating length and gap needed to preclude pounding. Based on the experimental results from chapters 4 and 5, a detailed procedure is suggested to estimate the minimum span/support overlap and clearance for adjacent structures to preclude pounding. The formulae are based on results without the SSI effect due to the observation that the relative displacements between any adjacent spans with fixed foundations are, in most of the cases considered, greater than those with subsoil effect (see chapter 5 for details). This is the result of the reduced (as is often the case) bridge fundamental frequency including subsoil and foundation being on the ascent curve of the design spectra. Therefore, the proposed formulae are only valid for the case where the SSI effect is beneficial. If the fundamental frequency of the bridge span is somewhere on the descent curve of the spectra, the proposed approach may underestimate the required seating lengths or gap sizes. All the tests are pounding exclusive.

Figure 6.5 shows the differences between the overall maximum relative displacements (opening or closing, whichever is greater) resulting from uniform ground motions and spatially varying ground motions of soft soil conditions. The use of the overall maximum relative displacement ensures the overlap or the gap for the deck joint is still adequate when the earthquake comes from the opposite direction. In this comparison, segment 1 is varied with a different mass attached to achieve a range of fundamental frequency while segment 2 remains unchanged. As shown in this figure, spatially varying ground motions, as expected, result in larger relative displacement than uniform ground motions. The relative displacements due to uniform ground excitations decrease to zero when approaching a frequency ratio of '1'. Weakly correlated ground motions cause larger relative displacements than highly correlated excitations.

Figure 6.5 Comparison between the overall maximum relative displacements due to uniform ground motion and those due to spatially varying ground motions of different correlations

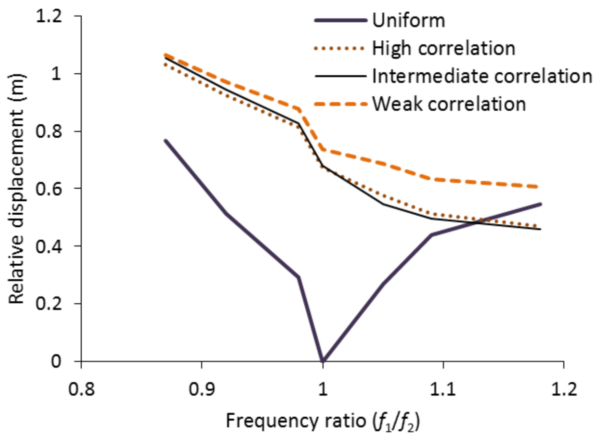
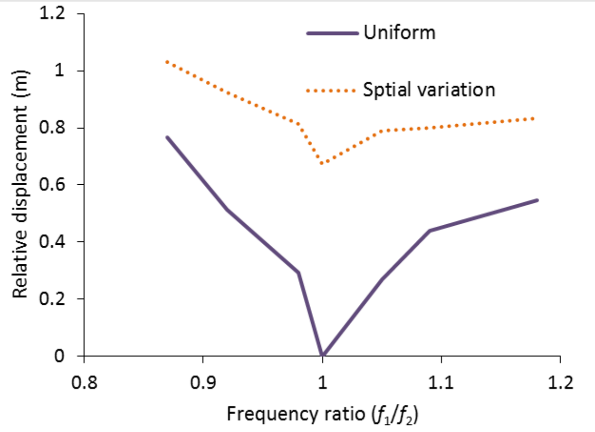


Figure 6.6 shows the overall maximum relative displacement of the two spans for each frequency ratio due to spatially varying ground motions and uniform ground motions of soft soil condition. In order to consider the most unfavourable configuration, ie when the seismic waves excite the more flexible structure first, the results up to and beyond the unit frequency ratio are obtained by reversing the direction of the incoming seismic waves. The difference between the relative displacements due to spatially varying ground motions (the dotted

line) and those due to uniform ground motions (thick solid line) reaches its maximum when the frequency ratio is 1. As the frequency ratio goes away from 1, the difference of the relative displacements can be deemed proportional to the corresponding frequency ratio. Their linear relation is attempted in figure 6.7.

Figure 6.6 Comparison between the overall maximum relative displacements resulting from assuming uniform ground motions and those due to highly correlated spatially varying ground motions of soft soil condition from the experiment



In figure 6.7, the frequency ratio is obtained by dividing the fundamental frequency of the more flexible segment f_1 by that of the reference segment f_2. As can be seen in this figure, a linear relationship is revealed between the frequency ratio and the difference of the largest absolute relative displacements resulting from assuming uniform ground motions, and those due to spatially varying ground motions. The similar linear relations are also observed in the cases of shallow soil and strong rock conditions (not presented in this report). According to figure 6.7, a model for calculating the gap or the overlap, $S_{g/o}$ that deck-to-deck joints must have, can be defined from the following equation:

$$S_{g/o} = a \cdot f \cdot L - b \cdot L + d_{uni} + 0.1 \quad \text{(in m)} \qquad \text{(Equation 6.10)}$$

where

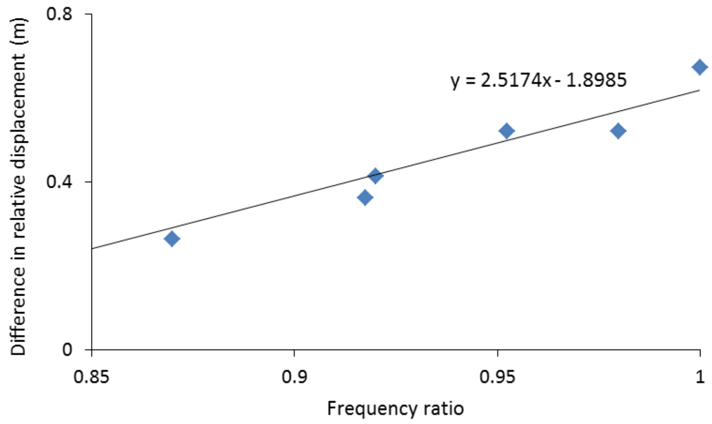
L (m) is the span of the longer bridge

d_{uni} (m) is the maximum relative displacement obtained from applying uniform ground motions

f is the fundamental frequency ratio defined by the flexible bridge segment over the stiffer bridge structure.

For soft, shallow and strong rock soil, a and b have the value of 0.026, 0.019 and 0.012, and 0.018, 0.013 and 0.008, respectively. To account for the effect of coherency loss, an additional length of 0.1m is provided. The proposed equation accounts for the influence of the frequency ratio of neighbouring structures, the effect of soil condition and the effect of spatially varying ground motions, although only empirically.

Figure 6.7 Relation between the difference of the relative displacement of two segments from the experimental results and those obtained from uniform ground motions and the corresponding frequency ratio due to excitation of soft soil condition with high correlation without pounding



The equation for the required gap or overlap for abutment/girder joints is also proposed. This approach involves derivations of the overall maximum relative displacement, d_{uni}, due to uniform ground motions considering the abutment movement with the ground as a rigid body and the ground relative displacement, d_{rg}, due to spatially varying ground motions. An equation for the overlap and clearance for abutment-girder joints is proposed as follow:

$$S_{o/g} = d_{uni} + d_{rg} \quad \text{(in m)} \qquad \text{(Equation 6.11)}$$

$$d_{rg} = \varepsilon_g d_b D_s \qquad \text{(Equation 6.12)}$$

where d_b (m) is the maximum displacement of the ground due to uniform ground motion

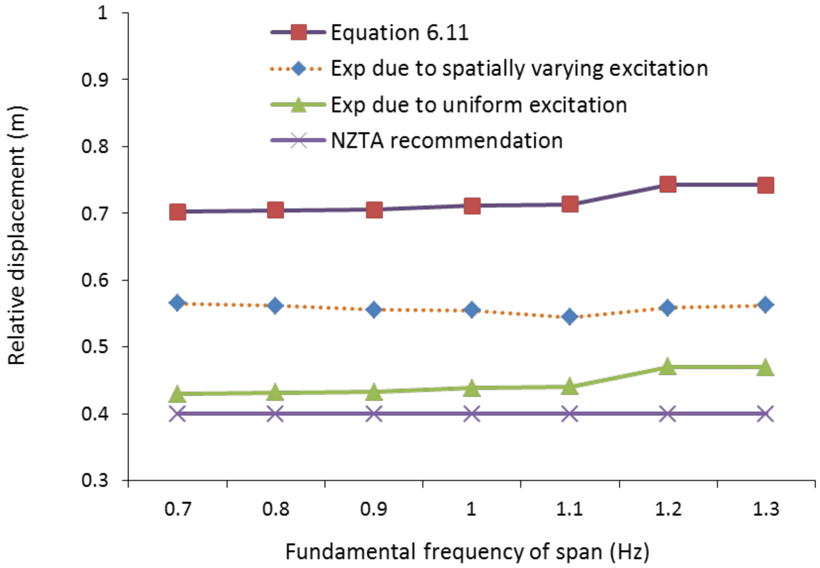
D_s (m) is the site distance between the abutment and the adjacent bridge segment

$\mathcal{E}_g$ has the value of 0.015 (m^{-1}) for all soil conditions.

This value is conservatively selected based on the New Zealand spectra, weakly correlated ground motions used in this study. $\mathcal{E}_g$ was determined as the maximum ratio of d_{rg} to the product of d_b and D_s. In this research D_s = 50 m and movable abutments are considered. The average values are obtained from the corresponding results due to the 20 sets of ground motion for cases as listed in table 4.1 (the third row and fourth column).

Figure 6.8 compares the average maximum relative response between the bridge and its abutment with the prediction from the proposed approach (equations 6.11 and 6.12) due to weakly correlated excitations of soft soil conditions. As shown in figure 6.8, the proposed equation provides sufficient seating length and gap size for an abutment/girder joint to accommodate the measured maximum relative displacement.

Figure 6.8 Comparison between the predicted overlap and clearance for abutment-deck joints and the actual maximum relative displacement obtained from the experiments considering weakly correlated excitations of soft soil condition without pounding



To derive the required gap or seating length for abutment/girder joints, designers should take the following steps:

- Develop earthquakes suitable for the site of the bridge.
- Obtain the maximum relative displacement by applying uniform ground motions to the abutment and its adjacent bridge structure from two opposite sides. Abutment movement due to ground motions should be considered.
- Compute the maximum displacement of the ground and the site distance for the abutment and the bridge support.
- Apply equations 6.11 and 6.12.
- Run three to seven simulated earthquakes (if possible) to obtain the maximum seating length or gap.

Since all the experiments were conducted using New Zealand design spectra, the proposed equations are suitable for New Zealand bridges. However, they are based on experimental results with particular conditions, eg an assumption of uniform soil-structure interaction and adjacent bridge structure with the same slenderness and structural members. Hence, it may be valid only for similar conditions and more verification is recommended.

The NZTA *Bridge manual* requires the designers to preclude pounding. Common practice for New Zealand bridges is to use knock-off nibs. In addition to that, MEJs are also a potential solution. MEJs can accommodate movements of up to 2m (Washington State Department of Transportation 2011). MEJs are suitable for bridges located in near-fault earthquake regions, where extremely large relative displacements are expected.

6.6 Summary

The provisions of the NZTA *Bridge manual* for estimating minimum overlaps and clearance were reviewed based on the physical results obtained from the previously described experiments. A total of 10,570 experiments were performed. The effect of spatial variation of ground motions was addressed. By comparing the measured average maximum opening relative displacement resulting from each study with the corresponding estimation based on the NZTA code, the following conclusions were made:

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, especially when adjacent segments have similar fundamental frequencies.
2. For high importance bridges, spatial variation of ground motions should be closely modelled and applied to the suitable numerical bridge model to derive the maximum relative displacements.
3. Although the NZTA code requires designers to preclude pounding by providing sufficient clearance between adjacent segments, the experiments revealed that the recommended clearance is likely to be insufficient to accommodate the possible closing relative displacement.
4. Spatial variation of ground motions is critical in determining relative displacement, and hence should be considered in the current NZTA *Bridge manual*.
5. Two sets of equations for predicting the total minimum gap and seating length are proposed based on both the experimental results and the New Zealand design spectra.
6. Modular expansion joints could be used as a solution to accommodate large opening relative displacement as well as to avoid pounding.

7 Conclusions

This research consisted of a number of shake table experiments on physical models of different scales, including tests of an abutment-bridge system, two-segment bridge, three-segment bridge and field test of a single segment bridge (shaker in inertial actuator mode). The objectives of this research were to understand the influences of the following factors: spatial variation of ground motions, dynamic characteristics of bridge structures, SSI and pounding. The combined effect of these factors as well as individual influences was investigated. For most of the studies, over 2000 tests were conducted. More than 4000 tests were performed for the field investigation of the 1:22 scale bridge. The results were also used to evaluate the current NZTA *Bridge manual* and make recommendations.

The experiments revealed that:

1 With pounding, spatially varying ground motions can increase the relative displacement at the inter-segment expansion joints and cause large pounding forces due to out-of-phase movement of the bridge girders. Neglect of spatial variation of ground motions will underestimate the damage potential between the girders due to pounding.

2 In all the considered cases, excitations of soft soil condition result in the greatest response over shallow and strong rock soil conditions.

3 In contrast to the recommendation of most of current bridge design specifications matching the frequency of adjacent segments is often inadequate to prevent or reduce out-of-phase movement when spatially varying ground motions are expected.

4 Pounding can contribute to girder unseating. However, further investigations including abutment effect are necessary to clarify the consequence of overall system performance.

5 Pounding with movable abutments will generally reduce the support bending moment. Abutments with a stiff interface stiffness lead to greater pounding forces due to a larger coefficient of restitution, than when there is a softer interface. Consideration of reducing the stiffness at the possible pounding locations, for instance, by installing a rubber buffer, may be a solution to mitigate the pounding effect in the future.

6 In general, the SSI effect can reduce the relative movement between adjacent bridge structures.

The conclusions and recommendations for NZTA *Bridge manual* are:

1 The current manual may not be sufficient for recommending the minimum seating length and the clearance at the girder joints or the joint for a span and its abutment if spatially varying ground motion is justified.

2 The recommended clearance by the current manual may still result in pounding even if pounding is not allowed in New Zealand bridge design.

3 With the influence of spatially varying excitation, having the same fundamental frequency for two segments does not necessarily suppress out-of-phase movement. Hence, spatial variation of ground motions should be addressed in the manual.

4 For high importance bridges, vigorous modelling of spatially varying ground motions is recommended.

5 Two equations have been proposed, based on the experimental results, for calculating the necessary gap or overlaps for bridge joint to exclude pounding and prevent unseating based on New Zealand design spectra.

6 Modular expansion joints could be used as a solution to accommodate large opening relative displacement while avoiding pounding.

8 References

American Association of State Highway and Transportation Officials (AASHTO) (1999) *Standard specifications for highway bridges*. Washington DC, USA.

Al-Homoud, A and R Whitman (1999) Seismic analysis and design of rigid bridge abutments considering rotation and sliding incorporating non-linear soil behavior. *Soil Dynamics and Earthquake Engineering 18*: 247--277.

Arias, JG and I Buckle (2010) *The M8.8 offshore Maule earthquake Chile seismic performance of transportation systems*. Accessed 25 September 2012. http://nees.unr.edu/outreach/eeri/Chile_EQ_UNR_Presentation.pdf.

Bi, K and H Hao (2011) Modelling and simulation of spatially varying earthquake ground motions at sites with varying conditions. *Probabilistic Engineering Mechanics***,** published online.

Bi, K, H Hao and N Chouw (2010) Required separation distance between decks and at abutments of a bridge crossing a canyon site to avoid seismic pounding. *Earthquake Engineering and Structural Dynamics 39*: 303–323.

Bi, K, H Hao and N Chouw (2011) Influence of ground motion spatial variation, site condition and SSI on the required separation distance of bridge structure to avoid seismic pounding. *Earthquake Engineering and Structural Dynamics 40*, no.9: 1027–1043.

Bolt, BA (1982) *Preliminary report on the SMART 1 strong motion array in Taiwan*. University of California at Berkeley, Earthquake Engineering Research Center.

BS EN 1998-2 (2005) *Eurocode 8 – design of structure for earthquake resistance. Part 2 – Seismic design of bridges*. London.

Buckingham, E (1914) *On physically similar systems: illustration of the use of dimensional equations*, Bureau of Standards, the American Physical Society.

California Department of Transportation (CALTRANS) (2010) *Seismic design criteria. Design manual – version 1.1*. Sacramento, California, USA.

Chau, K, X Wei, X Guo and C Shen (2003) Experimental and theoretical simulations of seismic poundings between two adjacent structures. *Earthquake Engineering and Structural Dynamics 32*: 537–554.

Chouw, N (2008) Unequal soil-structure interaction effect on seismic response of structure. *Proceedings of the 18th New Zealand Geotechnical Society Geotechnical Symposium on Soil-Structure Interaction – From rules of thumb to reality*, University of Auckland, 4–5 September 2008. pp214–219.

Chouw, N and H Hao (2005) Study of SSI and spatially varying ground motion effect on pounding between bridge girders. *Soil Dynamics and Earthquake Engineering 25*: 717–728.

Chouw, N and H Hao (2006) Comments on bridge girder seating length under current design regulations. *JSCE Journal of Applied Mechanics 9*: 691–699.

Chouw, N and H Hao (2008a) 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*: 75–85.

Chouw, N and H Hao (2008b) Significance of SSI and nonuniform near-fault ground motions in bridge response II: effect on response with modular expansion joint. *Engineering Structures 30*: 154–162.

Chouw, N and H Hao (2008c) Significance of SSI and nonuniform near-fault ground motions in bridge response I: Effect on response with conventional expansion joint. *Engineering Structures 30*: 141–153.

Chouw, N, H Hao and H Goldsworthy (2011) Some observations of damage in the 22nd February Christchurch earthquake. *Earthquake Reconnaissance Report.* Australian Earthquake Engineering Society. www.aees.org.au/News/110222_CHCH/Christchurch_report_May_2011.pdf.

Chouw, N and H Hao (2012, in print). Pounding damage to structures in the 2011 Christchurch earthquake, *International Journal of Protective Structures.*

Clyde, D and A Whittaker (2000) Full-scale dynamic testing of expansion joints for seismically isolated bridges, *Pacific Earthquake Engineering Research Center report no. EERCL/2000-01*, University of California at Berkeley.

Connor, RJ and RJ Dexter (1999) Fatigue design of modular bridge expansion joints. *Transportation Research Record: Journal of the Transportation Research Board 1688*: 124–130.

Crewe, AJ and JA Norman (2006) Experimental modelling of multiple support excitation of long span bridges. *Proceedings of the 4th International Conference on Earthquake Engineering Taipei, Taiwan. National Center for Research on Earthquake Engineering,* Paper 127.

Deodatis, G (1996) Non-stationary stochastic vector processes: seismic ground motion applications. *Probabilistic Engineering Mechanics 11*: 149–167.

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

Dumanogluid, A and K Soyluk (2003) A stochastic analysis of long span structures subjected to spatially varying ground motions including the site-response effect. *Engineering Structures 25*: 1301–1310.

Goldsmith, W (1960) *Impact:the theory and physical behaviour of colliding solids.* London UK, Edward Arnold.

Guo, A, Z Li, H Li and J Ou (2009) Experimental and analytical study on pounding reduction of base-isolated highway bridges using MR dampers. *Earthquake Engineering and Structural Dynamics 38*: 1307–1333.

Hall, JF (1994) *Northridge earthquake, January 17, 1994. Preliminary reconnaissance report.* Earthquake Engineering Research Institute.

Hao, H (1998) A parametric study of the required seating length for bridge decks during earthquake. *Earthquake Engineering and Structural Dynamics 27*: 91–103.

Hao, H and N Chouw (2007) Design of bridges with modular expansion joints for prevention of earthquake-induced girder poundings. *The Australian Earthquake Engineering Society.* Wollongong, Australia.

Hao, H and N Chouw (2008) Seismic design of bridges for prevention of girder pounding. *Electronic Journal of Structural Engineering,* EJSE special issue**,** 133–141.

Hao, H, C Oliveira and J Penzien (1989) Multiple-station ground motion processing and simulation based on SMART-1 array data. *Nuclear Engineering and Design 111*: 293–310.

Jankowski, R (2010) Experimental study on earthquake-induced pounding between structural elements made of different building materials. *Earthquake Engineering and Structural Dynamics 39*: 43–354.

Jankowski, R, K Wilde and Y Fujino (1998) Pounding of superstructure segments in isolated elevated bridge during earthquakes. *Earthquake Engineering and Structural Dynamics 27*: 487–502.

Johnson, N, RT Ranf, MS Saiidi, D Sanders and M Eberhard (2008). Seismic testing of a two-span reinforced concrete bridge. *Journal of Bridge Engineering 13*: 173–182.

Japan Road Association (JRA) (2004) *Specifications for highway bridges – part V seismic design*. 5th ed, 406pp (in Japanese).

Kim, SH, SW Lee, JH Won and H Mha (2000) Dynamic behaviors of bridges under seismic excitations with pounding between adjacent girders. *Proceedings of the 12th World Conference on Earthquake engineering,* Auckland, New Zealand.

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

Li, B, K Chan, N Chouw and JW Butterworth (2010). Seismic response of two-span scale bridge model due to spatially varying ground excitation and varying subsoil conditions. *Proceedings for 2010 NZSEE Conference*, Wellington, New Zealand.

Lin, C, H Hung, Y Liu and J Chai (2008) Reconnaissance report of 0512 China Wenchuan earthquake on bridges. *Proceedings of the 14th World Conference on Earthquake Engineering,* Beijing.

Malhotra, PK (1998) Dynamics of seismic pounding at expansion joints of concrete bridges. *Journal of Engineering Mechanics 124*: 794–802.

Maragakis, EA and PC Jennings (1987) Analytical models for the rigid body motions of skew bridges. *Earthquake Engineering and Structural Dynamics 15*: 923–944.

Moehle, J, G Fenves, R Mayes, N Priestley, F Seible, CM Uang, S Werner and M Aschheim (1995) Highway bridges and traffic management. *Earthquake Spectra 11:* 287–372.

Moehle, JP and MO Eberhard (eds) (2000) *Earthquake Damage to Bridges,* Boca Raton: CRC Press.

Moncarz, P and H Krawinkler (1981) Theory and application of experimental model analysis in earthquake engineering. *NASA STI/Recon Technical Report N 82*.

Mylonakis, G and G Gazetas (2000) Seismic soil-structure interaction: beneficial or detrimental? *Journal of Earthquake Engineering 4*: 277–301.

New Zealand Standard 1170.5 (2004) *Earthquake actions*. Wellington, New Zealand.

New Zealand Transport Agency (NZTA) (2005) *Bridge manual*. 2nd ed, amended 2005. Wellington: New Zealand.

Papadrakakis, M and HP Mouzakis (1995) Earthquake simulator testing of pounding between adjacent buildings. *Earthquake Engineering and Structural Dynamics 24*: 811–834.

Park, R, IJ Billings, GC Clifton, J Cousins, A Filiatrault, DN Jennings, LCP Jones, ND Perrin, SL Rooney and J Sinclair (1995) The Hyogo-ken Nanbu earthquake (the great Hanshin earthquake) of 17 January 1995. *Bulletin of the New Zealand National Society for Earthquake Engineering 28*: 1–98.

Raheem, SEA (2006) Seismic pounding between adjacent building structures. *Electronic Journal of Structural Engineering 6*: 66.

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

Rizza, MC (1973) *Modular expansion joint*. US patent application 3880540-A.

Saadeghvaziri, MA, A Yazdani-Motlagh and S Rashidi (2000) Effects of soil-structure interaction on longitudinal seismic response of MSSS bridges. *Soil Dynamics and Earthquake Engineering* 20: 231–242.

Spuler, T, E Ancich, P Savioz and C O'Suilleabhain (2011) Developments in modular expansion joint technology –codes and testing in Australia, America and Europe, *Proceedings of the 8th Austroads Bridge Conference,* Sydney.

Uzarski, J and C Arnold (2001) Chi-Chi, Taiwan, earthquake of September 21, 1999 reconnaissance report. *Earthquake Spectra 17*: 37–60.

van Mier, JGM, AF Pruijssers, HW Reinhardt and T Monnier (1991) Load-time response of colliding concrete bodies. *Journal of Structural Engineering 117*: 354–374.

Wang, CJ and MH Shih (2007) Performance study of a bridge involving sliding decks and pounded abutment during a violent earthquake. *Engineering Structures 29*: 802–812.

Washington State Department of Transportation (2011) *Bridge manual*.

Yang, Q, MS Saiidi, W Hang and A Itani (2002) Influence of earthquake ground motion incoherency on multi-support structures. *Earthquake Engineering and Engineering Vibration 1*: 167–180.

Youd, TL, JP Bardet and JD Bray (2000) Kocaeli Turkey earthquake of August 17. *1999 reconnaissance report.* Earthquake Engineering Research Institute.

Zanardo, G, H Hao and C Modena (2002) Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion. *Earthquake Engineering and Structural Dynamics 31*: 1325–1345.

Zerva, A (1990) Response of multi-span beams to spatially incoherent seismic ground motions. *Earthquake Engineering and Structural Dynamics 19*: 819–832.

Zhang, Y, Q Li, J Lin and F Williams (2009) Random vibration analysis of long-span structures subjected to spatially varying ground motions. *Soil Dynamics and Earthquake Engineering 29*: 620–629.

Zhu, P, M Abe and Y Fujino (2002) Modelling three-dimensional non-linear seismic performance of elevated bridges with emphasis on pounding of girders. *Earthquake Engineering and Structural Dynamics 31*: 1891–1913.

Appendix A: Disseminated research outputs

List of presentation and publications resulting from this project:

Presentation and publication at the Austroad Bridge Conference in 2009:

Chouw, N and H Hao (2009) Expansion joints for bridge structures under strong earthquakes. *Proceedings of the 7th Austroads Bridge Conference,* Auckland, 26–29 May 2009.

Presentation and publication at the New Zealand Earthquake Engineering Conference in 2010:

Li, B, K Chan, N Chouw and JW Butterworth (2010) Seismic response of two-span scale bridge model due to spatially varying ground excitation and varying subsoil conditions. *Proceedings of the Annual New Zealand Earthquake Engineering Society Conference,* Wellington. Paper 33, 26–28 March 2010.

Li, B, M Jamil, N Chouw and JW Butterworth (2010) Model bridge shake-table tests with soil-structure interaction, spatially varying ground motion and pounding. *Proceedings of the Annual New Zealand Earthquake Engineering Society Conference, Wellington.* Paper 62, 26–28 March 2010.

Presentation and publication at the international conferences in 2010 and 2011:

Chouw, N and H Hao (2010) Mitigation of bridge girder movement effect during strong earthquakes. *Proceedings of the 14th European Conference on Earthquake Engineering,* Ohrid, Macedonia. Paper 351, 30 August–3 September 2010.

Li, B, N Chouw and JW Butterworth (2010) Simulation of a two-span bridge under spatially varying ground excitations. *Proceedings of the 14th European Conference on Earthquake Engineering,* Ohrid, Macedonia. Paper 497, 30 August–3 September 2010.

Li, B, N Chouw and JW Butterworth (2010) Experimental investigation of the influence factors on the seismic response of adjacent bridge girders. *Proceedings of the 11th International Symposium on Structural Engineering,* Guangzhou, China. Paper 70, 18–20 December 2010.

Sun H, B Li B, K Bi, N Chouw, JW Butterworth and H Hao (2011) Shake table test of a three-span bridge model. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Society,* Auckland, New Zealand. Paper 149, 14–16 April 2011.

Manson, SE, OH Woodhouse, B Li, N Chouw and JW Butterworth (2011) Field dynamic testing of a large scale bridge using shakers. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Society,* Auckland, New Zealand, Paper 154, 14–16 April 2011.

Lindsay, B, B Li, N Chouw and JW Butterworth (2011) Seismic pounding of bridge superstructures at expansion joints. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Society,* Auckland, New Zealand, Paper 115, 14–16 April 2011.

Behrens, E and N Chouw (2011) Nonlinear SSI effect on adjacent bridge structures with pounding. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Society,* Auckland, New Zealand, Paper 219, 14–16 April 2011.

Chouw, N and E Behrens (2011) Numerical simulation of adjacent bridge structures with nonlinear SFSI. In M. Papadrakakis, M. Fragiadakis, V. Plevris (eds) *Proceedings of the 3rd Conference on Computational Methods in Structural Dynamics and Earthquake Engineering.* Corfu, Greece, 25–28 May 2011.

Publication in the New Zealand Society for Earthquake Engineering Bulletin:

Chouw, N and H Hao (2009) Seismic design of bridge structures with allowance for large relative girder movements to avoid pounding. *Bulletin of the New Zealand National Society for Earthquake Engineering*, *42*, no.2: 75–85.

Publication in an international journal:

Bi, KM, H Hao and N Chouw (2010) Required separation distance between decks and at abutments of a bridge crossing a canyon site to avoid seismic pounding. *Earthquake Engineering and Structural Dynamics 39*: 303–323.

Bi, KM, H Hao and N Chouw (2011) Influence of ground motion spatial variation, site condition and SSI on the required separation distance of bridge structure to avoid seismic pounding. *Earthquake Engineering and Structural Dynamics 40*, no.9: 1027–1043.

Li, B, KM Bi, N Chouw, JW Butterworth and H Hao (2012) Experimental investigation of spatially varying effect of ground motions on bridge pounding. *Earthquake Engineering and Structural Dynamics 41,* no.14*:* 1959–1976.