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Effects of spatially varying ground motions on bridge response

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A thesis submitted in fulfilment of the requirements for
the Degree of Doctor of Philosophy

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Abstract

Adjacent bridge structures can move relatively to each other in an earthquake, resulting in pounding or more severely, unseating. Pounding occurs when the relative closing movement is larger than the structural gap whereas girder unseating takes place due to the relative opening movements being larger than the seating length provided. Relative displacements arise from unequal fundamental frequencies of adjacent bridge structures, spatial variation of ground motions and different soil-structure interaction (SSI). The objective of this thesis is to investigate the influence of these factors, especially spatial variation of ground motions on the bridge response.

To achieve the objective, a series of experiments were conducted on the bridge models made of polyvinylchloride (PVC) using either shake tables or inertial actuators. Ground motions of the soft soil, shallow soil and strong rock conditions based on the New Zealand design spectra were simulated. The ground motions of the soft soil condition were further classified as highly, intermediately and weakly correlated ground motions to account for coherency loss effect of excitation spatial variation. These experiments include testing a 1:125 scale bridge with three identical bridge segments to study the effect of spatially varying ground motions with pounding, testing the same model but with the footings on sand contained by rubber boxes to consider the effects of spatially varying ground motions and SSI with pounding, testing one of those bridge segments with movable abutments to investigate the effect of excitation spatial variation considering abutment excitation and pounding, testing a 1:125 scale bridge model with two identical bridge segments with artificial plastic hinges to study the effect of spatially varying ground motions on inelastic bridge response with pounding, and field testing a 1:22 scale bridge segment subjected to spatially varying ground motions to determine the minimum total gap of a modulus expansion joint required to avoid pounding. A total of 8660 tests were performed.

Research found that spatial variation of ground motions can increase the relative displacement of adjacent bridge girders and pounding forces. Based on the experimental results, the New Zealand Transport Agency (NZTA) Bridge manual was reviewed. It was found that the current NZTA Bridge manual is unable to suggest sufficient minimum seating length to accommodate the measured relative opening displacements. A set of
empirical equations is therefore proposed to calculate adequate minimum support seating lengths to prevent girder unseating.
Dedications

To my parents with love
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Symbols and notations

\[ \int F(t)dt \quad \text{Impulse of the colliding bodies} \]

\( d_{ij} \quad \text{Distance between the two locations } i \text{ and } j \)

\( \gamma_{ij} \quad \text{Ground motion due to coherency loss from locations } i \text{ to } j \)

\( \Delta t \quad \text{Difference in the arrival time of the waves to the accelerometers} \)

\( \Delta u \quad \text{Maximum average normalised relative displacement} \)

AASHTO American Association of State Highway and Transportation Officials

\( a_g \quad \text{Ground acceleration} \)

\( c_a \quad \text{Apparent wave velocity} \)

CALTRANS California Department of Transportation

\( d_{ave} \quad \text{Mean of the maximum displacements of the two girders with fixed-base foundations subjected to the uniform ground motions} \)

\( d_{eg} \quad \text{Effective relative displacement of the span and the abutment due to differential seismic ground displacement} \)

\( d_{es} \quad \text{Effective seismic displacement of the support due to the deformation of the structure} \)

\( d_g \quad \text{Design value of the peak ground displacement} \)

\( d_{sv} \quad \text{Maximum relative girder-to-abutment displacements due to spatially varying ground motions} \)

\( d_{uni} \quad \text{Maximum relative displacement obtained from applying uniform ground motions with fixed-base foundations} \)

\( dx \quad \text{Distance between the two accelerometers} \)

\( E \quad \text{Relative movement between span and support} \)
EC8-2  Eurocode8-2

$E_g$  Constant for different soil categories

$f$  Frequency

$F$  Maximum average pounding force

$H$  Pier height in metres

$I_{\text{deck}}$  The second moment of area of the bridge deck

$I_{\text{pier}}$  The second moment of area of the bridge pier

JRA  Japan Road Association

$K_{\text{bending}}$  bending stiffness of the bridge structure

$L$  Distance between two substructures

$l$  Length of the effective span

$L_{\text{eff}}$  Effective deck length

$L_g$  Distance beyond which ground motions may be considered uncorrelated

$l_m$  Minimum support length securing the safe transmission of the vertical reaction $\geq 40$ cm

$L_s$  Span length in metres

LVDT  Linear variable differential transformer

$M$  Maximum average normalised bending moment at pier base

$m_1, m_2$  Masses of the two impacting bodies

MEJ  Modular expansion joint

$N$  Scale factor

NZS  New Zealand Standard

NZTA  New Zealand Transport Agency
\(P\)  Maximum average normalised pounding force

PVC  Polyvinylchloride

\(S\)  Seating length

SDC  Seismic design categories

\(S_F\)  Soil factor

\(SL\)  Minimum seating length in metres

SMART-1  Strong Motion Array in Taiwan

SSI  Soil-structure interaction

\(T\)  Period

\(u_G\)  Relative displacement of the ground occurring due to ground deformation between piers

\(u_g\)  Ground displacement

\(u_{rel}\)  Maximum average relative displacement

\(V\)  Relative velocity of the colliding bodies

\(v_{rg}\)  Phase velocity of Rayleigh wave

\(v_s\)  Shear wave velocity

WSDOT  Washington State Department of Transportation

\(\beta, a, b, c\)  Constant parameters for coherency loss functions

\(\nu\)  Poisson ratio of the soil
Chapter 1

Introduction

1.1 Motivation and scope

Pounding damage to bridges has been observed in major earthquakes. Pounding and unseating are caused by the relative response of bridge exceeding the gap and seating length. One reason is that uniform ground motions are often assumed in bridge design. In reality, ground motions are spatially non-uniform along bridge supports. This assumption of uniform ground excitation often leads to underestimation of the minimum seating length required. In addition to this reason, soil-structure interaction (SSI) could result in larger bridge response. Neglecting SSI may underestimate the relative response between bridge structures. Pounding is also found responsible for girder unseating (Abdel Raheem, 2009). Although the effects of spatially varying ground motions have been investigated, especially after the Strong Motion Array in Taiwan (SMART-1 array) (Bolt, et al., 1982) was established, the studies are mainly numerical. Only a very limited number of experimental investigations has been reported, and most of them did not consider pounding or SSI. The objective of this research is to contribute to closing part of this knowledge gap by experimentally investigating the effect of the spatially varying ground on bridge response including pounding and SSI.

Moreover, the conclusions of some research outcomes may contradict to each other. It is argued that whether pounding can increase or decrease bridge response, e.g. bending moment at the pier support and relative displacement between adjacent bridge decks. This research is to validate and clarify these existing contradicting arguments.

The bridge specification of Japan Road Association (JRA) is regarded as one of the most advanced bridge design specifications in the world. Chouw and Hao (2006) found that JRA can still underestimate the necessary seating length to prevent girder unseating, when spatial variation of ground motions has been taken into account. The current New
Zealand Transport Agency (NZTA) Bridge manual (2005) assumes uniform ground motions. It may also underestimate the minimum seating length. By performing laboratory and field tests in this PhD research, the relative displacements of adjacent bridge structures are investigated considering spatially varying ground motions and SSI with and without pounding. The measured relative displacements are compared with the seating lengths recommended by the NZTA Bridge manual. The ground motions used are simulated based on the New Zealand design spectra (NZS 1170.5, 2004).

1.2 Methodology

To investigate the influence of the spatial variation of ground motions on bridge response, a 1:125 scale bridge segments with almost identical properties were constructed based on one of the Newmarket Viaduct replacement bridge segments. The similitude scaling rules (Moncarz and Krawinkler, 1981) were followed. Based on these bridge segments, the effect of two-sided pounding and spatially varying ground motions was investigated. The spatially varying ground motions were applied using three shake tables. Pounding was allowed by placing the adjacent segments with zero gaps. This experiment was conducted with fixed base. Another experiment studied the effect of two-sided pounding and spatially varying ground motions with SSI by testing those three segments with sandboxes made of soft rubber. The effect of abutment excitation accounting for spatial variation was also studied by using one of those segments and two steel frames. The abutments were assumed infinitely stiff and to move with the ground. Different fundamental frequencies of the segment were achieved by altering the mass attached to the model. The gaps between the girder and the abutments were almost zero. The influence of pounding stiffness was investigated by making the contact surface of the abutments with two different spring stiffness. To investigate possible plastic hinge development, artificial plastic hinges were constructed at each pier base and at each connection between the pier and the girder. A field test was carried out on a 1:22 scale bridge, which was developed based on the same prototype as for the 1:125 scale model. The purpose of this field test was to determine the minimum total gaps for modular expansion joints (MEJ) to avoid pounding. SSI effect was studied by comparing the maximum relative displacements with those of fixed-base condition conducted in the
laboratory. Bridge models with different fundamental frequencies ranging from 1.7 Hz to 2.3 Hz were tested by altering the mass on the model.

Twenty excitations were simulated for three soil conditions, i.e. 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 (Bi and Hao, 2012), taking into account of the consequence of a finite seismic wave velocity and coherency loss effect.

For the study of SSI effects on the 1:22 scale model, a total of 4200 tests were performed. For the three-segment bridge pounding study, 420 tests were conducted, and for the study of the abutment pounding, a total of 3500 tests were performed. A total of 400 tests were carried out for the study on SSI effect in the laboratory involving rubber sandboxes. Another 120 tests were conducted for the study on inelastic bridge response with pounding.

For the tests based on the 1:125 scale model, relative displacement and pounding forces between two segments or between one segment and the abutments, bending moments at pier base and the deck displacement were measured. For the experiments on inelastic bridge response due to spatially varying ground motions, instead of the girder displacement, the relative displacement between the girder and the pier base was measured. For the 1:22 scale model testing, the deck displacement and the bending moment at the pier supports were recorded. To ensure the generality and comparability of the results obtained under different seismic loadings, the recorded data was normalised by the corresponding reference values obtained by averaging all the maximum responses due to the excitations of the same soil condition without pounding. The results from the studies were used to derive recommendations for the NZTA Bridge manual.

1.3 Outline

Chapter 2: Literature review

This chapter presents a literature review about the following topics:

1. Influence of spatial variation of ground motion on bridge response
1. Introduction

2. Influence of pounding on bridge response
3. Influence of SSI on bridge response
4. Influence of pounding and excitation spatial variation on inelastic bridge response
5. Experimental research on pounding and spatial variation of ground motions
6. Bridge design specifications on minimum seating length

Chapter 3: Effect of spatially varying excitation on elastic bridge response with pounding

The objective of this chapter was to experimentally evaluate the influence of spatial variation of ground motions on the pounding behaviour of three adjacent bridge segments. The effects of spatially varying ground motions on bridge pounding response were revealed by comparing with the results obtained from uniform ground motions. The effects of excitation soil condition and coherency loss on the bridge response were discussed. Furthermore, a two-segment bridge was tested and the results were compared with those of three-segment tests to reveal the influence of two-sided pounding on the relative displacement and the pounding force between adjacent segments.

Chapter 4: Effect of soil flexibility on bridge response with pounding

After studying the influence of spatially varying ground motions on bridge response with a fixed-base assumption, this chapter took a step further to study the effect of SSI on the bridge response based on the model used in Chapter 3. To incorporate SSI, sandboxes were fabricated using soft rubber to minimise the effect of rigid box walls and to allow movement of the sand. The influence of SSI was discussed by comparing the results with those obtained from the fixed base tests.

Chapter 5: Effect of excitation spatial variation on inelastic bridge response with pounding

In the previous experimental studies involving spatially varying ground motions, elastic material behaviour of bridge structures is often assumed. The effect of spatially varying ground motion on inelastic bridge response is unclear. Therefore this chapter studied the inelastic response of bridges subjected to spatially varying ground motions and pounding by testing two of the 1:125 scale bridge segments with artificial plastic hinges installed at
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each pier base and each connection between the pier and the girder. The results were compared with those obtained from the elastic bridge model to illustrate the importance of considering inelastic bridge response.

Chapter 6: Effect of abutment excitation on bridge response with pounding

Abutments are an important bridge structure component and will affect the girder response when pounding is involved. In most previous studies, the abutments were assumed fixed in position. However, the abutments will inevitably move under the ground excitation during an earthquake. This chapter therefore considered a single segment bridge and two abutments subject to spatially varying ground motion and pounding. Rigid body motion was assumed for the abutments and the bridges with various fundamental frequencies were considered. The effect of abutment excitations was studied by comparing the results with those obtained from considering the abutments fixed in position. Furthermore, the effect of contact stiffness at the abutment-girder interface was considered.

Chapter 7: Field tests of a bridge segment for determining minimum total gap of modular expansion joint

Modular expansion joint (MEJ) can be used to accommodate large relative displacement between adjacent bridge structures, and therefore can avoid damage of pounding and unseating. However, the studies on the minimum total gap for a MEJ to avoid pounding are limited. This chapter investigated the relative displacements of a three segment bridge considering spatial variation of ground motions and SSI. Pounding was not considered. The field test results were compared with those obtained from the fixed base tests conducted in laboratory.

Chapter 8: Recommendations for minimum seismic seating length

In this chapter, the maximum relative opening displacements obtained from Chapters 2 and 6 were compared with the minimum seating lengths calculated based on the suggestions in the NZTA Bridge manual. The inadequacy of the current NZTA Bridge manual suggestion was revealed. A set of empirical equations was recommended for
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obtaining adequate minimum seating lengths to prevent girder unseating based on the results obtained from Chapters 6 and 7.

Chapter 9: Conclusions and recommendations for future research

The conclusions drawn from each chapter were summarised in this chapter together with the recommendations for future studies.
Chapter 2

Literature review

Many bridges have suffered damage due to seismic-induced structural pounding. Examples can be found in almost all major earthquakes, e.g. the 1994 Northridge earthquake (Hall, 1994), the 1995 Kobe earthquake (Park, 1995), the 1999 Kocaeli earthquake (Youd, 2000), the 1999 Chi-Chi earthquake (Uzarski and Arnold, 2001), the 2008 Wenchuan earthquake (Lin, et al., 2008), the Chile earthquake in 2010 (Arias and Buckle, 2010) and the Christchurch earthquake in 2011 (Chouw and Hao, 2012). Gaps are needed between bridge segments to accommodate expansion and contraction due to temperature change. Although an expansion joint can allow some movement in earthquakes, it has often proved to be insufficient. With a conventional expansion joint, the gap is only a few centimetres (normally less than 4cm) to ensure a smooth traffic flow. When the relative closing 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 seating length. A reconnaissance report (Lin, et al., 2008) on the 2008 Wenchuan Earthquake in China identified that insufficient seating length was the first cause in the list of damage to bridges. Pounding damage and the probability of unseating, primarily arise from the relative girder displacement, which is attributable to several factors: dynamic properties of the structures; spatially varying ground excitation; and interaction between the supporting soil and the structures. This chapter reviews the findings of the previous research about the effects of spatially variation of ground motions, SSI and pounding on bridge response.

2.1 Influence of spatial variation of ground motion on bridge response

Spatial variation of ground motions denotes the differences in the amplitude and phase of seismic motions recorded over extended areas. The inconsistency of excitations results
from a variety of reasons, such as delayed arrival of seismic waves as a result of finite wave speed, known as the wave passage effect; the coherency loss effect due to reflection and refraction of waves through heterogeneous soil medium, and superposition of waves from multiple sources; the site response effect arising from differences in local soil conditions (Kiureghian and Neuenhofer, 1992).

The effect of spatially varying ground motions was initially studied in 1960’s. In early stage, the spatial variation of ground motions was attributable to only the wave passage effect, i.e. it was considered that the excitations at various supports were different only due to the propagation of the waves with finite speed, using simple finite element models (Bogdanoff, et al., 1965, Johnson and Galletly, 1972, Masri, 1976, Werner, et al., 1977). These studies recognised the importance of the wave passage effect on response of large dimensional structures.

Albdel-Ghaffar and Rubin (1982) investigated the seismic response of suspension bridges under correlated and uncorrelated multi-support excitations. The ground motions used were the finite Fourier transforms of existing recorded ground displacements of the 1979 Imperial Valley earthquake at the El Centro Differential array (Spudich and Cranswick, 1984), which was one of the first arrays installed. Spectral correlation analysis of the multiple-support excitations were estimated by using these Fourier transforms. It was concluded that the response values associated with the correlated multiple-support excitations are significantly different from those obtained from the uncorrelated case.

Investigations of the consequences of spatially varying ground excitation became popular after installation of the Strong Motion Array (SMART-1 array) in Lotung, Taiwan (Bolt, et al., 1982). The SMART-1 array consists of 37 stations placed in the configuration shown in Figure 2.1, which covers approximately 12 km². With data acquisition points in a 360º circle that cover such a large area, the data recorded are highly beneficial to the understanding of the three causes for spatially non-uniform ground excitations. Analytical models were developed based on the empirical data recorded at the SMART-1 array, e.g. (Harada, 1984, Harichandran and Vanmarcke, 1986, Hao, et al., 1989).
Zerva led extensive research on the spatial variation effects of ground motions, placing a huge emphasis on the importance of considering spatially varying ground motions in the analysis of pipeline response. In the study of the dynamic response of two- and three-span beams subjected to spatially varying ground motions, Zerva (1990) found that the structural response due to partially correlated motions may be either higher or lower than that due to fully correlated motions, depending on the properties of the structural systems and the seismic demand under evaluation. Zerva (1991), through the investigation of multi-supported structures, again confirmed that fully correlated ground motions may result in underestimation of the structural response. Another research by Zerva (1994) concluded that a low apparent wave velocity can lead to the most considerable lifeline response (the buried pipelines and the bridges) and the spatial incoherence may introduce significant quasi-static internal forces in these structures. Zerva and Zervas (2002) reviewed the analytical and numerical studies of spatial variation of ground motions prior to 2002.

Nazmy and Abdel-Ghaffar (1992) performed a nonlinear analysis on the response of cable-stayed bridges considering ground motion spatial variability. They concluded that the spatial variation of ground motions may increase the bridge response substantially,
2. Literature review

especially for rigid bridge and for bridge having different local soil properties. The significance of the increase depends particularly on span length, rigidity and structural redundancy.

Harichandran et al. (1996) studied the response of the Golden Gate suspension bridge, the New River Gorge bridge and the Cold Spring Canyon deck arch bridge due to spatially varying ground motion, and compared the results with those obtained due to uniform and time-delayed excitations. It was found that uniform ground motions underestimated the response of these long-span bridges and the use of time-delayed excitations is acceptable only for the longitudinal response of short arch bridges.

Monti et al. (1996) conducted a numerical study on the nonlinear response of bridges of varying stiffness and ductility subjected to spatially varying ground motions and concluded the spatially varying ground motions can reduce the ductility demand in the central piers and increase the ductility demand for those close to the abutments. Mylonakis et al. (1999) however, through a study of a curved multi-span bridge (SR14/I5) using both linear and nonlinear analysis, found spatially varying ground motions may have significant influence on the seismic demand of the bridge.

Hao (1998) analysed the effects of various bridge and ground motion parameters on the required seating lengths for bridge decks to prevent the pull-off-and-drop collapse with consideration of different intensities, different cross-correlations and different site conditions for the ground motions. It was concluded that the spatial variation of ground motions is the main reason for relative displacement development between bridge segments with similar dynamic properties.

Price and Eberhard (1998) investigated the effect of wave passage and coherency loss effects on the response of two-span prismatic beam bridge models with different spans and fundamental periods. The results indicated that for short bridges, spatial variation of ground motions may increase the central support reaction and the central support moment, but may decrease the end support reaction and the mid-span displacement. Sextos et al. (2003) found that the wave passage and coherency loss effects of spatial motions are generally beneficial for short bridges in terms of reducing the pier base bending moments and the absolute displacement, while detrimental in terms of increasing the relative displacements. Based on the curved bridges, Sextos et al. (2004) revealed the benefit of
the wave passage and coherency loss effects on reduction of the absolute displacement. It was also found that the spatial variation effect can increase the seismic force demand.

By studying a one-span frame and a reduced model of a 24-span bridge using linear response history analysis, Yang et al. (2002) revealed that the coherency loss and site response effects are more significant than the wave passage effect. Zhang et al. (2009) studied a multi-supported bridge subjected to spatially varying ground motions considering the wave passage, coherency loss and site response effects, and concluded that the bridge response depends heavily on all these effects. Chouw and Hao (2009) examined the relative girder movements of two bridge structures subjected to spatially varying ground motions without pounding. It was found that considering only the wave passage effect can underestimate the relative movements and spatially varying ground motions of soft soil condition can result in larger relative response than those simulating hard soil condition.

In summary, the importance of ground motion spatial variation effect has been recognised by almost all the researchers. In only a few cases, spatial ground motions can reduce the pier base bending moment and the girder displacement. However, for most of the time, ground motion spatial variation will cause larger relative girder movement and increase the risk of girder unseating. All the causes of ground motion spatial variation, i.e. the wave passage effect, the coherency loss effect and the site response effect are found to be important in affecting the structural response, and therefore, considered in this research.

2.2 Influence of pounding on bridge response

Pounding takes place when the relative closing movement of neighbouring bridge structures is in excess of the gap between them. Pounding can cause severe local damage to the bridge girder beams, enough to affect the functionality of the bridge (Chouw and Hao, 2008a).

In 1979, Kawashima and Penzien (1979) launched a research to correlate dynamic response of a 1:30 scale curved bridge model with those obtained from their analytical model. The experiment was conducted by Williams and Godden (1979). They found that the multiple collisions of girders have significant influence on both the amplitude and the
2. Literature review

frequency characteristics of bridge response, causing larger contact forces to be
developed at expansion joints. However, this research considered only the uniform
ground motions. Kawashima and Yabe (1996) analysed different unseating prevention
and energy dissipation devices to suppress the bridge response in order to minimise the
abutment pounding.

Malhotra (1998) adopted two uncoupled rods to represent bridge segments with
fundamental frequencies of 0.67 Hz and 1 Hz for the shorter and longer rod, respectively
to study the pounding response. The 1940 North-South El Centro earthquake was used. It
was concluded that seismic pounding generally decreased column deformation due to the
loss of energy via pounding and did not appear to increase the longitudinal separation at
the hinges. It was also found that pounding could generate large axial forces which were
not transmitted to the bridge column and foundations.

Ruangrassamee and Kawashima (2001) numerically studied a two-span bridge using 80
ground motion records. Fundamental periods of the bridge segments ranging from 0.05 s
to 3 s were used in multiple combinations in order to cover the typical bridges. It was
found that pounding can increase the relative displacement between decks, resulting in
the need of a larger seating length. It was also found that the colliding decks would
exchange their velocities after pounding and cause an increase in the displacement of the
long-period structure and hence increased the relative displacement between two
structures. However, this study ignored the influence of spatially varying ground motions.

DesRoches and Muthukumar (2002) performed a nonlinear analysis on a two-degree-of-
freedom bridge system to understand the influence of pounding on bridge responses. Two
fundamental period ratios between adjacent bridge spans were considered, i.e. highly out-
of-phase bridge spans ($T_1/T_2 = 0.32$) and slightly out-of-phase bridge spans ($T_1/T_2 = 0.71$).
For highly out-of-phase bridge spans, pounding reduced the displacements of the flexible
span. However, the displacements the stiffer span increased. It indicates that the stiff span
acted as a barrier to the flexible span while the flexible frame increased the response of
the stiff span since it naturally wants to displace more. For the slightly out-of-phase
bridge spans, pounding increased the displacements of the stiffer frame, however not as
much as the other case. The flexible frame was shown to have a similar response for both
the pounding and non-pounding cases. However, this study also ignored the influence of
spatially varying ground motions.
2. Literature review

Chouw and Hao (2005) numerically investigated the influence of spatially varying ground excitations and SSI on the pounding potential of two bridge frames. The spatially varying ground excitations are simulated stochastically based on the Japanese design spectra (JSCE, 2000) for soft and medium soil conditions accounting for the wave apparent velocity and the coherency loss effects. It was concluded that under spatially varying ground motions, SSI causes a larger required gap to avoid pounding.

Chouw et al. (2006) studied the effect of multi-sided pounding on bridge response considering spatially varying ground excitations. The numerical model consisted of a four-span bridge and was subject to 20 sets of spatially varying ground motions for each of three different apparent wave velocities. The ground motions used were developed based on the Newmark-Hall design spectra (Newmark and Hall, 1969). Abutments were considered. It was found that multi-sided pounding restricted the displacements of the central spans and hence increased the rate of pounding and the pounding force. Analyses of bridge seismic response without taking into account multi-sided pounding and abutment effects would result in underestimation of the pounding force.

Chouw and Hao (2008a) studied the influence of spatially varying near-source ground motions on the relative response of two bridge frames with varying fundamental frequencies. The ground motions were simulated based on the Japanese design spectra using an empirical coherency loss function (Hao, et al., 1989) and considered different wave apparent velocities. They confirmed that the impediment of girder movement due to pounding can reduce pier bending moment, and structures subjected to ground motions of hard soil condition may experience larger pounding forces than those subjected to ground motions of soft soil condition.

The size of the gap at bridge expansion joint is a large influencing factor of the pounding response of a bridge. Jankowski et al. (1998) conducted a numerical study on a five-span bridge, and it was shown that for a small gap size (0.01 m) there was more frequent pounding, but of smaller magnitude. However, for larger gap sizes (0.11 m) there was less pounding but of larger magnitude. For a range of ground motions this same pattern was observed. Furthermore it was shown that for larger gap sizes the base shears were higher compared to that for small gap sizes. However, if the gap size was increased enough to prevent pounding then the base shear would drop significantly.
Pounding between an abutment and its adjacent girder has also been investigated. Maragakis and Jennings (1987) modelled the rigid body motions of a short skew bridge and the pounding response with the abutments by a spring. They concluded that the impact between the deck and the abutment contributed to the planar rigid body rotation of the deck. Kim et al. (2000) examined the pounding response of a multi-span bridge with abutments. It was found that the largest relative displacements occurred between the abutment and the nearby girder, rather than between the girders. Lou and Zerva (2005) in the investigation of the response of a short, three-span, skewed, reinforced concrete bridge, and concluded that spatially varying ground motions can increase the pounding force at the abutment and the relative opening displacement with the end span.

Huo and Zhang (2013) numerically studied the effects of pounding and skewness on the seismic behaviours of typical multispans RC highway bridges considering spatially varying ground motions and SSI using the fragility function method. The authors concluded that pounding resulted in increased damage of skewed bridges which aggravate with large skew angles.

2.3 Influence of soil-structure interaction on bridge response

Soil-structure interaction (SSI) is the term used to describe the interplay of the characteristics of the structure, the supporting soil and the experienced ground motions. The seismic loading experienced by a structure depends on the response of the structure in relation with the soil (Wolf, 1985). The significance of SSI depends on the relative stiffness of the soil to that of the structure and its foundation (Spyrakos, 1990) and also on the characteristics of ground motion (Jeremić, et al., 2004). For the past twenty-five years, there has been vigorous debate upon the effects of SSI on structural response.

Conventional design specifications considered SSI to be beneficial for seismic response and therefore SSI was often neglected. SSI occurs in the presence of deformable soil supporting a structure and thus has the tendency to increase the fundamental period of structures. SSI can also allow energy dissipation, result in more damping and reduce the seismic forces acting on a structure (Mylonakis and Gazetas, 2000). The energy dissipation and damping mentioned above can occur due to wave propagation or hysteretic damping of the soil material (Pitilakis, et al., 2008).
By shake-table testing a single column with a mass block on its top and a 12-story RC frame model on soft soil, Lu et al. (2002) confirmed that SSI can elongate the fundamental period of the structure and increase damping compared to the fixed base case.

However, many researchers have highlighted the importance of SSI in the seismic response of bridges. It was found that a heavy structure, for example, a nuclear power station or an elevated-highway bridge, founded on relatively soft soil will suffer SSI effects (Wolf, 1985). A study by Mylonakis and Gazetas (2000) has pointed out that an increase in fundamental period of a structure due to SSI does not necessarily lead to smaller response. The examples can be found in (Mylonakis, et al., 2006, Celebi, 1998, Reséndiz and Roesset, 1985).

A study conducted by Spyrakos (1990) considered a single span bridge including abutments and the surrounding soil in order to investigate the significance of SSI on the seismic response of short span bridges. This study focused on pier behaviour. The author concluded that SSI could cause larger pier displacements and therefore should be considered in the design specifications to ensure the safety of bridges. Similar conclusions were drawn by Tongaonkar and Jangid (2003) in their investigation of the soil flexibility effects on an isolated bridge.

In fact, whether the SSI effects are beneficial or detrimental depends on many parameters (Pender, 1993, Wolf, 1994, Gazetas and Mylonakis, 1998), such as the dominant frequency, the angles of incidence waves of ground motions, the stiffness and the damping of the soil, the geometry, stiffness, slenderness and dynamic characteristics of the structure (Sextos, et al., 2004).

Simultaneous effects of spatially varying ground motions and SSI on bridge responses were also considered. Sextos et al. (2003) performed a parametric study on RC bridges accounting for spatial variability of ground motions, site response effects and SSI and concluded the coupling of the above effects can strongly affect the bridge responses. In addition, some studies also incorporated the effect of pounding. Mylonakis et al. (1999) conducted an analytical study on the performance of the southbound separation and overhead bridge considering the influence of spatial variability of ground motion, SSI and non-linear contact at the expansion joints. The results show that the spatial variability of ground motion appears to have lesser impact on the structural response than SSI, probably due to the high coherence of the ground motions considered in the analyses. Chouw and
Hao (2008a and b) investigated the influence of spatially varying ground motions and SSI on the relative response of two bridge segments with and without pounding. Both studies found that neglecting SSI can lead to underestimation of bridge relative responses. Another study performed by Chouw and Hao (2005) addressed the effect of spatial variations of ground motion with different wave propagation apparent velocities in soft and medium soil, and the influence of SSI on the pounding response of two adjacent bridge frames. It was found that considering SSI resulted in a larger gap required to avoid pounding under the uniform and non-uniform ground excitations of soft soil condition.

According to the past research, SSI may play an important role in the seismic response of bridges, especially when pounding is present. However, to the author’s best knowledge, no experimental investigation has incorporated all the effects of spatial variation of ground motions, SSI and pounding. Therefore, in this study, experimental investigations on these influence factors were carried out.

2.4 Influence of pounding and spatial variation of ground excitation on inelastic bridge response

Plastic hinges are often used in bridge structures. Spatially varying ground motions and pounding can have influence on the inelastic bridge response. However, very limited research has addressed this matter.

Research conducted by Singh and Fenves (1994) studied the nonlinear effect on the seismic response of two-level viaduct type bridges considering site response effects. It suggests that nonlinear analysis should be considered and the design provisions should ensure that the plastic hinge would occur in the column under strong earthquakes.

DesRoches and Muthukumar (2002) investigated the effect of pounding on inelastic response of multiple-frame bridges considering uniform ground motions. The numerical results indicated that pounding can have different influence on the inelastic bridge behaviour, depending on the relative stiffness of the participating structures and the ratio between the characteristic frequency of the ground motion and the fundamental frequency of the stiffer bridge.
Kim and Kim (2001) studied effects of pounding at expansion joint of concrete bridges with different gap sizes and peak ground acceleration. They concluded that the pounding effect is generally negligible on the ductility demand of bridges.

Lou and Zerva (2005) investigated the effects of spatially varying ground motions on the seismic response of a skewed, multi-span, RC highway bridge using linear and nonlinear finite element models. It was found that neglecting the yielding of the bridge components can underestimate the shear force and bending moment demand when spatially varying ground motions are considered.

Saxena et al. (2000) studied the relative significance of the wave passage, the coherency loss and the site response effects of spatially varying ground motions on the seismic response of long multi-span reinforced concrete highway bridges. The ductility demand at the piers was used for comparison. It was found that for the case of the soil under the bridge supports having different local soil conditions, the ductility demand could be twice as large as the corresponding value for the case of same local soil conditions. For the bridges considered, the incoherence effect appears to be more significant than the wave passage effect with the exception of the longer bridge subjected to relatively low apparent velocities of wave propagation. This study also revealed that an assumption of uniform ground motions can always underestimate the ductility demand for bridge analysed.

Sextos et al. (2003) performed a parametric study on twenty bridge structures of various structural types, dimensions and ground motion characteristics considering inelastic response of bridges and SSI. This research concluded that spatial variability and SSI both play an important role in the inelastic dynamic response of bridges. It was also found that SSI effects are, in general, beneficial in terms of forces developed while could be detrimental in terms of relative displacement. The ground motion variation can also result from local soil amplification, kinematic interaction and asynchronous pier yielding. Ignoring interrelation between these factors can result in an underestimation of the ductility demand in bridge piers by 25% on average.

According to these numerical studies, it is evident that spatial variation of ground motion can have influence on inelastic bridge responses. However, most studies are numerical. Experimental studies on effect of spatial ground motion on inelastic bridge response have been rarely reported.
2.5 Experimental research on pounding and spatial variation of ground motions

Early experimental investigations of bridge pounding can be traced to 1979, when Williams and Godden (1979) conducted a shake table test to investigate the dynamic responses of a 1:30 scale curved bridge model including pounding. The experimental model consisted of three segments with unequal spans and abutments. Uniform ground motions with different intensities were considered. It was concluded that large pounding force and relative movement can be expected at the expansion joints and energy absorbers were recommended.

Zhu et al. (2002) performed a shake table test on a girder and an abutment using a sinusoidal excitation to verify their proposed three-dimensional non-linear model. However, a fixed abutment was used, and hence the effect of abutment excitations on the pounding response was not unconsidered.

Johnson et al. (2008) shake table tested a quarter-scale, two-span reinforced concrete bridge with the objective of investigating the SSI effects. It was found that motion incoherency has little effect on symmetric bridge response.

In order to evaluate the recommendation of Eurocode 8 (2005) regarding the use of spatially varying ground motions on bridges longer than 400 m, Crewe and Norman (2006) carried out an experimental study on a 1:50 scale bridge model considering the wave passage effect of ground motions and pounding. They concluded that for bridges as short as 200 m, spatially varying excitation can still affect the bridge response significantly.

Guo et al. (2009) conducted a shake table test on a 1:20 scaled two-span base-isolated bridge model subjected to uniform ground motions to evaluate the linear and nonlinear viscoelastic impact models. It was found that both the impact models can predict the experimental results well and hence the linear viscoelastic model is recommended.

Dryden (2009) conducted a shake-table experiment on a two-span and four-span bridge at 1:4 scale to study the influence of soil-foundation-structure interaction on bridge seismic response. This study considered non-uniform ground motions. The effects of pounding and abutments were considered. The excitation of the abutments was taken into account.
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However, the research did not focus on the pounding response of the bridge, but on verification of the numerical model to simulate the soil-foundation-structure interaction.

Guo et al. (2012) conducted a pounding experiment on a two-span highway steel bridge model using shake tables in order to evaluate the impact stiffness of four types of contact-element models. This research focused on modelling of pounding, rather than on the global response of the bridge.

Wieser et al. (2012) investigated the seismic performance of seat-type abutments considering pounding with abutment using a 1:2.5 scale curved bridge model using four shake tables. The experimental results suggest that the engagement of the abutments during seismic events reduces the ductility demands on the columns.

Saiidi et al. (2013) studied the effect of bi-directional motions and abutment-superstructure interaction using a four-span bridge of 32 m on three shake tables and two actuators. This study concluded that pounding between superstructure and abutment led to large residual drifts at the piers near the abutments. However, spatially varying ground motions were not considered.

2.6 Bridge design specifications

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, 2009).

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 (2009) 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
2. Literature review

adjacent segments is not enough to determine the structural response; their own frequencies are also important.

Bridge design specifications should provide good indications for the seating length required to minimise the risk of girder unseating. Critical reviews of current specifications on seating lengths are limited. Chen (2004) reviewed the US seismic displacement requirements for bridges in a low-moderate seismic zone and evaluated the formula suggested by American Association of State Highway and Transportation Officials (AASHTO) (2010) for predicting the minimum seating length. The author indicated that the formula recommended by AASHTO provides a conservative abutment seating length and offers a more realistic estimate for the relative displacements of adjacent spans with a possible cumulative effect on the displacements from the many expansion joints. Chouw and Hao (2006) assessed the Japan Road Specifications (JRA) (2004) on the minimum girder seating length and found that under spatially varying ground motions, the seating length recommended by JRA specifications is insufficient. Chouw (2008) also found that when bridge structures experience different SSI, JRA can underestimate the necessary seating length. Sextos and Kappos (2009) reviewed the Eurocode 8-2 provisions (2005) and concluded that Eurocode 8-2 provisions can, in some cases, significantly underestimate the seating length required if spatially varying ground motions are neglected.

Spatial variation of ground motions has gained wide recognition for its importance in bridge seismic design. However, to the author’s best knowledge, amongst all the bridge design regulations, only Eurocode 8-2, AASHTO specifications and JRA specifications consider spatial variation of ground motion. Even if JRA considers spatially varying ground motions, research shows that JRA still may not be able to predict sufficient seating length under strong earthquakes.

Chouw and Hao, 2006) still showed that the recommendation of JRA on minimum seating length is insufficient. Therefore, there is a strong need to evaluate the New Zealand Bridge manual (NZTA, 2005).
3. Effect of spatially varying excitation on bridge pounding

Chapter 3

Effect of spatially varying excitation on elastic bridge response with pounding

Related paper:


3.1 Introduction

It has been found that experimental research on bridge pounding in general is insufficient to date, whilst experimental investigations on multi-segment pounding are even rarer. Although the wave passage effect of ground motions on structural behaviour has been experimentally investigated (Crewe and Norman, 2006), the coherency loss and site response effects have not been experimentally studied. The objective of this chapter is to experimentally evaluate the influence of ground motion spatial variation on the structural responses of a bridge with three identical segments. The wave passage, coherency loss and site response effects are addressed. The effect on the response of a given segment due to pounding on one or both ends is also studied. The models were scaled to the similitude laws wherever possible - an aspect neglected in most of the previous studies.
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3.2 Experimental model

3.2.1 Prototype structure and model

In order to investigate the response of a bridge segment subjected to the effects of pounding from adjacent segments on one or both sides, a system consisting of three identical segments was considered (Figure 3.1). For simplicity, it is assumed that each of the bridge segments experiences the ground excitation $a_{o}(t)$ acting at the middle of the bridge segment, where $o$ is the number of the considered sites ($o = 1, 2$ and 3). Each of the bridge segments has a span of 100 m. To limit the number of influence factors, consideration of abutments is deferred to Chapter 6.

![Figure 3.1 Considered system](image)

Table 3.1 Dimensions and dynamic characteristics of the prototype structure

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Value</th>
<th>Quantity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span length</td>
<td>100 m</td>
<td>Pier mass</td>
<td>$1.93 \times 10^{5}$ kg</td>
</tr>
<tr>
<td>Pier height</td>
<td>15.5 m</td>
<td>$I_{\text{deck}}$</td>
<td>$9.337$ m$^{4}$</td>
</tr>
<tr>
<td>Pier width</td>
<td>3.44 m</td>
<td>$I_{\text{pier}}$</td>
<td>$0.387$ m$^{4}$</td>
</tr>
<tr>
<td>Pier thickness</td>
<td>1.48 m</td>
<td>$K_{\text{bending}}$</td>
<td>$7.189 \times 10^{7}$ N/m</td>
</tr>
<tr>
<td>Segment mass</td>
<td>$1.702 \times 10^{4}$ kg/m</td>
<td>Fundamental frequency</td>
<td>$0.98$ Hz</td>
</tr>
<tr>
<td>Effective mass (at height of bridge deck)</td>
<td>$1.895 \times 10^{6}$ kg</td>
<td>Modulus of Elasticity (50 MPa concrete)</td>
<td>$30$ GPa</td>
</tr>
</tbody>
</table>

The prototype bridge structure of the whole research was based on one segment of the Newmarket Viaduct replacement bridge in Auckland, New Zealand. The modelled
3. Effect of spatially varying excitation on bridge pounding

Segment was a single cell box girder with a total length of 100 m. The two bridge piers are 15.5 m high and 50 m apart. The construction drawings of the Newmarket Viaduct were used to construct the numerical model from which the dynamic characteristics of the prototype structure are 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 50 MPa.

The experimental models were constructed based on the prototype bridge structure using scale modelling. A detailed description of the principle and the derivation of the similitude relations can be found in the report by Dove and Bennett (1986). For the models used in this research, 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 $Q$. 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. Polyvinylchloride (PVC) was selected for constructing the scale models mainly because of its low modulus of elasticity of 2.5 GPa, 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.
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**Table 3.2** Original and revised similitude requirements and scale ratios of the model structure

<table>
<thead>
<tr>
<th>Physical quantity</th>
<th>Original similitudes</th>
<th>Revised similitudes</th>
<th>Scale factor (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (L)</td>
<td>$N_L$</td>
<td>$N_L$</td>
<td>125</td>
</tr>
<tr>
<td>Time (t)</td>
<td>$N_t$</td>
<td>$N_t$</td>
<td>2</td>
</tr>
<tr>
<td>Modulus of elasticity (E)</td>
<td>$N_E$</td>
<td>$N_E$</td>
<td>12</td>
</tr>
<tr>
<td>Mass (M)</td>
<td>$N_M = \frac{N_E N_L^2}{N_a}$</td>
<td>$N_M = \frac{N_E N_L^2}{N_a}$</td>
<td>187,500</td>
</tr>
<tr>
<td>Acceleration (a)</td>
<td>$N_a = \frac{N_L}{N_t^2}$</td>
<td>$N_a = \frac{N_L}{N_t^2}$</td>
<td>31.25</td>
</tr>
</tbody>
</table>

**Table 3.3** Scaled values of the parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span length</td>
<td>800 mm</td>
<td>Girder depth</td>
<td>20 mm</td>
</tr>
<tr>
<td>Pier height</td>
<td>124 mm</td>
<td>Girder width</td>
<td>120 mm</td>
</tr>
<tr>
<td>Pier width</td>
<td>21 mm</td>
<td>Deck thickness</td>
<td>3 mm</td>
</tr>
<tr>
<td>Pier thickness</td>
<td>3 mm</td>
<td>$I_{\text{deck}}$</td>
<td>$3.82 \times 10^4$ mm$^4$</td>
</tr>
<tr>
<td>$I_{\text{pier}}$</td>
<td>45 mm$^4$</td>
<td>Bending stiffness</td>
<td>$1.53 \times 10^3$ N/m</td>
</tr>
<tr>
<td>Seismic mass</td>
<td>10.11 kg</td>
<td>Fundamental frequency</td>
<td>1.96 Hz</td>
</tr>
</tbody>
</table>

3.2.2 **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 × 1 mm cross section. The pounding head consists of a 25 mm 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 1 N to a total of 10 N. 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.
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Figure 3.2 Schematic drawing of the model with details of the pounding and measuring heads

3.2.3 Equipment and testing

Three APS Model 400 Electro-Seis shake tables, each with a frequency range of 0-200 Hz, and a peak-to-peak stroke of 158 mm were utilised to apply the earthquake motions to the models. The shake tables are unidirectional with platform dimensions of $350 \times 350$ mm. Displacement transducers were installed between adjacent segments to measure the relative girder displacement. Three unidirectional accelerometers with a measuring capacity of $\pm 2$ g 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 1.965 Hz, 1.966 Hz and 1.965 Hz with corresponding damping ratios of 1.9%, 2% and 2.1%, respectively.

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
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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 8 kg 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 3 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](image)

3.2.4 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
3. Effect of spatially varying excitation on bridge pounding

(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 (2012) 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, 2012). 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)

\[
\gamma_{ij} = \exp(-\beta d_{ij}) \exp(-\alpha d_{ij}^{1/2} f^2) \exp(-i2\pi f d_{ij} c_a)
\]  

(3.1)

where \(\beta\) 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 \(\alpha\) is a function of the following form:

\[
\alpha(f) = \frac{a}{f} + bf + c \quad f \leq 10 \text{ Hz}
\]

\[
\alpha(f) = \alpha(10) \quad f > 10 \text{ Hz}
\]  

(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 (2012).
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Table 3.4 Parameters for coherency loss functions

<table>
<thead>
<tr>
<th>Coherency loss</th>
<th>$\beta$</th>
<th>$a$</th>
<th>$b$</th>
<th>$c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highly (SMART1-Event45)</td>
<td>$1.109 \times 10^{-4}$</td>
<td>$3.583 \times 10^{-3}$</td>
<td>$-1.811 \times 10^{-6}$</td>
<td>$1.177 \times 10^{-4}$</td>
</tr>
<tr>
<td>Intermediately</td>
<td>$3.697 \times 10^{-4}$</td>
<td>$1.194 \times 10^{-2}$</td>
<td>$-1.811 \times 10^{-5}$</td>
<td>$1.177 \times 10^{-4}$</td>
</tr>
<tr>
<td>Weakly</td>
<td>$1.109 \times 10^{-3}$</td>
<td>$3.583 \times 10^{-3}$</td>
<td>$-1.811 \times 10^{-5}$</td>
<td>$1.177 \times 10^{-4}$</td>
</tr>
</tbody>
</table>

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. 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 500 m/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 25 Hz, 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 Simulated ground motions: (a) Accelerations $a_{g1}$, $a_{g2}$ and $a_{g3}$ and (b) displacements $u_{g1}$, $u_{g2}$ and $u_{g3}$ for high correlation of excitations of soft soil condition

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 0.72 g. The coherency was highly correlated. The separation distance between different supports was 100 m. Figure 3.5 shows 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.
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![Response spectra of simulated ground motions and target design spectra for three different soil conditions](image1)

**Figure 3.5** Response spectra of simulated ground motions and target design spectra for three different soil conditions

![Coherency loss of simulated highly correlated ground motions with distance of 100 m and model coherency loss function](image2)

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

3.3 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
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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.

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

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil condition</th>
<th>Correlation of ground motion</th>
<th>Reference response</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Absolute displacement (mm)</td>
<td>Bending moment (Nm)</td>
</tr>
<tr>
<td>1</td>
<td>Soft soil</td>
<td>Highly</td>
<td>4.15</td>
</tr>
<tr>
<td>2</td>
<td>Soft soil</td>
<td>Intermediately</td>
<td>4.15</td>
</tr>
<tr>
<td>3</td>
<td>Soft soil</td>
<td>Weakly</td>
<td>4.15</td>
</tr>
<tr>
<td>4</td>
<td>Shallow soil</td>
<td>Highly</td>
<td>3.02</td>
</tr>
<tr>
<td>5</td>
<td>Strong rock</td>
<td>Highly</td>
<td>2.33</td>
</tr>
</tbody>
</table>

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 1 for the reference cases. The reference case was the situation when the bridge segment 1 was excited by the ground motions based on design spectra for different soil stiffnesses at site 1 without pounding. From the corresponding 20 maximum values, i.e. the peak absolute displacements and the peak bending moments obtained respectively from the 20 sets of ground motions at site 1, 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
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Illustrate the consequences of different considered ground motions and the number of bridge segments involved for the bridge response.

3.3.1 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 1 when the two adjacent segments were subjected to 0.2 s delayed but perfectly correlated, spatially varying and uniform ground motions with and without pounding. The 5 s 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 1 (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 1 appears to be less than that caused by uniform ground motion most of the time, except at 1.6 s and 2.3 s. 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 3.8 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.
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Figure 3.7 Time histories (1 s to 6 s) of the absolute displacements of segment 1 under highly correlated ground motions of soft soil condition

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, i.e. 4.15 mm (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
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3.9(b) compares the maximum pounding forces corresponding to the 20 excitations of different cases, i.e. uniform, time delayed and spatially varying ground motions involving all three segments. From this figure, the excitations with 0.2 s time delay resulted in greater pounding force development than spatially varying ground motions (comparing an averaged maximum pounding force of 35.6 N with 25.6 N). 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 Figure 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%.
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**Figure 3.9** Consequence of spatially non-uniform ground motions. (a) Normalised opening relative displacements between segments 1 and 2, (b) pounding force between segments 2 and 3, and (c) bending moment at pier B, due to highly correlated ground motions of soft soil condition considering three segment pounding

3.3.2 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
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displacements and pounding forces between segments 1 and 2 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 3.98 mm being the maximum opening relative displacement between segments 1 and 2. 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 2.32 mm and 1.54 mm. From Figure 3.10(b), the averaged maximum pounding force between segments 1 and 2 when experiencing ground motions of soft soil condition was found to be 31.62 N, 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 1 and 2 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, 1.97 Hz, 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.
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Figure 3.10 Consequences of soil type excitation for the maximum values of (a) the normalised relative displacement and (b) the pounding force between bridge segments 1 and 2 considering high correlation for the excitations.

3.3.3 Effect of ground motion variations due to coherency loss

Figure 3.11 compares the response of segments 1 and 2 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 0.51 Nm, 0.563 Nm and 0.615 Nm, compared with the 0.798 Nm 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
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exception that excitation set 7 caused the maximum pounding force of 42.7 N, 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.

![Graphs showing normalised maximum bending moment, normalised relative displacement, and maximum pounding force](image)

**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 1 and 2 under spatially varying excitations with pounding
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3.3.4 Effect of two-sided pounding

The time-histories shown in Figure 3.12 depict the relative displacements between segments 1 and 2 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](image.png)

**Figure 3.12** Comparison between the time histories (2 s to 7 s) of the relative displacement between segments 1 and 2 due to one and two-sided pounding under intermediately correlated spatially varying ground motions of soft soil condition.

Figure 3.13 displays the pounding forces during impact of segments 1 and 2 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 1 and 2 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.
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Figure 3.13 Comparison between the time histories of the pounding force between segments 1 and 2 due to one and two-sided pounding 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 2, 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 1 and 2, which was less than the response of 0.812 due to pounding on one side only. From the results shown in Figure 3.14(a) and (b), the magnification of the bridge responses, i.e. the bending moment of pier C and the opening relative displacement of segments 1 and 2, 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 Figure 3.12 and Figure 3.13, the pounding development was found to be greater for two-sided pounding than one-sided, noticing that there was a 2.5 N 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 3, which introduced more energy to segment 2, but limited the movability of segment 2, resulting in greater pounding force between segments 1 and 2 but smaller separation and smaller bending moment than those caused by one-sided pounding.
3. Effect of spatially varying excitation on bridge pounding

![Diagram](image)

**Figure 3.14** Consequences of one and two-sided pounding for: (a) bending moment at pier C, (b) the normalised relative displacement between segments 1 and 2, and (c) the pounding force between bridge segments 1 and 2 under spatially varying excitations of strong rock condition

3.3.5 **Effect of pounding on unseating potential**

Figure 3.15(a) shows the time histories of the opening relative displacement of segments 1 and 2 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 1 and 2 were given an initial separation sufficient to avoid
3. Effect of spatially varying excitation on bridge pounding

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 1 and 2, compared with the non-pounding case (solid line). Figure 3.15(b) summarises the peak opening normalised relative displacements of segments 1 and 2 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 1.687 mm to 4.009 mm), and hence increases the seat length required to prevent unseating. This finding has provided evidence for the suspicion (Abdel 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 excitation set 12: (a) Time histories of relative opening displacement, and (b) the normalised maximum opening relative displacement between segments 1 and 2, with and without pounding, under highly correlated ground motions of soft soil condition.
3.4 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:

\[
F = \begin{cases} 
  k_h (u_1 - u_2 - g)^{3/2} + c_h (\dot{u}_1 - \dot{u}_2) & u_1 - u_2 - g > 0 \\
  0 & u_1 - u_2 - g \leq 0 
\end{cases} 
\]

(3.3)

where  
- \( k_h \) = nonlinear impact spring stiffness
- \( c_h \) = impact damping coefficient
- \( u_1 \) = displacement of bridge superstructure segment 1
- \( u_2 \) = displacement of bridge superstructure segment 2
- \( g \) = 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 244 kN/m and 30.46 N 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,
3. Effect of spatially varying excitation on bridge pounding

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.

Table 3.6 Average difference between calculated and measured pounding forces

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
<th>Case 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake set number*</td>
<td>Average difference (N)</td>
<td>Earthquake set number</td>
<td>Average difference (N)</td>
<td>Earthquake set number</td>
</tr>
<tr>
<td>5</td>
<td>3.12</td>
<td>1</td>
<td>3.68</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>2.94</td>
<td>2</td>
<td>3.65</td>
<td>10</td>
</tr>
<tr>
<td>13</td>
<td>2.54</td>
<td>6</td>
<td>2.84</td>
<td>12</td>
</tr>
<tr>
<td>18</td>
<td>3.54</td>
<td>9</td>
<td>2.61</td>
<td>16</td>
</tr>
</tbody>
</table>

*The earthquake record number corresponds to the earthquake record from the 20 records generated for each ground motion case (see Table 3.5).
3. Effect of spatially varying excitation on bridge pounding

Figure 3.16 Comparison of measured and calculated pounding force

3.5 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 0.1 mm. 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. Figure 3.17(a) and (b) compare the girder displacement of the bridge segment 1 relative to the ground without pounding effect and the pounding force development at the contact location of segments 2 and 3 under highly correlated ground motions of shallow soil condition. A reasonably good agreement between the numerical and experimental results is visible.
3. Effect of spatially varying excitation on bridge pounding

Figure 3.17 Comparison between numerical and experimental results. (a) Displacement of segment 1 relative to the ground and (b) pounding forces between segments 2 and 3 under highly correlated excitation of shallow soil condition

3.6 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, i.e. 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 girder displacement by as much as 20% of the average maximum absolute displacement due to 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.
3. Effect of spatially varying excitation on bridge pounding

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 excitations 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.
3. Effect of spatially varying excitation on bridge pounding

8. The Hertz-damped impact model has shown the potential to calculate the pounding force expected between bridge superstructures during earthquakes.
Chapter 4

Effect of soil flexibility on bridge pounding

4.1 Introduction

This chapter focuses on the seismic response between three identical segments considering the simultaneous effects of spatially varying ground motions, SSI and pounding. To the author’s best knowledge, no experimental investigation has yet incorporated all these effects. SSI was incorporated by placing the models on sand contained in the soft rubber sandboxes. The spatially varying ground motions considered are based on the New Zealand design spectra (NZS 1170.5, 2004) for soft soil, shallow soil and strong rock conditions. The results with SSI were compared with those obtained from the fixed base tests. In this work the movability of the foundation soil was accounted, which has been reported before. Although the response of the sand filled rubber boxes did not well reflect the real soil behaviour, the results are still useful as at least, to some extent.

4.2 Experimental model

4.2.1 Prototype bridge, model and testing

This study was based on the same bridge structures considered in Chapter 3. Section 3.2 provides detailed information about the prototype, the model and the spatially varying ground motions used.

To incorporate soil flexibility, instead of fixed foundations, square footings simulating shallow foundations were adopted and placed on subsoil contained in flexible rubber
boxes. Each footing was constructed using a square plywood board with dimensions of 90 × 90 × 10 cm. The footings were bedded on sand with the top surface of the footings flush with the sand surface. No footing sliding was observed in the pre-tests. With footings on sand, the fundamental frequency and the damping ratio of the models were found to be around 1.90 Hz and 4.6%, respectively. To ensure that the same ground excitations were applied by the shake tables for both types of test, i.e. with and without SSI, an additional mass of 5 kg was loaded on each shake table for the fixed base case. This was done in order to maintain a consistent total mass on each shake table, with the additional 5 kg mass substituting for the mass of the two boxes with sand used in the SSI case. The setup of the shake tables, bridge models, and rubber boxes is shown in Figure 4.1.

**Figure 4.1** Setup of the test system with rubber boxes

### 4.2.2 Flexible rubber boxes

To incorporate the soil flexibility effects, six identical rubber boxes were constructed with the dimensions shown in Figure 4.2. Rubber was used to reduce the influence of the boundary condition (the rigid wall effect) arising from wave reflection on the boundary and variation of system vibration mode (Lu, et al., 2002). Natural rubber with a hardness of 40-50 points (Durometer Shore hardness scale) was used. Side barriers made of plywood with dimensions of 100 × 100 × 10 mm were glued vertically on the wooden base (shown in Figure 4.2(c)). The side barriers were used to prevent the sand expanding sideways, as this study only considered the soil flexibility in the excitation direction. Strip connectors were used to hold the rubber boxes together with bonding occurring along the wooden base. A gap of 5 mm was left between the bottom of the rubber box and the wooden base, in order to allow motion of the walls in a direction perpendicular to the direction of excitation, with minimal frictional effects. Plastic bags were used to prevent sand leakage.
For each shake table, a rectangular wooden base with dimensions of $540 \times 330 \times 8$ mm was used to support the rubber boxes on the shake table. Sand was used to fill the boxes to a depth of 100 mm. A single box with sand was weighed, and found to be 2.45 kg. By applying white noise, the fundamental frequencies of the rubber boxes with and without sand were found to be 24.7 Hz and 15.1 Hz, respectively. To ensure the same initial soil conditions for each test, the rubber boxes were refilled after each excitation and compacted by applying a sinusoidal signal with a frequency of 50 Hz and duration of 30 seconds.

Figure 4.2 Flexible rubber boxes: (a) Top view, (b) side view and (c) top view of boxes filled with sand

Figure 4.3 compares the displacements of box 1A (measured at the centre of the outward wall) with those of the shake table due to the ground motion under strong rock conditions. As can be seen, the displacements of box 1A and the shake table show a significant discrepancy, which shows the influence of the flexibility of the sandboxes.

Owing to the intrinsic difficulties associated with scaling soil particles, the bridge model on sand reflected the characteristics of a prototype structure founded on stiff soil. For this
4. Effect of soil flexibility on bridge pounding

study, only the influence of the characteristics of excitations of different soil conditions on the model response were considered while the same sand was used.

![Figure 4.3](image)

**Figure 4.3** Comparison of displacements of the rubber box 1A (Figure 4.1) and the shake table, subjected to the same highly correlated ground motion of associated with the strong rock condition.

### 4.3 Results and discussion

In order to compare the structural responses in a valid manner, the bending moment at the pier support, the relative girder displacement and the pounding force were normalised. The bending moment was normalised by the average of 20 maximum values due to ground motions under the same soil condition without pounding. Similarly, the relative displacement and ponding force were normalised by averages of the 20 maximum displacements and base shear of segment 1. The ground motions have an average peak ground acceleration of 1.0 g. Table 4.1 summaries the corresponding reference values for each case. The reference values reflected the characteristics of the seismic responses of the bridge due to the simulated ground motions based on New Zealand design spectra (NZS 1170.5, 2004). After normalising the responses resulting from ground motions with different frequency content, the responses then became comparable. Among the average maximum responses, the greater value at either side of the bridge was chosen to obtain the most conservative responses. The ratios of the normalised results are only valid for the cases considered or for very similar cases.
4. Effect of soil flexibility on bridge pounding

Table 4.1 Ground motion cases and reference responses

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil condition</th>
<th>Coherence</th>
<th>Reference response</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Displacement (mm)</td>
</tr>
<tr>
<td>1</td>
<td>Soft soil</td>
<td>Highly</td>
<td>4.85</td>
</tr>
<tr>
<td>2</td>
<td>Soft soil</td>
<td>Intermediately</td>
<td>4.85</td>
</tr>
<tr>
<td>3</td>
<td>Soft soil</td>
<td>Weakly</td>
<td>4.85</td>
</tr>
<tr>
<td>4</td>
<td>Shallow soil</td>
<td>Highly</td>
<td>3.55</td>
</tr>
<tr>
<td>5</td>
<td>Strong rock</td>
<td>Highly</td>
<td>2.74</td>
</tr>
</tbody>
</table>

4.3.1 Effect of SSI on girder displacement

Figure 4.4 shows the absolute displacements of segment 1 with and without SSI due to the excitations associated with different soil conditions. Pounding was excluded by providing sufficient gaps of sufficient width between the adjacent girders. In Figure 4.4(a), the maximum girder displacement under fixed base conditions of 4.75 mm was almost the same as that of 4.83 mm with consideration of SSI effects. Figure 4.4(b) shows that under the ground motion with shallow soil condition, SSI reduced the maximum girder displacement from 3.74 mm to 3.27 mm, i.e. a 13% reduction. From Figure 4.4(c), the maximum absolute displacements are 2.73 mm and 2.26 mm, with and without SSI, respectively, i.e. SSI effects cause a 21% of increase in the maximum girder displacement.

Figure 4.4(a) and (b) show that the patterns of the girder displacements with and without SSI were similar. In contrast, Figure 4.4(c) shows greater discrepancies between the girder displacements with and without SSI. SSI had a larger influence on the bridge model under the ground motions associated with the strong rock condition. The observations reveal that SSI can have a detrimental effect in terms of causing a greater displacement of the bridge model when the ground motions of strong rock condition apply.
4. Effect of soil flexibility on bridge pounding

Figure 4.4 Absolute displacements of segment 1, with and without SSI, for ground motions of (a) soft soil, (b) shallow soil, and (c) strong rock conditions

Figure 4.5 shows the 20 normalised maximum displacements of segment 1 with respect to the 20 corresponding ground motions of various soil conditions with and without SSI. Pounding effects are excluded. The reference responses are shown in Table 4.1. The number in brackets is the average of the 20 maximum responses.
4. Effect of soil flexibility on bridge pounding

Figure 4.5 Consequences of SSI for normalised maximum absolute displacements of segment 1, without pounding, due to ground motions of (a) soft soil, (b) shallow soil and (c) strong rock conditions

In Figure 4.5(a) and (b), it is shown that SSI effects resulted in almost the same average maximum absolute displacement of segment 1 due to the ground motions of the soft and shallow soil conditions, i.e. only a 2% decrease and 4% increase, respectively. When the ground motions of the strong rock condition were considered, an increase of 16% in the average maximum absolute girder displacement was observed. SSI can increase the maximum girder displacement by up to 8% (ground motion set 17), 32% (ground motion set 8) and 36% (ground motion set 9) due to the ground motions of soft soil, shallow soil and strong rock conditions, respectively. In Figure 4.5(c), only two ground motions (sets 6 and 18) among the twenty were found to reduce girder displacements when SSI effects were considered. It is almost certain that SSI can increase the girder displacement when the ground motions of the strong rock condition were applied.
4. Effect of soil flexibility on bridge pounding

4.3.2 Simultaneous effect of SSI and spatially varying excitation

To investigate the combined effects of SSI and spatially varying ground motions, the normalised maximum relative opening displacements between segments 1 and 2, with and without SSI, were compared (see Figure 4.6). The relative opening displacements were measured by the displacement transducer shown in Figure 1, and are equivalent to the difference between the displacements of segments 1 and 2. This is done only for the purpose of isolating the influence of pounding. Under actual and real conditions, this does not have any significant practical meaning. To avoid pounding, the segments were given a sufficient gap between them. High correlations between the ground motions at adjacent supports were considered.

Figure 4.6 shows that relative displacement between identical segments cannot be avoided if spatially varying ground motions are anticipated. In Figure 4.6(a), very little variation between the normalised maximum relative opening responses was observed. This is expected as the result of Figure 4.5(a) has revealed that SSI, on average, had very little influence on the absolute displacement of the bridge girder under the ground motions of the soft soil condition. Therefore, with a high correlation of the ground motions, the absolute displacements of segment 2 with SSI can be expected to be similar with those of the fixed base condition. Still, noticeable differences between the normalised relative displacements with and without SSI were observed in ground motion sets 9, 18 and 19. The difference due to ground motion 9 was more than 10% of the reference displacement (0.98 cf. 0.89 for fixed base and SSI, respectively). In contrast, Figure 4.6(b) and (c) show that SSI increased the relative opening displacements of the segments by, on average, 7% and 11% of the reference displacements due to the highly correlated ground motions of the shallow soil and strong rock conditions, respectively. The most significant increase of the normalised maximum relative opening displacements that included SSI effects occurred during ground motion set 1 of the shallow soil condition, and set 3 of the strong rock condition, with respective increases of 27% and 37%. As shown in Figure 4.6(b), in almost all cases, the maximum relative displacements with SSI are greater than those of the fixed base cases, with only two exceptions (sets 13 and 17). The increase of maximum relative displacements due to SSI can also be seen in all cases in Figure 4.6(c).
4. Effect of soil flexibility on bridge pounding

The results show that without pounding, SSI led to a greater increase in the maximum relative opening displacement under the spatially varying ground motions of the strong rock condition, than those of the shallow soil condition. Under the ground motions of the soft soil condition, SSI had negligible effect on relative movement between the girders.

**Figure 4.6** Consequences of SSI for the normalised maximum relative opening displacements between segments 1 and 2, without pounding effects, resulting from highly correlated ground motions of (a) soft soil, (b) shallow soil, and (c) strong rock conditions

### 4.3.3 Simultaneous effect of SSI, spatially varying excitation and pounding

In order to investigate the simultaneous effects of SSI and the spatial variation of excitations including pounding, the relative opening displacement and the pounding force time histories between segments 1 and 2 are compared (see Figure 4.7). The segments were given a zero initial gap, and subjected to the highly correlated excitations of the shallow soil condition. Including the effects of SSI, results in both the maximum relative opening displacement and the maximum bending moment at the pier base, being larger...
than the measured values for the fixed base assumption. In contrast, the maximum pounding force with the fixed base condition is larger than that with SSI effects included.

The damage potential of bridge girders depends not only on the magnitude of the contact force, but also on the number of strong impacts. As shown in Figure 4.7(b), the fixed base case resulted in more strong impacts than the SSI case. SSI reduced the impact forces and lessened the number of strong pounding impacts. In the cases considered, SSI can reduce pounding force. However, SSI resulted in larger relative opening displacements. Hence, SSI may increase the risk of unseating. The results presented in Figure 4.7 show that with pounding, SSI can contribute to larger bridge displacement responses, even though SSI can increase the damping in a structure.

Figure 4.7 Consequence of SSI for (a) relative opening displacement, (b) pounding forces of segments 1 and 2, and (c) bending moment of pier 1B, due to highly correlated excitation of shallow soil condition

In Figure 4.8(a), the normalised pounding forces in the fixed base case are larger than those of SSI case, for almost all cases with the exception of the ground motion set 8. For set 8, the maximum responses with and without SSI, are essentially the same. More than
50% of the pounding force due to ground motion set 1 was reduced, when SSI was considered. On average, the pounding force for the SSI case is 19% less than that of the fixed base case. In Figure 4.8(b), SSI, for most cases considered, caused larger relative opening displacements, leading to a 33% increase in the average maximum relative opening displacements compared to the fixed base case. This is equivalent to a 10 cm difference in the prototype structure. Increases in the normalised bending moments due to SSI conditions were also observed in Figure 4.8(c). The average normalised maximum bending moment of pier 1B with fixed base assumption is 0.74, while for the SSI case it is 0.95 (an increase of 28%).

![Figure 4.8](image_url)

**Figure 4.8** Consequences of SSI under highly correlated excitations of the shallow soil condition. Maximum values of the normalised (a) pounding force, (b) relative opening displacement between segments 1 and 2, and (c) bending moment of pier 1B

Figure 4.9 shows the differences between the maximum responses, with and without SSI effects, subjected to the highly correlated ground motions of soft soil, shallow soil and strong rock conditions. The results are presented in a normalised form with the reference responses indicated in Table 4.1. The positive and negative values indicate whether SSI has, respectively, increased or reduced the structural responses. The averages of the
4. Effect of soil flexibility on bridge pounding

differences corresponding to the 20 ground motions of different soil conditions are shown in brackets.

The positive averages shown in Figure 4.9(a) indicate that SSI can increase the pier bending moment of the middle segment regardless of soil conditions. The values in brackets show that the spatially varying ground motions of the strong rock condition resulted in the largest averaged difference, i.e. 0.13, which means that SSI can increase the bending moment of pier 2A by up to 13% of the corresponding reference bending moment (presented in Table 4.1). The differences due to the ground motions of the soft and shallow soil conditions have a spread above and below zero, while those due to the excitations of the strong rock condition are all above zero except for the case due to ground motion set 16. This observation confirms that SSI will induce amplification of the pier bending moments of the middle segment, under spatially varying ground motions of the strong rock condition.

SSI can reduce the maximum pounding forces, regardless of soil condition. It can be seen that all the average values shown in Figure 4.9(b) are negative, except for a few associated with ground motions of the soft soil condition. By comparing these average values (in brackets), it can be seen that compared to the results due to excitation of other soil conditions, SSI resulted in more reduction of the pounding force under the ground motions of the strong rock condition, i.e. 2.58 times of the reference base shear. The maximum reduction was found when the bridge was subjected to the ground motion set 12 of the shallow soil condition, i.e. 4.4 times of the reference base shear. The average results in Figure 4.9(c), show that SSI tends to increase the relative opening displacement. Hence, the potential of unseating will be increased when ground motions of the soft soil condition is anticipated.
4. Effect of soil flexibility on bridge pounding

4.3.4 Effect of SSI and coherency loss of ground motions

To investigate the simultaneous influence of coherency loss and SSI on structural response, the differences between the maximum responses for the SSI and the fixed based case, are shown in a normalised form in Figure 4.10 (average values given in brackets). Ground motions based on the soft soil spectrum with high, intermediate and weak correlation were considered. Positive averages were observed for all the coherency loss cases in Figure 4.10(a), indicating that SSI can increase the bending moment of pier 3A. By comparing the averages of the normalised differences, the spatially varying ground excitations with the weak correlation caused a 10% increase in the maximum bending moment. From Figure 4.10(b), it can be seen that under most of ground motions, SSI reduced the maximum pounding force between segments 2 and 3, regardless of the correlation of ground motions. If a weak correlation is anticipated, the pounding force can be reduced by almost 1.5 times, compared to the maximum base shear without pounding.
4. Effect of soil flexibility on bridge pounding

...effect resulting from the same set of ground motions. In an extreme case, a reduction as high as 5 times of the reference force can even be induced by SSI. Figure 4.10(c) shows that SSI increased the relative opening displacement between segments 2 and 3, especially when the bridge was subjected to highly correlated ground excitations. Notably, at least 13% greater seating length is demanded to avoid unseating between segments 2 and 3, when the structure is subjected to highly correlated spatial excitations where pounding and SSI are considered simultaneously.

**Figure 4.10** Consequence of SSI and coherency loss for the maximum values of the normalised (a) bending moment of pier 3A, (b) pounding force, and (c) relative opening displacement between segments 2 and 3

4.3.5  *Simultaneous effect of pounding and SSI on unseating potential*

In order to investigate the potential for girder unseating, the normalised maximum relative opening displacements of the adjacent girders with and without pounding, under spatially varying ground motions are presented in Figure 4.11. For the results of Figure 4.11, SSI effects are considered. The differences between these normalised average...
4. Effect of soil flexibility on bridge pounding

maximum responses, with and without pounding, are summarised in Table 4.2, with positive values indicating increased the relative opening displacement due to pounding.

Table 4.2 shows that pounding, for most cases, increased the relative opening displacements between the adjacent girders. There was one exception, and this was between segments 1 and 2 under the ground motions of the soft soil condition. Between segments 2 and 3, the increase of the relative opening displacement due to the excitations of the soft soil condition, in terms of percentage of the reference displacements (shown in Table 4.1) is most significant, while those due to the excitations of the strong rock condition is the least significant. However, such trends are not found between segments 1 and 2. In fact, the ground motions of the strong rock condition caused the larger increase in the maximum normalised relative opening displacement between segments 1 and 2. In most cases, pounding increased the unseating potential, when both SSI and spatially varying ground motions were considered. However, pounding affected the bridge response differently, between different adjacent girders. Segments 2 and 3 tended to suffer the effect more than segments 1 and 2.

From Figure 4.11, it can also be observed that regardless of soil conditions, the relative opening displacements between segments 1 and 2 have a greater maximum value than those between segments 2 and 3. In the case of the excitations of the strong rock condition, the unseating potential of segments 1 and 2 can be as much as twice that of segments 2 and 3, when pounding effects are neglected.

Therefore, according to the results shown in Figure 4.10(c) and 4.11, when designing the seating length to include both pounding and SSI effects, for highly correlated ground motions, the maximum relative opening displacement between the first two segments should be adopted for all of the other bridge joints.

Table 4.2 Consequence of excitation characteristics, SSI, and pounding for normalised relative opening displacements

<table>
<thead>
<tr>
<th>$u_{\text{pounding}} - u_{\text{nonpounding}}$</th>
<th>Ground motion soil condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soft soil</td>
</tr>
<tr>
<td>Segments 1 and 2</td>
<td>-0.06</td>
</tr>
<tr>
<td>Segments 2 and 3</td>
<td>0.21</td>
</tr>
</tbody>
</table>
4. Effect of soil flexibility on bridge pounding

Figure 4.11 Consequence of pounding and SSI for the normalised relative opening displacements between different segments, for (a) soft soil, (b) shallow soil and (c) strong rock conditions

4.4 Summary

This chapter addressed the response of three bridge models with almost identical properties, to account for the spatial variation of ground motions, and soil-structure interaction (SSI). Pounding effects were also considered. The spatially varying ground motions were stochastically simulated and based on the New Zealand design spectra for the soft soil, shallow soil and strong rock conditions. Coherency loss effects of the
spatially varying ground motions were studied by considering high correlation for all three soil conditions, as well as intermediate and weak correlations for the ground motions of the soft soil condition. The influence of the flexibility of the proposed rubber sandboxes was quantified by comparing the displacements of the rubber box and the shake table.

In order to understand the effects of SSI, the bridge models with both fixed base and SSI conditions, were separately subjected to the same ground excitations. The relative opening girder displacements, pounding forces, and bending moments at the pier bases were measured and normalised by the corresponding reference responses obtained for each soil condition without pounding. In total, 200 experiments considering SSI and another 200 with fixed base supports were conducted. By comparing the normalised results obtained from these experiments, the following conclusions are drawn:

1. If ground motions of strong rock condition are anticipated, and pounding effects excluded, SSI is likely to increase the maximum girder displacement compared to fixed base case.

2. In most of the cases considered, the combined effect of SSI and pounding is shown to increase both the maximum pier bending moment and the maximum relative opening displacements.

3. Compared to the fixed base case, SSI results in smaller maximum pounding forces.

4. In the cases of weakly correlated ground motions, SSI increases the maximum bending moment and reduces the maximum pounding force more significantly, compared to that due to higher correlated ground motions. However, SSI under highly correlated ground motions tends to have a largest increase in the maximum relative opening displacement.

5. Pounding can increase unseating potential.
Chapter 5

Effect of excitation spatial variation on inelastic bridge response

5.1 Introduction

This chapter experimentally studied the inelastic response of a bridge with two identical segments subjected to spatially varying ground motions including pounding. The reason for this is because to the author’s best knowledge, there have not been any reported experimental studies of the ground motion spatial variation effect on inelastic bridge behaviour.

5.2 Experiment description

5.2.1 Prototype structure and model

The prototype bridge structure consists of two identical segments, each comprising a deck with a length of 100 m, and two piers with a height of 21.5 m. It is assumed that each of the bridge segments experiences the ground excitation $a_{go}(t)$ acting at the middle of the bridge segment, where $o$ is the number of the considered sites ($o = 1$ and 2). Figure 5.1 shows the prototype structure considered. Table 5.1 provides more information about the prototype structure.
5. Effect of pounding on bridge inelastic response

### Table 5.1 Prototype parameters

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<td>Fundamental frequency</td>
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</tbody>
</table>

![Prototype structure](image)

**Figure 5.1** Prototype structure

To simulate the plastic deformation of the piers, the models used in previous chapters (Figure 3.2) were modified by constructing artificial plastic hinges at the base support and the pier-girder joint. Figure 5.2 shows the locations of the artificial plastic hinges. Figure 5.3 depicts one plastic hinge. Rotational slippage of the hinges (the plastic hinge development) is triggered when the overturning moment exceeds the moment capacity of hinge. Such artificial plastic hinges were originally used in the work by Qin et al. (2013). A torque of 2.5 Nm was applied to each hinge using a digital torque wrench (ACDelco ARM601-3). According to the pushover analysis of the whole structure, an overturning moment of 5.0 Nm will initiate the rotation of the artificial plastic hinges. To ensure that the plastic hinge conditions are reproducible, i.e. the surfaces of the steel plates would not be worn out due to friction, Teflon washers were placed between the steel plates (see Figure 5.3 (c)).

![Modified model with artificial plastic hinges](image)

**Figure 5.2** Modified model with artificial plastic hinges

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5. Effect of pounding on bridge inelastic response

Figure 5.3 Plastic hinge: (a) Drawing, (b) front view and (c) side view with Teflon washers

5.2.2 Testing and spatially varying ground motions

Two shake tables were utilised to apply the earthquake motions to the models. A draw wire displacement transducer was used to measure the relative girder displacement. The girder displacements, i.e. the relative displacement between the girder and the base for each segment were measured by linear variable differential transformers. Strain gauges were glued to one side of each pier at the base to measure the bending moment at the pier support. Pounding forces were measured by the measuring heads. The setup of the experiments is shown in Figure 5.4.

Figure 5.4 Experimental setup

For this experiment, only fixed base foundation was considered. SSI effects were not incorporated. Spatially varying ground motions were simulated based on the soft soil condition (Class D) from the New Zealand design spectra. To incorporate the effect of
coherency loss, the spatially varying ground motions were further categorised with the high, intermediate and weak correlations. For each coherency loss, 10 sets of excitations were simulated, resulting in a total of 30 sets of stochastically independent spatially varying ground motions. A peak ground acceleration of 1 g was targeted. More details about the model and the spatially varying ground motions are provided in Section 3.2.

5.3 Results and discussion

5.3.1 Plastic hinge development

Figure 5.5 shows the consequences of the plastic hinge development for the rotation and the bending moment at pier B support and the corresponding “girder displacement” in comparison with that of elastic response. The “girder displacement” refers to the displacement of the girder relative to the base (termed hereafter “girder displacement”). Pounding was not considered. The positive girder displacement and bending moment imply that the girder moved to the right relative to the base. As anticipated in Figure 5.5(a), the rotational slippage was triggered when the bending moment at the pier B support reached the column moment capacity of 2.5 Nm.

There are two times of slippage in Figure 5.5(a). The first hinge rotation took place when the girder displacement reached 8.5 mm, which occurred after 1.7 s (see Figure 5.5(b)). The second hinge rotation took place at 4.7 s. Finally, a residual displacement of 6.1 mm was observed, where the residual displacement refers to the overall permanent displacement of the girder relative to its base.

![Figure 5.5](image_url) (a) Moment-rotation relation due to ground motion (set 7), and (b) the corresponding girder displacement of segment 1 without pounding
5. Effect of pounding on bridge inelastic response

5.3.2 Effect of spatially varying ground motions without pounding

To investigate the influence of spatially varying ground motions on the plastic hinge development, the results obtained from the spatially varying ground motions were compared with those due to the uniform ground motions. To limit the number of influence factors, pounding was excluded. Figure 5.6 shows the development of the bending moment at the pier D support due to the uniform and the spatially varying ground motions. The time lag between the solid and dashed lines is attributable to the wave passage effect. The occurrences of rotational slippage of the hinge were marked by circles. Given that the triggering moment of the hinge rotation is 2.5 Nm, it was noted that two hinge rotations occurred under the uniform ground motion. The first one occurred at 2.3 s, initiated by a positive activated bending moment of 2.55 Nm, followed by another rotation at 2.6 s in the opposite direction due to a negative activated bending moment of 2.75 Nm. The difference in both activated bending moments resulted in unequal hinge rotations in the opposite directions, leading to a residual bending moment of -0.14 Nm (a permanent girder displacement to the left). On the contrary, the spatially varying ground motion resulted in only one hinge rotation, which took place at 2.7 s, resulting in a residual bending moment of -0.28 Nm. Larger residual bending moment corresponds to larger residual girder displacement. The results show that uniform ground motion can overestimate the number of hinge rotations, resulting in underestimation of the residual bending moment and residual girder displacement.

![Figure 5.6](image-url)  
**Figure 5.6** Effect of plastic hinge development on bending moment at base of pier D due to uniform and highly correlated ground motion (set 2) without pounding
5. Effect of pounding on bridge inelastic response

Figure 5.7 shows the consequences of the spatially varying ground motions for the inelastic responses of segment 2. The values in brackets are the average of the 10 absolute residual displacements. In Figure 5.7(a), there are four spatially varying ground motions, i.e. sets 2, 6, 8 and 9, which resulted in larger absolute residual displacement than those obtained from the uniform ground motions. By comparing the averages, the uniform ground motions caused greater residual displacements than the spatially varying ground motions. In Figure 5.7(b), the average maximum girder displacement of 8.92 mm due to the uniform ground motions is only slightly larger than that due to the spatially varying ground motions of 8.61 mm. There are only three spatially varying ground motions, i.e. sets 5, 8 and 9, which resulted in larger maximum girder displacements than those obtained from neglecting ground motion spatial variation. The overall largest residual displacement of 9.58 mm occurred when ground motion set 9 with spatial variation was considered. Furthermore, spatially varying ground motion set 9 also caused the overall largest girder displacement of 12.72 mm. The uniform ground motion can lead to larger maximum girder displacement and the residual displacement if comparing only the average values.

**Figure 5.7** Consequence of highly correlated spatial variation of ground motions for (a) the absolute residual displacements and (b) the maximum absolute girder displacements of segment 2 without pounding
5. Effect of pounding on bridge inelastic response

The results reveal that spatial variation of ground motions can influence the plastic hinge development even without pounding, inducing different hinge rotations, even if the ground motions applied for the two segments were highly correlated. Assuming uniform ground motions can overestimate the maximum girder and residual displacement. More importantly, assuming uniform ground motions will underestimate the relative displacement (not presented) because identical segments move identically and hence a zero relative displacement is expected. Therefore, for a more realistic analysis of inelastic bridge response, spatial variation of ground motions should be considered.

5.3.3 Effect of spatially varying ground motions with pounding

Under spatially varying ground motions, relative displacements between two identical bridge segments will inevitably occur, resulting in pounding if the gap is not sufficient to accommodate the relative closing displacement. The consequence of spatially varying ground motions for the inelastic bridge response including pounding is demonstrated in Figure 5.8. The number in bracket is the average response due to the ten ground motions. In Figure 5.8(a), an average absolute residual displacement of 2.86 mm was obtained from considering uniform ground motions whereas a smaller average of 2.29 mm due to the spatially varying ground motions was found. Among the ten spatially varying ground motions, only sets 7 and 8 caused greater residual displacements than the uniform ground motions. In Figure 5.8(b), the spatially varying ground motions resulted in a slightly larger average maximum absolute girder displacement than the uniform ground motions (comparing 8.92 mm with 9.40 mm).
5. Effect of pounding on bridge inelastic response

Figure 5.8 Consequence of highly correlated spatially varying ground motions on (a) the absolute residual displacement and (b) the maximum absolute girder displacement of segment 1 with pounding

Figure 5.9 demonstrates the simultaneous effect of spatially varying ground motion and pounding on the maximum absolute girder displacement of segment 2. According to the average maximum girder displacements, the uniform ground motions are more likely to cause larger girder displacement than the spatially varying ground motions. The combined effect of spatially varying ground motions and the pounding resulted in an average maximum girder displacement of 8.30 mm. A comparison of the results in Figure 5.9 and Figure 5.7(b) shows that allowing pounding increased the average maximum girder displacement of segment 2. For example, pounding increased the maximum girder displacement due to set 4 ground motion with spatial variation from 8.30 mm in Figure 5.7(b) to 9.45 mm (in Figure 5.9). In contrast, without pounding, ground motion set 9 considering spatial variation caused the overall maximum girder displacement among all the ground motions considered, i.e. 12.72 mm (see Figure 5.7(b)). When pounding was included, the maximum girder displacement resulting from the same ground motion decreased to 8.38 mm. Overall, the observations imply that the pounding induced by the spatially varying ground motions may interact with the plastic hinge development, resulting in more girder displacement than without pounding.
5. Effect of pounding on bridge inelastic response

Figure 5.9 Consequence of highly correlated spatially varying ground motions for the maximum absolute girder displacement of segment 2 with pounding

5.3.4 Effect of pounding on plastic hinge development and unseating potential

Based on the results shown in Figure 5.8 and Figure 5.9, it was found that pounding can influence the plastic hinge development. In order to understand how pounding contributes to the plastic hinge development, in Figure 5.10, the bending moment at the pier B support was plotted against the hinge rotation due to highly correlated ground motion with and without pounding. From Figure 5.10(a) and (b), it can be seen that when pounding was excluded, the hinges of pier B rotated anticlockwise twice, i.e. the girder moved to the left relative to the ground. With pounding, three hinge rotations in anticlockwise direction were observed. It can be concluded that pounding may lead to more hinge rotations. To understand the plastic hinge development, the girder displacements of segment 1 with and without pounding due to ground motion set 7, in conjunction with the corresponding pounding force time history for the case with pounding were depicted in Figure 5.11. The positive girder displacement implies that the girder moved to the right relative to the base.

From Figure 5.11(a), the segment had anticlockwise hinge rotations as the girder displacements were mainly negative. The first hinge rotation for both cases with and without pounding was developed at 1.74 s due to an activated girder displacement of nearly 9 mm, causing a negative rotation of approximately 0.006 radians. The negative hinge rotation corresponds to a permanent displacement of the segment to the left. When pounding was considered, an impact of 6 N was developed during the first hinge rotation (see Figure 5.11(b)), resulting in a hinge rotation of approximately -0.01 radians (see
5. Effect of pounding on bridge inelastic response

Figure 5.10(b)). The second hinge rotation for both cases with and without pounding was triggered at the same time, i.e. at 4.67 s. Pounding did not contribute to the second hinge rotation as no impact was induced during the plastic hinge development. After these two hinge rotations, the total permanent rotations for the cases with and without pounding were -0.016 and -0.012, respectively – an additional rotation of 0.004 radians attributed to pounding. This increased total hinge rotation induced more bending moment at the pier B support. The increased bending moment, together with an impact of 15.5 N at 6.02 s, triggered the third plastic hinge rotation. As it can be seen in Figure 5.11(a), this hinge rotation resulted in the maximum girder displacement of -12.07 mm. Pounding led to more hinge rotations and larger gap between the two segments. The increased gap contributed to the insignificant pounding after 6 s. Overall, the hinges rotated 0.024 radians anticlockwise in the case with pounding while only 0.012 radians in the same direction in the case without pounding. Hence a larger residual displacement due to pounding was observed in Figure 5.11(a).

From Figure 5.10 and Figure 5.11, it can be seen that pounding can contribute to plastic hinge development, resulting in larger permanent displacements of the girders. However, the overall consequence of spatially varying ground motion with pounding for the plastic hinge development is complex as it is much affected by the interaction of many factors, including the ground motion and dynamic characteristics of the bridge segments. Therefore, the reduced responses were also observed in Figure 5.8 and Figure 5.9 when pounding was included.

![Figure 5.10](image-url)  
**Figure 5.10** Moment-rotation relation at pier B support due to highly correlated ground motion (set 7) (a) without and (b) with pounding
5. Effect of pounding on bridge inelastic response

Figure 5.11 (a) Consequence of pounding for the girder displacement of segment 1, and (b) pounding force due to the highly correlated ground motion (set 7)

Figure 5.12 compares the maximum relative opening displacements corresponding to the ten weakly correlated excitations for the cases with and without pounding. For the non-pounding case, the segments were given an initial separation sufficient to avoid subsequent pounding. Note that the relative opening displacement was measured only for comparisons with the pounding case to investigate the contribution of pounding to unseating potential. In real life, it does not have practical meaning. By comparing the average maximum relative opening displacements in the brackets, without pounding, the ten ground motions resulted in an average of 8.14 mm. When pounding was involved, the average maximum relative displacement was increased by about 17%, to 9.54 mm. Among the ten weakly correlated ground motions, six appeared to make the bridge suffer from larger relative opening displacements due to impacts, i.e. sets 4, 5, 6, 7, 8 and 10. The results are consistent with the finding of the study (Abdel Raheem, 2009). Chapter 3 which was based on elastic structures also revealed that pounding can increase the maximum relative opening displacement between the girders. For a bridge with plastic hinges, pounding increases the relative opening displacement by enhancing the hinge
rotations of segments 1 and 2 in opposite directions, resulting in more permanent displacement. Hence larger relative opening displacement may take place.

![Figure 5.12](image)

**Figure 5.12** Consequence of pounding and plastic hinge development on the maximum relative opening displacement of the bridge segments under the weakly correlated ground motions

5.3.5 *Effect of coherency loss on inelastic responses*

To investigate the influence of coherency loss on the inelastic bridge response, the responses of the bridge model due to ground motions with high, intermediate and weak correlation were compared in Figure 5.13. Pounding was included. The numbers in brackets are the averages of the ten responses. For all the responses considered, the averages due to the weakly correlated spatially varying ground motion were distinguishingly larger than those due to the other coherence cases. In Figure 5.13(a), the average maximum relative opening displacement due to the weakly correlated ground motions, i.e. 9.54 mm was 25% and 27% larger than 7.62 mm and 7.52 mm due to the intermediately and highly correlated ground motions, respectively. Similarly, the weak correlation of ground motions caused an average maximum pounding force of 30 N, which is 25% larger than the 24 N due to the intermediately correlated ground motions, and 31% more than the 23 N due to highly correlated ground motions.

These figures also show that the average response of the model subjected to the intermediately correlated ground motions was slightly larger than that due to the highly correlated ground motions. Almost all the overall maximum responses were caused by the weakly correlated ground motions, i.e. the overall maximum relative displacement of 13.74 mm (caused by ground motion set 10), the overall maximum pounding force of
5. Effect of pounding on bridge inelastic response

40.68 N (caused by ground motion set 3), and the overall maximum girder displacement of 13.2 mm of segment 2 (caused by ground motion set 9). The results show that coherency loss of the ground motions at adjacent supports can have significant influence on the relative response and pounding force between segments. The weaker the coherence of the ground motions, the more likely the bridge will suffer more damage.

Figure 5.13 Consequence of coherency loss for maximum values of (a) relative displacement, (b) pounding force and (c) absolute girder displacement of segment 2

5.3.6 Effect of plastic hinge development on possible pounding-induced damage and unseating potential

To understand the effect of plastic hinge development on the damage potential due to pounding and unseating, the elastic bridge response was also considered. Figure 5.14
5. Effect of pounding on bridge inelastic response

compares the maximum relative opening displacements and the maximum pounding forces between the cases with and without plastic hinges. To ensure elastic bridge response was achieved, the artificial hinges were applied with a sufficiently large torque so that they are unable to rotate.

For both responses, considering plastic hinges led to smaller averages. The maximum relative opening displacement decreased with plastic hinge development for all the weakly correlated ground motions. Under ground motion set 6, considering plastic hinges reduced the maximum relative opening displacement by almost 32%. Similarly, in Figure 5.14(b), all the maximum pounding forces with plastic hinges were smaller than those without except that due to ground motion set 2 (only 2% larger). On average, allowing plastic hinge development decreased the maximum pounding force by 10%.

![Graph showing the effect of plastic hinges on maximum relative opening displacement and pounding force](image)

**Figure 5.14** Consequence of plastic hinge for maximum values of (a) relative opening displacement and (b) pounding force under weakly correlated ground motions

Figure 5.15 shows the differences between the maximum relative opening displacements and maximum pounding forces obtained from cases with and without plastic hinges due to the ground motions of different coherence. Positive values indicate that considering plastic hinges resulted in larger responses than considering elastic structures. The averages of the differences corresponding to the 10 ground motions of different coherence
are shown in brackets. The positive averages in the brackets indicate that allowing plastic hinge development tends to result in smaller maximum relative opening displacement and smaller pounding force regardless of the ground motion coherence. The weakly correlated ground motions would decrease the responses most significantly compared to highly and intermediately correlated ground motions. Given the average responses with plastic hinges shown in Figure 5.13(a) and (b), the reductions of the maximum pounding force due to ground motions of highly, intermediately and weakly correlated ground motions were 8%, 8% and 11%, respectively, while the reductions of the maximum relative opening displacement were 15%, 11% and 22%, respectively. Therefore, allowing plastic hinge development can reduce the maximum pounding force and maximum relative opening displacement when spatially varying ground motions are anticipated, especially for weakly correlated ground motions.

**Figure 5.15** Consequence of plastic hinge development and coherency loss for the maximum values of (a) relative opening displacement and (b) pounding forces

Another aspect of pounding damage potential arises from the number of the events of significant pounding. Figure 5.16 indicates the number of significant impacts due to intermediately correlated ground motion with and without plastic hinges. The impacts are considered significant if the magnitude is greater than 12 N, i.e. half of the average maximum pounding force, 24.07 N due to the intermediately correlated ground motions.
with plastic hinges (see Figure 5.13(b)). It is noted that under all the ground motions, in the case with plastic hinges, the bridge resulted in fewer number of significant impacts than the case without. On average, allowing plastic hinge development reduced the significant impact from 10.6 times to 7.6 times.

![Figure 5.16](image)

**Figure 5.16** Consequence of plastic hinge development for the number of impact larger than 12 N under intermediately correlated ground motions

According to Figure 5.15(b) and Figure 5.16, plastic hinge development can reduce possible pounding-induced damage. In the considered cases, it is mainly caused by two reasons: (1) Plastic hinge development reduced the total energy in the system, causing smaller impacts and (2) the development of the plastic hinges may result in a large gap, which prevents the development of significant pounding.

### 5.4 Summary

The effect of spatial variation of ground motions on the inelastic response of a two-segment bridge model with pounding was experimentally studied using shake tables in this chapter. Artificial plastic hinges were constructed and installed at the base supports and the pier-girder joints. The bridge segments were subjected to stochastically simulated ground motions based on the response spectrum of the New Zealand earthquake for soft soil condition. Different coherency of the spatially varying ground motions, i.e. high, intermediate and weak were considered. To understand the effect of the plastic hinges, the elastic bridge response was used for a comparison. A total of 120 tests were performed. Abutments and soil-structure interaction were not considered.
5. Effect of pounding on bridge inelastic response

This chapter concludes that:

1. Spatial variation of ground motion can influence the plastic hinge development of the bridge structures. The residual rotation and the maximum girder displacement can increase or decrease as a result of plastic hinge development depending on the ground motion considered.

2. With pounding, the number of plastic hinge rotation may increase.

3. When plastic hinges are considered, the weakly correlated ground motions tend to result in the most significant relative opening displacement, pounding force and girder displacement, while highly correlated ground motions showed the least response. On average, the maximum relative opening displacement and the maximum pounding force resulting from the weakly correlated ground motion can be 27% and 31% larger than those resulting from the highly correlated ground motion.

4. Pounding can contribute to the girder unseating when plastic hinge development is allowed. In the cases considered, pounding caused 17% increase in the maximum relative opening displacement. However, further investigations including abutment effect and multiple segment pounding are necessary to clarify the consequence of overall system performance.

5. Plastic hinge development can contribute to reducing the maximum relative opening displacement and the maximum pounding force. Assuming elastic bridge response may overestimate the possible pounding-induced damage and unseating potential.
Chapter 6

Effect of abutment excitation on bridge pounding

Related paper:


6.1 Introduction

Bridge damage due to pounding at girder-abutment joints has been observed in many 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. In other words, abutments were assumed to be fixed in position at all time. Therefore the objective of this chapter was to experimentally study the influence of abutment movement on the pounding behaviour of a system comprising a single segment bridge and its abutments using three shake tables. Movable abutments are rigidly connected to the ground and move with the ground as rigid bodies. 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.
6. Effect of abutment excitation on bridge pounding

6.2 Experimental model

6.2.1 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 6.1). To limit the influence factors, it was assumed that the system experienced ground excitation $a_{go}$ along the span direction only, where $o$ was the number of the considered sites ($o = 1, 2$ and $3$). The distance from the centre of the girder to the centre of each abutment was assumed to be $50$ m. 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 3.2.

![Figure 6.1 Considered prototype system](image)

6.2.2 Pounding and measuring head

Pounding forces at the joint of the bridge structure and the abutment were measured by a measuring head fabricated using a force sensor made by a strain gauge bonded to the back of a highly elastic spring steel strip with cross-section dimensions of $10$ mm $\times$ $1$ mm. Two types of measuring head, i.e. 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 magnitudes of the contact stiffness for the stiff and soft measuring heads were found to be $244$ N/mm and $30.5$ N/mm, respectively. The pounding heads were made of a PVC cylinder with a diameter of $25$ mm in order to allow even contact with the force sensor. Each measuring head was fixed to a rigid steel frame.
6. Effect of abutment excitation on bridge pounding

to measure the pounding force generated at each side of the girder. Pounding heads were bonded into both ends of the girder. Detailed dimensions of the measuring heads are shown in Figure 6.2.

Figure 6.2 Schematic drawing of the measuring heads

6.2.3 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 ± 2 g were fixed to the girder and the shake table platforms.

In this chapter, only fixed base supports to the piers were considered, and the SSI effect was excluded. Fundamental frequencies ranging from 1.4 Hz to 2.6 Hz with an increment of 0.2 Hz 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 1.4 Hz to 2.6 Hz.

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
6. Effect of abutment excitation on bridge pounding

Abutments were subjected to spatially varying ground motions. 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 6.3.

![Figure 6.3 Setup of the testing system](image)

6.2.4 Spatial variation of ground motions

The spatially varying ground motions were simulated based on New Zealand design spectra (NZS 1170.5, 2004). The soil conditions categorised as Class A (strong rock), Class C (shallow soil site) and Class D (soft soil site) were considered. The excitation correlation of each soil condition for adjacent sites was the same as in Section 3.2.3. The simulation of spatially varying ground motions was achieved using the same method described in Section 3.2.4, except $d_j$ is 50 m. Figure 6.4 shows one set of the simulated ground motion time histories with peak ground acceleration of 1.0 g. Figure 6.5 illustrates the difference in the coherency loss of the simulated ground motions and the empirical coherency loss function.
6. Effect of abutment excitation on bridge pounding

Figure 6.4 Simulated ground: (a) accelerations and (b) displacements for site 1 to 3 with a 50 m site distance

Figure 6.5 Coherency loss of simulated highly correlated ground motions with a distance of 50 m and model coherency loss function

6.3 Results and discussion

The results were obtained from tests involving a bridge-abutment system with and without pounding. The bridge model considered had different fundamental frequencies as
given in Table 6.1. 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, and denoted as “stiff spring” and “soft spring” with bending stiffnesses at the girder-abutment contact location of 244 N/mm and 30.5 N/mm respectively. The discussion focuses mainly on the “stiff spring” abutments as their stiffness after back-scaling, i.e. 3 MN/mm, is more realistic for a prototype bridge compared with the “soft spring”. The cases considered are summarised in Table 6.1. 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 6.1 Summary of the parameters considered for the tests**

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<tr>
<td></td>
<td></td>
<td>Strong rock</td>
<td></td>
</tr>
<tr>
<td>Movable abutment</td>
<td></td>
<td>Soft soil</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.4 ~ 2.6 Hz</td>
<td>Stiff and Soft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soft soil</td>
<td>Highly</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shallow soil</td>
<td>Intermediately</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strong rock</td>
<td>Weakly</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shallow soil</td>
<td>Highly</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strong rock</td>
<td>Highly</td>
<td></td>
</tr>
<tr>
<td>Uniform excitations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.4 ~ 2.6 Hz</td>
<td>Stiff and Soft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soft soil</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shallow soil</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strong rock</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

*N/A: Not applicable

In order to compare the structural responses, i.e. bending moment at pier support, relative opening displacements at girder ends and pounding forces for different fundamental frequencies, the actual responses were normalised by the reference values (given in Table 6.2), which are respectively the averages of the 20 maximum bending moments, the
6. Effect of abutment excitation on bridge pounding

maximum displacements of the bridge girder and the base shear due to the 20 corresponding ground motions without pounding. The reference values reflect the varying seismic loading magnitudes of different bridge structures (unequal fundamental frequencies) corresponding to the New Zealand design spectra for the considered soil conditions. By normalising the absolute responses using the reference values due to the same excitations without pounding effect, the results are then comparable. Among the average maximum responses, the greater value at either side of the bridge was chosen to obtain the most conservative responses. The ratios of the normalised results are only valid for the cases considered in this paper or for very similar cases.

Table 6.2 Summary of the reference responses

<table>
<thead>
<tr>
<th>Fundamental frequency (Hz)</th>
<th>Reference response</th>
<th>Soft soil</th>
<th>Shallow soil</th>
<th>Strong rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Absolute displacement (mm) / Bending moment (Nm) / Base shear (N)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.4</td>
<td>4.12 / 0.731 / 5.90</td>
<td>2.91 / 0.502 / 4.05</td>
<td>2.13 / 0.376 / 3.03</td>
<td></td>
</tr>
<tr>
<td>1.6</td>
<td>4.15 / 0.743 / 5.99</td>
<td>2.93 / 0.503 / 4.06</td>
<td>2.22 / 0.379 / 3.06</td>
<td></td>
</tr>
<tr>
<td>1.8</td>
<td>4.17 / 0.781 / 6.30</td>
<td>2.99 / 0.516 / 4.16</td>
<td>2.27 / 0.404 / 3.26</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>4.18 / 0.800 / 6.45</td>
<td>3.03 / 0.524 / 4.23</td>
<td>2.30 / 0.417 / 3.36</td>
<td></td>
</tr>
<tr>
<td>2.2</td>
<td>4.20 / 0.767 / 6.19</td>
<td>3.09 / 0.541 / 4.36</td>
<td>2.29 / 0.404 / 3.26</td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>4.30 / 0.833 / 6.71</td>
<td>3.31 / 0.595 / 4.80</td>
<td>2.48 / 0.440 / 3.55</td>
<td></td>
</tr>
<tr>
<td>2.6</td>
<td>4.30 / 0.856 / 6.90</td>
<td>3.33 / 0.617 / 4.98</td>
<td>2.50 / 0.452 / 3.65</td>
<td></td>
</tr>
</tbody>
</table>

6.3.1 Effect of abutment movement

During an earthquake, abutments will inevitably move when their surrounding soil mass moves, meaning that “fixed abutment” assumptions considered by earlier researchers cannot normally arise. Owing to the complexity in incorporating all the influence factors, previous studies, for simplicity, often assumed the irrational fixed abutments condition and neglected the abutment excitation by the ground movements during an earthquake. Thus, only the effect on bridge structures was considered in those simplified studies. In this chapter, it is only for comparison purposes that this fixed abutment case was considered. To investigate the consequences of abutment movements for pounding force 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 6.6 Consequence of movable abutments for the average maximum normalised (a) bending moment, (b) relative opening displacement, and (c) pounding force

Figure 6.6 compares the average maximum structural responses with fixed and movable abutments under uniform and spatially varying ground motions with zero initial gaps. Figure 6.6(a) shows the maximum bending moment at pier A support normalised by the reference values given in Table 6.2. As shown under the soft soil highly correlated ground motions, in all cases the greatest support bending moments occurred when fixed abutments were assumed, and the average normalised maximum bending moments at pier supports appeared to have very similar values, i.e. 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 had a zero value which disabled the girder movement (apart from small deformations of the stiff pounding heads), the bending of the bridge was 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
subtle difference between different frequencies was caused solely by the different small movements of the bridge deck under the different site 2 ground motions that are stochastically independently simulated for each case considered. With all the normalised bending moments less than 1, Figure 6.6(a) also implies that if the responses of abutments are considered, the bending moment is always less than the case with fixed abutments assumed. This is because with abutments responding to ground movements, the relative movement between girder and the pier support became less because of the 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 was not rigidly blocked as in the case of non-moving abutments, the pier sway was still impeded by the moving abutments. Consequently, the bending moments obtained without an abutment constraint are generally larger than the case when abutments are considered.

As shown in Figure 6.6(b), when the two abutments were fixed, the relative displacement between abutment and bridge span was very small, equal to the compressive deformation of the pounding head. When the response of the abutments was considered, between the girder and the abutment A the uniform ground motions resulted in a normalised average maximum relative opening displacement of 1.0 at 1.4 Hz, corresponding to 4.12 mm, indicating that there was still some significant out-of-phase movement 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 6.6(b) is that under spatially varying ground excitations the normalised relative opening displacements were 1.16 and 0.95 for bridge structures with the fundamental frequencies of 1.4 Hz and 2.6 Hz, corresponding to 4.78 mm and 4.08 mm, respectively. However, uniform excitations resulted in 4.12 mm and 1.76 mm relative opening displacements for structures with the corresponding frequencies. Therefore, considering non-uniform ground motions led to greater separation between the segment and the abutment (unseating potential of the bridge girder) than spatially uniform ground motions. Figure 6.6(c) shows that the non-uniformity of ground motion also caused larger normalised impact forces compared to uniform ground motions for all the cases considered. The average maximum normalised pounding forces due to both uniform and non-uniform ground motions decreased with an
increase in the bridge fundamental frequency. Further explanation for this observation is referred to Figure 6.12.

Designers should consider the mobility of abutments and spatial variation of ground motions when designing bridges. 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 to the integrity of bridge design, the following discussion focuses on the effect of abutments engaging in spatially varying ground motions.

6.3.2 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 were applied to the abutment-bridge system with zero initial gaps. Figure 6.7 shows the influence of different soil condition ground motions on structural responses normalised by the reference values given in Table 6.2. A stiff contact stiffness and highly correlated ground motions were considered.

As shown in Figure 6.7, almost all the normalised responses decreased with an increase in the bridge fundamental frequency. The greatest normalised bending moment occurred at the fundamental frequency of 1.4 Hz due to soft soil condition excitation, i.e. 0.839, corresponding to 613 Nmm. For shallow soil and strong rock site conditions, the maximum normalised bending moments were found to be 0.770 and 0.809, corresponding to 387 Nmm and 305 Nmm, respectively. Soft soil condition also caused greater average maximum relative opening displacement and pounding force for all the systems considered. For the bridge with fundamental frequency of 1.4 Hz, the relative opening displacement and the pounding force resulting from soft soil condition excitation were respectively 4.82 mm and 78.6 N, approximately double those resulting from strong rock conditions, i.e. 2.3 mm and 45.3 N, respectively. When pounding is considered, soft soil condition excitations generally lead to the largest structural responses over shallow soil and strong rock.
6. Effect of abutment excitation on bridge pounding

Figure 6.7 Consequence of different soil conditions for the average maximum normalised
(a) bending moment of pier A, (b) relative opening displacement, and (c) pounding force
between bridge abutment A and girder

6.3.3 Effect of ground motion variation due to coherency loss

Figure 6.8 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 6.8(a), ground motions with weak correlation resulted
in slightly larger normalised bending moment at the support of bridge pier A than
intermediately and highly correlated excitations. However, the differences tended to
decrease as the fundamental frequency of the bridge structure increases. The largest
difference between the normalised bending moment among different correlations of
ground motion occurred at 1.4 Hz (between weakly correlated and intermediately
correlated excitations), being only 10%. The differences in all the structural responses due
6. Effect of abutment excitation on bridge pounding

to excitation incoherency effect became less obvious, except for 2.2 Hz, 2.4 Hz and 2.6 Hz (see Figure 6.8(b)), where the normalised maximum relative opening displacements between abutment A and the girder due to weakly correlated excitations were slightly smaller than those due to highly and intermediately correlated excitations.

Figure 6.8 Consequence of coherency loss for the average maximum normalised (a) bending moment of pier A, (b) relative opening displacement, and (c) pounding force between bridge abutment A and girder

Figure 6.8(c) compares the normalised 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 were very similar. Therefore, when the site distance is 50 m or less, for simplicity, the coherency loss effect of non-uniform excitation can be neglected and for a conservative consideration of pounding damage, highly correlated
6. Effect of abutment excitation on bridge pounding spatially varying ground motions can be assumed. However, further investigations are recommended.

6.3.4 Effect of contact stiffness

In this research, different contact stiffness at the abutments-girder interface was considered by using two types of measuring heads with stiff and soft springs, respectively. The contact stiffnesses considered are given in Table 6.1. Figure 6.9 illustrates the effect of the contact stiffness on structural responses. The pounding force obtained with a “stiff spring” in Figure 6.9(a) exceeded that resulting from “soft spring” by a significant amount for all the fundamental frequencies, especially for 1.4 Hz and 1.6 Hz, where the differences are almost doubled. Larger normalised relative opening displacements were also obtained when “stiff spring” was considered as shown in Figure 6.9(b). The observations show that the spring stiffness effect remained consistent for different bridges.

![Figure 6.9](image)

**Figure 6.9** Consequence of contact stiffness for the average maximum (a) pounding force and (b) normalised relative opening displacement between the abutment A and the bridge girder

The reason for the observation in Figure 6.9 can be found in Figure 6.10(a) and (b), which show the development of the relative velocity and the pounding force within the time window between 4.2 s to 5.7 s due to soft and stiff springs. Figure 6.10(c) and (d) are zoomed-in plots of the portion surrounded by the dashed rectangle in Figure 6.10(a) and (b) for a clearer presentation of the relations between the relative velocity and the pounding forces developed at the abutment-girder interface. In the relative velocity time histories, positive values represent a separation velocity while negative values an approach velocity. The pounding commenced when the negative relative velocity reached its negative maximum due to contact of the structures (indicated by the dash-dotted line...
6. Effect of abutment excitation on bridge pounding

as shown in Figure 6.10(c) and (d)), and finished when the relative velocity reached its positive maximum. As the contact surfaces deform, kinetic energy was transferred into elastic strain energy. Soon after the pounding force reached a maximum, the abutment and the girder moved 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 6.1), 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 was constant in the cases considered, the pounding force was directly related to the rate of change of the relative velocity, which is indicated by the slope of the relative velocity curves.

\[
\int F(t)dt = \Delta mV
\]  

(6.1)

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 6.2):

\[
m = \frac{m_1 + m_2}{m_1m_2}
\]  

(6.2)

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 6.10 Consequence of contact stiffness for the relative velocity and the pounding force at the abutment-girder interface for the bridge fundamental frequency of 1.4 Hz between the time windows (a) and (b) of 4.2 s and 5.7 s; (c) and (d) of 4.21 s and 4.35 s
6. Effect of abutment excitation on bridge pounding

Figure 6.10(c) shows that the slope of the relative velocity resulting from pounding with the stiff springs was steeper than that with the soft springs. Hence, a larger pounding force was observed in Figure 6.10(d). Compared to the stiff spring, the soft spring was associated with longer contact duration, resulting in smaller rate of change of the relative velocity. Therefore, larger contact stiffness can cause more significant pounding force than those of smaller stiffness. As a result, lowering 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, e.g. by installing a rubber buffer, strong enough to carry heavy traffic loading and flexible enough to deform when the girder impacts.

6.3.5 Effect of variation of bridges

For the conclusions of this study to be more widely applicable, different bridges were included by varying the fundamental frequency of the model. In Figures Figure 6.6 to Figure 6.9, bridges of different fundamental frequencies were considered. Figure 6.11 shows the actual maximum bending moments of pier A, relative opening displacements and pounding forces between abutment A and the girder with fundamental frequencies of 1.4 Hz, 2 Hz and 2.6 Hz. The results were obtained from the 20 highly correlated ground motions simulating soft soil condition. Abutment excitations were considered. The numbers in parentheses are the averages of the 20 values. It is very rare that the fluctuations of the actual bending moments and relative displacements recorded exceed 40% of the average. For the pounding force measurement, the variation is within 15% of the average. Figure 6.11 shows that more flexible bridges would, in general, result in more significant responses if the excitation of the abutments is considered.

By comparing the relative velocity and the normalised pounding force time histories at the abutment A-girder interface between bridge structures with fundamental frequencies of 1.4 Hz and 2 Hz, Figure 6.12(a) and (b) illustrate the case of the bridge with lower fundamental frequency having larger normalised pounding forces than the one with higher fundamental frequency (see also Figure 6.6(c)).
6. Effect of abutment excitation on bridge pounding

For the relative velocity time histories, positive values indicate that the abutment and the bridge structure were separating while negative values indicate that they were approaching. The normalised pounding force time histories for bridge structures with fundamental frequencies of 1.4 Hz and 2 Hz were obtained based on the base shear due to the corresponding sets of ground motions without pounding, i.e. 5.95 N and 6.43 N, respectively. As can be seen at 3.6 s (the circled instant), the maximum approach velocity just before pounding in the case of the 1.4 Hz bridge was 0.042 m/s while the 2 Hz bridge moved at only 0.03 m/s. The separation velocity in the case of the 1.4 Hz bridge structure reached 0.046 m/s immediately after pounding while for 2 Hz, it moved away from
6. Effect of abutment excitation on bridge pounding

abutment A at only 0.032 m/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 1.4 Hz system has larger impact forces than the 2 Hz system. Similar observations can be found at 1.3 s, 2.1 s, 2.5 s, 3.25 s and 4.4 s. It should be noted that the conclusion made is valid only for the cases considered, since the fundamental frequencies can also be altered when the mass is constant while the bending stiffness of the piers varies.

![Graph](image)

**Figure 6.12** Relationship between (a) activated relative velocity and (b) pounding force due to highly correlated soft soil condition excitation and movable abutments

### 6.4 Summary

The effect of the spatial non-uniformity of ground motions and the abutment excitation on the pounding responses of an abutment-span-abutment system with an abutment-bridge span of 50 m was experimentally investigated using three shake tables. The bridge model was closely scaled from the prototype structure based on the similitude laws. Bridges with a fundamental frequency ranging from 1.4 Hz to 2.6 Hz with an increment of 0.2 Hz were 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. The effect of contact stiffness on the structural response was also investigated by using measuring heads with different spring stiffnesses. A total of 3,080 pounding tests and 420 non-pounding experiments were conducted for this study.

This chapter reveals:

1. Movable abutments and spatially varying ground motions generally lead to an increase in relative opening displacement and pounding force in comparison to the fixed abutment cases.

2. In the cases of abutment excitation, the more flexible the bridge structure the stronger the activated pounding force.

3. With pounding, the actual structural responses of the bridge model due to ground motions of soft soil condition are more severe than those of shallow soil or strong rock conditions.

4. Pounding with movable abutments will generally reduce the bending moment of the pier.

5. Abutments with a stiff contact interface lead to greater pounding force and relative opening displacement than a softer contact interface. Installing buffers made of softer materials such as rubber at the possible pounding locations can be an effective mitigation measure for reducing pounding.
Chapter 7

Field test of a bridge segment for modular expansion joint design

7.1 Introduction

Modular expansion joints (MEJ) (Rizza, 1973), consisting of one or more supporting beams spanning between adjacent bridge decks, are designed to accommodate large relative displacements of adjacent bridge segments. This device has a single beam or more supporting a number of centre beams with small gaps between them. An example of an MEJ is shown in Figure 7.1. The sum of these multiple small gaps provides a large gap for relative movement between decks. The gaps between the centre beams are filled with a flexible rubber membrane to ensure water tightness. 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. With a capacity to accommodate relative movements in excess of 2 m (WSDOT, 2011), their potential in seismic design has been proved by experimental research (Spuler, et al., 2011, Clyde and Whittaker, 2000), who found that the modular expansion joint could survive in most earthquakes.

Hao and Chouw (2007) identified the significant factors that influenced the behaviour of MEJs in seismic events; these were spatially varying ground motions, the fundamental frequency ratio of adjacent segments and SSI. Chouw and Hao (2008b) studied the influence of the SSI effect on the required separation distance of two adjacent bridge frames connected by an MEJ. They concluded that using an MEJ with a large gap was likely to completely preclude girder pounding. Today, MEJs have been widely used throughout the world, e.g. Tsing Ma Bridge in Hongkong and Pont Canal de Beauharnois.
Bridge in Canada with maximum relative displacements of 2 m and 1.52 m, respectively (Supler and Gianni, 2004).

Although MEJs have been implemented in new bridges, few experimental studies have addressed the gap design with consideration of spatially varying ground motions and SSI effects. To the author’s best knowledge, experimental investigations on the influence of SSI and spatial varying ground motion on the MEJ gap have not been reported. Therefore, this chapter was to investigate the maximum relative girder displacements by field testing a 1:22 scale bridge model considering spatially varying ground motions.

![Cross-section view of a modular bridge expansion joint system](Chouw and Hao, 2008b)

**Figure 7.1** Cross-section view of a modular bridge expansion joint system (Chouw and Hao, 2008b)

### 7.2 Experimental model

#### 7.2.1 Prototype structure and scale model

The scale bridge model was constructed based on the same prototype structure described in Section 3.2.1. In order to investigate the maximum relative displacement between multiple spans under spatially varying ground motions, a three-segment bridge without abutments was considered (Figure 7.2).
In order to mimic the seismic response of the prototype structure, the construction drawings of the prototype structure were used to determine the geometric and dynamic properties of the prototype structure from which the scale model was derived. In pursuit of the closest representation of the prototype, the fulfilment of the similitude requirements was attempted. Section 3.2.1 provides more information about the similitudes and the scaling method. The scale factors of the fundamental quantities are listed in Table 7.1. The details of the prototype structure and the experimental model are summarised in Table 7.2, where the bending stiffness of the bridge structure was measured as the ratio of the longitudinal force to the longitudinal displacement of one bridge structure at deck level. The compressive strength of the concrete was assumed to be 50 MPa.

**Table 7.1 Scale factors of the model structure**

<table>
<thead>
<tr>
<th>Length ($L$)</th>
<th>Time ($t$)</th>
<th>Modulus of Elasticity ($E$)</th>
<th>Mass ($M$)</th>
<th>Acceleration ($a$)</th>
<th>Force ($F$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>2</td>
<td>12</td>
<td>5800</td>
<td>5.5</td>
<td>32000</td>
</tr>
</tbody>
</table>

To ensure that the model excitations were strong enough to cause measurable response, the time scale factor was determined to be 2, resulting in scaled acceleration magnitudes with adequate signal to noise ratio. PVC was selected to construct the model owing to its high strength-to-weight ratio and excellent workability. More importantly, the low modulus of elasticity of PVC, 2.5 GPa, leads to a high mass scale ratio and hence a low model mass.
7. Field test of a bridge segment for modular expansion joint design

Table 7.2 Dimensions and dynamic characteristics of the prototype structure and the model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Prototype structure</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span length</td>
<td>100 m</td>
<td>4.55 m</td>
</tr>
<tr>
<td>Pier height</td>
<td>15.5 m</td>
<td>0.705 m</td>
</tr>
<tr>
<td>Pier 2nd moment of area</td>
<td>0.387 m$^4$</td>
<td>1.65 × 10$^{-6}$ m$^4$</td>
</tr>
<tr>
<td>Bending stiffness</td>
<td>7.189 × 10$^7$ N/m</td>
<td>4.95 × 10$^4$ N/m</td>
</tr>
<tr>
<td>Fundamental frequency</td>
<td>0.98 Hz</td>
<td>1.96 Hz*</td>
</tr>
<tr>
<td>Seismic mass</td>
<td>1.895 × 10$^6$ kg</td>
<td>324 kg*</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>30 GPa</td>
<td>2.5 GPa</td>
</tr>
<tr>
<td>(50 MPa concrete)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* This is one fundamental frequency amongst the seven considered along with the corresponding seismic mass of the bridge model considered in this study

To achieve the appropriate model characteristics, the columns were fabricated as 100 × 36 mm box sections with a thickness of 6 mm and the deck was a 330 × 140 mm box section with a thickness of 10 mm. The dimensions of the model are shown in Figure 7.3. A pair of concrete foundations (pad footings) with dimensions of 340 × 340 × 70 mm was used.

![Figure 7.3 Schematic drawing of the model](image)

7.2.2 Soil and structural properties and test setup

The field test was conducted on Auckland’s Kohimarama beach with the concrete footings founded at a depth of 200 mm on compacted sand. The compaction was performed with three sand layers to ensure that over a depth, the sand was uniformly compacted. The bridge was levelled. To determine the characteristics of the soil on the
field test location, an in-situ soil density test was performed. This gave a measured soil density of $1.67 \times 10^3$ kg/m$^3$. The Rayleigh wave velocity was measured by two accelerometers positioned with spacing of 1 m. An electromagnetic inertia force exciter was used to apply harmonic loading, with a frequency band of 3.4 to 7.0 Hz, to the centre of a round plate lying on the sand surface in line with the accelerometers. The selection of the frequency band is due to (1) that the phase velocity for frequencies lower than 3.4 Hz showed high uncertainty and (2) that the values of phase velocity for frequencies higher than 7.0 Hz may be affected by the higher modes of Rayleigh-waves (Tokeshi, et al., 2008). The phase velocity of Rayleigh wave ($v_R$) was estimated using Equation 7.1 (Park, et al., 1999):

$$v_R = \frac{dx}{\Delta t}$$

(7.1)

where $dx$ is the distance between the two accelerometers, and $\Delta t$ is the difference in the arrival time of the waves to the accelerometers. The shear wave velocity ($v_s$) can be approximated by Equation 7.2:

$$v_s = \frac{0.87 + 1.12\nu}{1 + \nu} v_R$$

(7.2)

where $\nu$ is the Poisson ratio of the soil. By assuming 0.33 for the Poisson ratio of the sand, the average shear wave velocity $v_s$ is found to be approximately 220 m/s.

To apply excitations to the model bridge, two electromagnetic exciters (APS Model 400 Electro-Seis) were placed on the deck above the columns as shown in Figure 7.4. The exciters have a frequency range of 0 - 200 Hz, a peak stroke of 158 mm and a mass of 73 kg. The excitations were the inertia force produced by the 18 kg attached masses driven by the exciters. The bridge deck displacements were recorded using a linear variable differential transducer (LVDT). Mass blocks (lead bar) each with a mass of 20 kg were used to achieve the seismic mass required. Figure 7.4 shows the model of the field experiment.
To understand the influence of SSI on the structural response, the same tests were conducted with a fixed base foundation in the laboratory. For both foundation conditions, the columns were bolted to the concrete footings as shown in Figure 7.5.

Figure 7.5 Foundation conditions. (a) Field test (concrete footing on sand) and (b) fixed base

To investigate the effect of bridge properties, a series of segment fundamental frequencies were considered. The total mass of the blocks was varied to achieve the targeted fundamental frequencies between 1.7 Hz and 2.3 Hz. Based on a time scale ratio of 2, these frequencies correspond to equivalent frequencies of 0.85 Hz – 1.15 Hz for the prototype structure. This frequency range covers most bridge structures. The damping of the bridge system (including soil in the field test case) for different fundamental frequencies was determined by performing snap-back tests. The damping coefficients ranged from 611 Ns/m to 308 Ns/m and from 341 Ns/m to 168 Ns/m respectively for the foundations on sand and the fixed foundations as the fundamental frequency of the bridge
7. Field test of a bridge segment for modular expansion joint design

decreased from 2.19 Hz to 1.62 Hz and from 2.3 Hz to 1.7 Hz, respectively. The fundamental frequencies obtained from the free vibration fixed-base tests and field tests are summarised in Table 7.3.

Table 7.3 Fundamental frequencies of the model obtained from the fixed base and field tests

<table>
<thead>
<tr>
<th>Seismic mass (kg)</th>
<th>Fundamental frequency (Hz)</th>
<th>Field test</th>
<th>Fixed base</th>
</tr>
</thead>
<tbody>
<tr>
<td>484</td>
<td>1.62</td>
<td>1.70</td>
<td></td>
</tr>
<tr>
<td>404</td>
<td>1.70</td>
<td>1.83</td>
<td></td>
</tr>
<tr>
<td>364</td>
<td>1.83</td>
<td>1.91</td>
<td></td>
</tr>
<tr>
<td>324</td>
<td>1.90</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>284</td>
<td>2.01</td>
<td>2.05</td>
<td></td>
</tr>
<tr>
<td>264</td>
<td>2.06</td>
<td>2.17</td>
<td></td>
</tr>
<tr>
<td>224</td>
<td>2.19</td>
<td>2.30</td>
<td></td>
</tr>
</tbody>
</table>

7.3 Results and discussion

The applied spatially varying excitations were simulated based on the design spectra for soft soil (Class D), shallow soil (Class C) and strong rock (Class A) conditions. For the soft soil condition, high, intermediate and weak correlations between the excitations at different supports were simulated while for the shallow soil and strong rock conditions, only high correlation of excitations was considered. To assure the generality of the results, for each soil condition and coherency loss, 20 sets of excitations were adopted, resulting in a total of 100 sets of spatially varying excitations. Since a controlled ground motion could not be applied to the underlying beach sand, the common approach was taken by applying horizontal acceleration with negative amplitude equal to the required ground acceleration. This has the effect of rendering the ground stationary and applying the true ground acceleration to the structural mass. The resulting inertia forces were generated by means of inertial force exciters, with the assumptions that:

1. The bases of each pair of piers as shown in Figure 7.2 are subject to the same ground motion.

2. The effective longitudinal structural mass is concentrated at the deck level, equally at the top of each pier.
7. Field test of a bridge segment for modular expansion joint design

3. There is no longitudinal structural interaction with adjacent segments at either end of the girder since the study focuses on the total gap required to avoid girder pounding.

4. Half the equivalent inertia forces are applied simultaneously to the deck at the tops of each pier (this is due to the limited capacity of the inertial exciters), and pier displacements are equal to those would result from direct ground excitation.

5. The model will behave as a SDOF system.

6. Only the influence of the characteristics of different soil type excitations on the model response is considered while the same beach sand is used.

This chapter investigates the consequences of SSI and spatially varying excitations on the total gap that a MEJ must have to prevent pounding between bridge segments of variable fundamental frequencies. The relative displacements were normalised by the corresponding reference displacement, calculated as the average of the 20 maximum absolute girder displacements of segment 1 with an assumed fixed foundations due to excitations of the same soil condition. The reference values for each soil condition are given in Table 7.4. Larger displacements resulted from the soft soil condition than the shallow and strong rock conditions. This is due to the higher spectral values of the New Zealand design spectra for the soft soil condition in the vicinity of the fundamental frequency of the scaled model. By normalising the relative displacements using the corresponding reference values, the effect of varying seismic input on the structure due to different soil types was neutralised. Hence, the results are comparable.

**Table 7.4 Summary of the reference displacements**

<table>
<thead>
<tr>
<th>Fundamental frequency (Hz)</th>
<th>Reference displacement (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soft soil</td>
</tr>
<tr>
<td></td>
<td>2.57</td>
</tr>
</tbody>
</table>

Some current bridge specifications, for example, the California Department of Transportation (CALTRANS, 2010) and Japanese Road Association (JRA, 2004), advocate designing adjacent bridge spans to have the same or similar fundamental frequencies to reduce the likelihood of out-of-phase movement. However, even if all structural segments have the same fundamental frequency, if spatially varying excitations are present, relative girder displacements will consequently certainly occur.
7. Field test of a bridge segment for modular expansion joint design

Figure 7.6(a) compares the relative displacements ($u_{rel}$) of segments 1 and 2, both with a fundamental frequency of 1.9 Hz due to spatially varying, time delayed and uniform excitations of the soft soil condition with SSI effects. For spatially varying excitations, intermediate correlation was considered. The results show that spatially varying excitations resulted in the largest relative displacements (the dotted line) while under uniform excitations (the thick solid line) zero relative movement was expected as the bridge segments respond identically. It shows that the recommendation of matching fundamental frequencies of adjacent structures does not necessarily ensure in-phase motion. On the contrary, keeping the fundamental frequencies of the neighbouring structures the same could cause out-of-phase response and hence relative displacements if spatial variation of excitations is anticipated.

It is observed from Figure 7.6(a) that the peak relative displacement (2.64 cm) due to the spatially varying excitation is larger than that (1.94 cm) resulting from time delayed excitations. As seen in Figure 7.6(b), with a site distance of 100 m and an apparent wave velocity of 500 m/s, the 0.1 s phase difference (due to a time scale factor of 2) is relatively small compared to the fundamental period of the bridge spans, i.e. 0.5 s, leading to smaller relative girder displacements compared to those due to spatially varying excitations as shown in Figure 7.6(c). Under the spatially varying excitation, because of the loss of coherency, the responses of segments 1 and 2 differed significantly, and hence resulted in larger relative displacements. The results indicate that spatial variation of ground motion should be given full consideration in the design of long-span bridges. Considering only the wave passage effect cannot ensure a sufficient minimum total gap of an MEJ.
The normalised relative displacements due to different soil conditions and coherency loss were compared in Figure 7.7 for different fundamental frequency ratios \( f_2 / f_1 \) or \( f_3 / f_2 \) considering SSI. When considering segments 1 and 2, the fundamental frequency of segment 1 was kept as 1.9 Hz and that of segment 2 was varied between 1.62 Hz and 2.19 Hz. For segments 2 and 3, segment 2 was kept with a fundamental frequency of 1.9 Hz. The reference displacements used to normalise the responses due to different soil conditions are provided in Table 7.4. Figure 7.7(a) and (b) reveal the consequences of different soil conditions of excitations on the relative girder response. For a frequency ratio of 0.85, the maximum normalised relative movement between segments 1 and 2 under the excitations of the soft soil condition was 1.6 while the minimum relative movement was no lower than 0.9, found in shallow soil condition at a frequency ratio of 0.85.
7. Field test of a bridge segment for modular expansion joint design

1.08. For frequency ratios below 0.96, the excitations of the soft soil and strong rock conditions resulted in similar normalised relative displacements. According to Figure 7.7(a) and (b), the soil conditions of the excitations have similar influence on the normalised relative response. In general, the relationship between the frequency ratio and the normalised relative displacements tends to be independent of the soil conditions of the ground motions.

Figure 7.7(c) and (d) show the influence of different coherency loss of excitations on the relative girder response. These figures indicate that the response of adjacent segments differed most when the corresponding site excitations had a weak correlation. For a frequency ratio of 0.85 between segments 1 and 2, the normalised average maximum relative displacement resulting from a weak excitation correlation was found to be 1.7, while high and intermediate correlations led to a normalised value of 1.6. When the frequency ratio was 1.15, the difference between the normalised average maximum relative displacements due to weak and intermediate correlations was approximately 0.2. These observations indicate that the consequences of having a weak correlation between the adjacent support excitations are more significant than having a high or intermediate correlation. If considering segments 2 and 3 (Figure 7.7(d)), the coherency loss effect of excitations became more pronounced as the relative response resulting from different coherency losses diverged more as the frequency ratio exceeded 0.9.
7. Field test of a bridge segment for modular expansion joint design

Figure 7.7 Normalised average maximum relative displacement between different segments (segments 1 and 2 or segments 2 and 3) considering SSI due to different (a) and (b) soil conditions of excitation with high correlation, and (c) and (d) coherency loss of excitations of soft soil condition.

Since equalising the fundamental frequencies of the adjacent structures cannot avoid out-of-phase movement in the presence of spatially varying excitations, it is important to determine the segment frequency combination that will result in minimum relative displacement even if spatial variability of excitations is considered.

In Figure 7.7, the descending trends of the normalised relative displacements can be clearly observed as the frequency ratio increases. Figure 7.8 reveals the reason for this descending trend with the plots of the relative and absolute girder displacements of segments 2 and 3 due to frequency ratios of 0.85 and 1.15, respectively. The maximum relative displacement resulting from each fundamental frequency ratio is indicated. As shown in Figure 7.8(a), segment 3 responded to the excitation more vigorously between...
3.7 s and 5.5 s. During this period, the segments were almost moving in phase. The maximum relative displacement occurred at about 3.5 s, when the displacement of segment 2 almost reached its maximum while that of segment 3 was nearly zero. In contrast, Figure 7.8(b) shows that the displacements of segments 2 and 3 were, for most of the time, asynchronous. The maximum relative displacement took place when both segments approached the respective extreme absolute displacements in the opposite directions. It results in a maximum relative displacement of 4.45 cm, which is almost two times larger than that resulting from a frequency ratio of 1.15, i.e. 2.42 cm as shown in Figure 7.8(a). This occurs even though the maximum displacement of segment 3 with a fundamental frequency of 2.19 Hz is larger than that with a fundamental frequency of 1.62 Hz.

After examining the displacement time histories of segments 2 and 3 due to all the other spatially varying excitations, it was found for $f_3/f_2 = 1.15$ that most of the substantial displacements of the segment occurred between 4.5 s and 6 s, during which, the segments responded almost in phase. For $f_3/f_2 = 0.85$ (Figure 7.8(b)), segments 2 and 3 moved completely out-of-phase at approximately 3 s and 6 s, where the absolute displacements of segments 2 and 3 are significant. It is attributable to the unequal fundamental frequencies of the segments and the coherency loss of the excitations. The observation shows that the fundamental frequencies of the participating soil-foothing-structure systems are important for determining the maximum relative displacements as they can contribute to the development of out-of-phase displacements of adjacent structures.
7. Field test of a bridge segment for modular expansion joint design

Figure 7.8 Consequence of frequency ratio for the relative displacements between segments 2 and 3 due to highly correlated ground motions of soft soil condition for (a) $f_3 / f_2 = 1.15$ and (b) $f_3 / f_2 = 0.85$

To study the effects of SSI, the average maximum absolute displacements of segment 1 due to SSI were compared with those without SSI in Table 7.5 for the excitations of different soil conditions. The larger girder displacement due to each foundation condition has been highlighted. It is clear that in most cases SSI caused smaller maximum displacements than the fixed base foundation for the same structural mass.

Table 7.5 Comparison of average maximum absolute displacement of segment 1 with and without SSI

<table>
<thead>
<tr>
<th>Fundamental frequency (Hz) (field test / fixed base)</th>
<th>Displacement (cm) of segment 1 (field test / fixed base)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soft soil</td>
</tr>
<tr>
<td>1.62 / 1.70</td>
<td>2.38 / 2.46</td>
</tr>
<tr>
<td>1.70 / 1.83</td>
<td>2.43 / 2.47</td>
</tr>
<tr>
<td>1.83 / 1.91</td>
<td>2.44 / 2.43</td>
</tr>
<tr>
<td>1.90 / 2.00</td>
<td>2.50 / 2.57</td>
</tr>
<tr>
<td>2.01 / 2.05</td>
<td>2.62 / 2.77</td>
</tr>
<tr>
<td>2.06 / 2.17</td>
<td>2.90 / 2.91</td>
</tr>
<tr>
<td>2.19 / 2.30</td>
<td>3.02 / 3.03</td>
</tr>
</tbody>
</table>
Figure 7.10 illustrates SSI effect on the normalised relative displacement between segments 1 and 2 and between segments 2 and 3 considering the wave propagating only from left to right. When considering segments 1 and 2, the fundamental frequency of segment 1 was kept as 1.9 Hz and that of segment 2 was varied between 1.62 Hz and 2.19 Hz. For segments 2 and 3, segment 2 was kept with a fundamental frequency of 1.9 Hz. Note that the frequency ratios for the SSI cases and the fixed base cases are similar but not the same. For the presentation, the frequency ratios for SSI case were used. The averages are shown in brackets. As observed from both figures, the majority of the differences are positive with the exception of some due to the soft soil condition for frequency ratios of 0.85, 1.06 and 1.08 in Figure 7.10(a) and 1.06 and 1.08 in Figure 7.10(b) with the negative differences insignificant compared to the positive ones. For the cases considered, SSI has a beneficial effect in terms of reducing the relative girder displacements. For segments 1 and 2, SSI reduced the relative displacements by a greater amount for the shallow soil and the strong rock conditions compared to the soft soil condition. The average reduction in the relative displacements for the shallow soil and strong rock conditions is 15% and 16% of the reference displacements i.e. 1.53 cm and 1.29 cm, respectively (see Table 7.5). In Figure 7.10(b), the ground motions of shallow soil condition caused more reduction, i.e. 28% of the reference displacement on average, than the other soil conditions. SSI resulted in more reduction in the relative displacements between segments 1 and 2 than between segments 2 and 3 under the ground motions of the strong rock condition by comparing the average reduction in brackets (i.e. 0.32 and 0.10).
In reality, the propagation direction of seismic waves is not predictable; however, it will affect the relative response of neighbouring girders. In order to determine the maximum relative displacement between two segments, the seismic wave was considered to reach the three-segment bridge from either direction. In Figure 7.2, the case of seismic waves propagating from left to right is indicated. In the case of the excitations applied from the opposite direction, segment 3 would experience site 1 excitation ($a_{g1}(t)$) while segment 1 would experience site 3 excitation ($a_{g3}(t)$). Figure 7.10 shows the minimum total gaps of MEJ between segments 1 and 2 of the prototype bridge required to avoid pounding considering an earthquake loading in either direction. The fundamental frequency of segment 2 was varied while segment 1 was kept with a constant fundamental frequency of 1.9 Hz. The results in Figure 7.10 were obtained by selecting the larger values between the average maximum relative displacements resulting from the propagation of the seismic wave in either direction. This figure reveals that segments with an identical fundamental frequency required the smallest MEJ gap to prevent pounding. Consequently, even if spatially varying excitations can be anticipated, matching the fundamental frequencies of the adjacent girders is still beneficial as the bridge can experience the seismic wave from either direction. The soft soil condition required the largest minimum total gap while the strong rock condition demanded the least. When the frequency ratio changed from 1 to 1.15, the total gaps due to the ground motions associated with the soft soil condition increased by 0.34 m while the increase was only 0.12 m due to those of the
strong rock condition. Thus, the influence of different fundamental frequencies of the segments on the total gap due to the ground motions of the strong rock condition was not as significant as for those of the soft soil condition.

![Graph showing the minimum MEJ total gap required for an MEJ to avoid pounding between segments 1 and 2 due to highly correlated ground motions with SSI.](image)

**Figure 7.10** Minimum total gap required for an MEJ to avoid pounding between segments 1 and 2 due to highly correlated ground motions with SSI.

Figure 7.11 reveals the minimum MEJ total gap between segments 1 and 2 under the ground motions of different soil conditions considering the wave propagation in either direction. The fundamental frequency of segment 1 was kept as 1.9 Hz for SSI case and 2 Hz for fixed foundation case while those of segment 2 were varied. Figure 7.11(b), (d) and (f) the difference \((S_{\text{spatial}} - S_{\text{uniform}})\) of the minimum total gap of structures with fixed foundation due to spatially varying and uniform ground motions and the difference \((S_{\text{fixed}} - S_{\text{SSI}})\) between the minimum total gap of structures with fixed foundation and SSI due to spatially varying ground motions. As shown for all soil condition cases, the influence of spatially varying excitations on the minimum MEJ total gap is significant as the spatial variation, for almost all the frequency ratios, resulted in larger relative displacements than the uniform excitations.

When the segments have the same fundamental frequency, the underestimation of the total gap required to avoid pounding due to the assumption of uniform ground motions is most significant. Therefore, considering spatially varying ground motions is critical for bridge segments with identical fundamental frequencies. As the fundamental frequency ratio drifts away from 1, the underestimation of the MEJ gap due to neglecting spatial variation of ground motions becomes less. This is because the relative displacements due to the difference in the dynamic properties of the adjacent bridge segments also have a
contribution to the total MEJ gap. The more different the adjacent bridge segments, the more dominant the influence of their dynamic properties, and the less the MEJ gaps is underestimated by neglecting spatially varying ground motions.

**Figure 7.11** Consequence of spatially varying ground motion and SSI for the minimum total gap required between segments 1 and 2 due to excitations of (a) (b) soft soil, (c) (d) shallow soil and (e) (f) strong rock conditions, all with high correlation.

SSI reduced the minimum total gaps needed for preventing pounding as the difference between the total gap due to fixed base and SSI is always positive. The largest decrease of the minimum total gap due to SSI takes place when the frequency ratio is 1.08 in the case
of soft soil condition and 1.05 in the case of shallow soil and strong rock conditions, equivalent to 19%, 18% and 20% of the minimum total gap due to spatially varying ground motions of respectively the soft soil, shallow soil and strong rock conditions with fixed base foundation.

### 7.4 Summary

In contrast to conventional bridge expansion joints, modular expansion joints (MEJs) can be used to prevent bridge pounding. To achieve this objective, the minimum total gap between the centre beams should be at least equal to the maximum relative displacement between adjacent segments. To understand the simultaneous effects of spatially varying ground motions and soil-structure interaction (SSI) on relative girder displacement, a 1:22 scale bridge model was constructed and tested on beach sand using inertial force exciters. A range of fundamental frequencies was considered by varying the masses on the deck. Spatially varying excitations were modelled based on New Zealand design spectra with an empirical coherency loss function. The simulated excitations considered the soft soil, shallow soil and strong rock conditions with the high correlation. In addition, for the soft soil condition, the intermediate and weak correlations were also considered. The tests were repeated with fixed foundations in order to compare with those obtained from considering SSI. A total of 4200 tests were performed.

Based on the results, the following conclusions are derived:

1. Since the excitation direction is unpredictable, the conventional bridge design recommendation of matching the fundamental frequencies of adjacent bridge structures can still be worth following as this will require the smallest total gap of an MEJ to prevent pounding even if spatially varying ground motions are considered.
2. Should the direction of the seismic wave propagation be given, making a bridge segment more flexible than the following segment can result in smaller relative girder displacements.
3. The combined effects of the wave passage and coherency loss can result in more than 30% larger relative girder displacements than considering the wave passage effect alone.
4. When SSI is considered, a weak correlation of spatial excitations may result in larger relative girder displacement than a high or an intermediate correlation.

5. In the cases considered, SSI is beneficial in terms of reducing the minimum total gap of an MEJ, even when spatially varying ground motions are considered. The reduction can be 20%.

6. An assumption of uniform ground motions for bridge segments with identical fundamental frequencies will underestimate the minimum gap significantly.
Chapter 8

Recommendations for minimum seismic seating length of bridge girders

8.1 Introduction

Spatial variation of ground motions has gained wide recognition for its importance in bridge seismic design. However, to the authors’ best knowledge, amongst all the bridge design regulations, only Eurocode 8 (2005), AASHTO specifications (2010) and JRA specifications (2004) consider spatial variation of ground motion. Even if JRA considers spatially varying ground motions, research by Chouw and Hao (2006) still showed that the recommendation of JRA on minimum seating length was insufficient. The most up-to-date New Zealand Transport Agency (NZTA) bridge manual (2005) was revised in 2005. The current formula for minimum seating lengths is still based on the assumption of uniform ground motions. It is possible that the current NZTA Bridge manual may underestimate the necessary seating lengths under spatially varying ground motions. This chapter, therefore, evaluated the current NZTA Bridge manual and provides recommendation for a possible amendment.

8.2 Current design regulations

Before the evaluation, some commonly used bridge specifications for straight bridges are discussed as follows:

Eurocode 8-2 (EC8-2). It is one of the seismic specifications that clearly recognises the importance of the spatial variability of seismic excitations and provides a detailed procedure for considering it when one or both of the following conditions apply: (1) soil properties at various bridge supports correspond to different categories, and (2) the length
8. Recommendations for minimum seismic seating length of bridge girders

of the continuous deck exceeds \( L_g / 1.5 \), where \( L_g \) is the distance beyond which ground motions may be considered uncorrelated (see Table 8.1). EC8-2 considers two cases as the most critical: (1) all the supports are subjected to displacements in the same horizontal direction but of different magnitudes, and (2) two successive supports are subjected to opposite displacements. The maximum displacements of the structure due to the two cases are then combined with the results of a typical uniform excitation analysis using the root mean squares rule. How EC8-2 addresses the spatial variability is not the main focus of this paper, and the details of these provisions are referred to (2005).

EC8-2 defines the minimum seating length (\( SL \)) for end support on an abutment as follows:

\[
SL = l_m + d_{eg} + d_{es}
\]  
(8.1)

\[
d_{eg} = \varepsilon_s L_{eff} \leq 2d_g
\]  
(8.2)

\[
\varepsilon_s = \frac{2d_g}{L_g}
\]  
(8.3)

\[
d_g = 0.025 a_g S_F T_C T_D
\]  
(8.4)

where \( l_m \) is the minimum support length securing the safe transmission of the vertical reaction \( \geq 40 \text{ cm} \); \( d_{eg} \) 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; \( S_F \) is the soil factor, defined in Table 8.1 along with \( T_C \) and \( T_D \); \( d_{es} \) is the effective seismic displacement of the support due to the deformation of the structure; \( L_{eff} \) is the effective deck length, taken as the distance from the deck joint considered to the nearest full connection of the deck to the substructure. If the deck is fully connected to more than one pier, \( L_{eff} \) 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 bearings, seismic links, or shock transmission units.
Table 8.1 Recommended values of the parameters for Types 1 and 2 elastic response spectra (reproduced from EC8-2 (2005))

<table>
<thead>
<tr>
<th>Case</th>
<th>$S$</th>
<th>$T_c$ (s)</th>
<th>$T_d$ (s)</th>
<th>$L_g$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spectrum type</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1, 2</td>
</tr>
<tr>
<td>Soil class A</td>
<td>1.2</td>
<td>0.4</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>Soil class B</td>
<td>1.15</td>
<td>0.5</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>Soil class C</td>
<td>1.35</td>
<td>0.6</td>
<td>0.25</td>
<td>2</td>
</tr>
<tr>
<td>Soil class D</td>
<td>1.5</td>
<td>0.8</td>
<td>0.3</td>
<td>2</td>
</tr>
<tr>
<td>Soil class E</td>
<td>1.6</td>
<td>0.5</td>
<td>0.25</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: Type 2 spectrum is recommended only for regions where the design earthquake has a surface wave magnitude less than 5.5.

EC8-2 also suggests an increased seating length by a safety factor of $\sqrt{2}$ for abutment supports to accommodate extreme relative displacements due to spatially varying ground motions.

**AASHTO specifications.** AASHTO requires each bridge to be designed for one of the four seismic design categories (SDC), A through D (refer to AASHTO Clause 3.5). For SDC A and SDC B, AASHTO requires a minimum seat length $SL$ for all the supports to satisfy the following equation, which prescribes $SL$ in terms of the span $L_s$ (m) and the height of the column or pier $H$ (m):

$$SL = 0.203 + 0.00167L_s + 0.00666H$$ (8.5)

A safety factor of 1.5 is introduced to the minimum seating length if the bridge is designed for SDC C and SDC D.

**JRA specifications.** JRA suggests that the seating length ($S$) of a girder at its support shall be calculated using Equation 8.6 and shall not be less than the minimum seating length ($SL$) obtained from Equation 8.7.

$$S = u_{red} + u_G \geq SL$$ (8.6)

$$SL = 0.7 + 0.005L$$ (8.7)

$$u_G = \varepsilon_G L$$ (8.8)
8. Recommendations for minimum seismic seating length of bridge girders

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

The most up-to-date JRA seismic specifications include the relative displacement response spectra generated using 63 strong ground motions recorded during earthquakes in Japan with a focal depth less than 60 km and magnitude equal to 6.5 or greater (Chouw and Hao, 2006).

**NZTA Bridge manual.** This manual specifies the requirement for the minimum seating length between the span and the support in Clause 5.6.2 (d) as follow:

\[
SL = 2.0 E + 0.1 \geq 0.4 \ m
\]

where \( E \) is relative movement between span and support.

It is noted that the AASHTO specifications empirically define the minimum seating length only by the span and the pier height for a straight bridge, which may be oversimplified. In contrast to the AASHTO provisions, the JRA specifications account for the effects of the frequency ratio of the neighbouring structures and spatially varying excitation by incorporating the relative movement between the superstructure and the substructure, and the empirical relative ground movements at adjacent supports. Despite the advanced empirical equations adopted by the JRA specifications, it could still underestimate the minimum seating length required to prevent deck unseating (Chouw and Hao, 2006). According to the study of Sextos and Kappos (2009), EC8-2 can significantly underestimate the actual seismic demand under spatially varying ground excitations.

The NZTA Bridge manual, on the other hand, relates the minimum seating length only to the relative displacement between span and support obtained by assuming uniform ground motions, even if the proposed equation (Equation 8.9) is intended to account for out-of-phase ground excitations. This equation depends heavily on the relative displacement, which could potentially be underestimated due to the assumption of uniform ground motions.
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8.3 Experiments

The required seating lengths were calculated for the models presented in Chapters 3, 6 and 7, based on the suggestions from the NZTA Bridge manual. The predicted values were compared with the results from the experiments of three identical segments, a single segment with abutments and the field test of a larger 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, the adequacy of the NZTA Bridge manual for suggesting minimum seating length can then be reviewed when spatially varying ground motions are anticipated.

8.4 Evaluation of NZTA Bridge manual

8.4.1 Relative displacements between segments

Figure 8.1 compares the maximum relative opening displacements obtained from the pounding tests involving two and three segments due to excitations of different soil conditions and coherency losses with the minimum seating lengths recommended by the NZTA manual. Pounding was considered, but SSI was not. According to Equation 8.9, the minimum seating length should be at least 0.4 m. This is because under uniform ground motions, identical segments will move identically, resulting in zero relative displacement. None of Figure 8.1(a)-(c) show the prescribed minimum seating length is sufficient to accommodate the maximum relative opening displacement when spatially varying ground motions were considered. Figure 8.1(b) shows that in the case of the three-segment bridge, the minimum seating length between segments 1 and 2 due to weakly correlated ground motions of soft soil condition was underestimated by the NZTA manual by almost 50%, i.e. 0.2 m while for the two-segment bridge, it was underestimated by more than 100%, i.e. 0.42 m. This reveals that the current NZTA Bridge manual can underestimate the seating length required to prevent unseating if the bridge experiences ground motions of soft soil condition, especially when only two segments are involved and the ground motions are weakly correlated.
Figure 8.1 Comparisons between the relative opening displacements and the recommended minimum seating lengths without SSI; (a) and (b): Three-segment bridge tests and (c) two-segment bridge test
8. Recommendations for minimum seismic seating length of bridge girders

Figure 8.2 reveals the maximum relative opening displacements due to the 20 spatially varying excitations of different soil conditions with high correlations in comparison with the minimum seating length recommended by the NZTA Bridge manual. Pounding was considered. This figure shows that with an exception of set 10, all the ground excitations of the soft soil condition demand a larger seating length than that the minimum seating length suggested by the NZTA Bridge manual is not sufficient to accommodate the relative opening displacements of a three-segment bridge. Even if the suggested seating length of 0.4 m can cope with most of the maximum relative opening displacements resulting from the excitations simulating the shallow soil condition, there are still occasions (sets 1 and 15) where the necessary seating length is slightly underestimated.

![Figure 8.2](image.png)

**Figure 8.2** Comparisons between the maximum relative opening displacements (segments 1 and 2) of the three segment bridge due to the 20 highly correlated ground motions of different soil conditions and the NZTA manual recommended seating length

Table 8.2 summarises the average maximum relative opening displacements of the two and three-segment bridges due to spatially varying ground motions with different correlations in comparison with the minimum seating lengths recommended by the NZTA Bridge manual. The table shows that a bridge with three segments requires shorter seating length than a bridge with two segments if pounding is allowed. This is because the pounding from the additional segment limits the movement of the middle segment and thus leads to smaller relative opening displacements. For the cases considered, the NZTA Bridge manual can underestimate the minimum seating lengths when the adjacent segments are identical and subjected to the spatially varying ground motions of soft soil conditions.
8. Recommendations for minimum seismic seating length of bridge girders

Table 8.2 Comparison between the seating length suggested by the NZTA Bridge manual and the recorded average maximum relative displacements of two and three identical segments (between segments 1 and 2) without SSI

<table>
<thead>
<tr>
<th>Relative opening displacement (m)</th>
<th>Soft soil of different correlation</th>
<th>Shallow soil</th>
<th>Strong rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High</td>
<td>Intermediate</td>
<td>Weak</td>
</tr>
<tr>
<td>Two segments</td>
<td>0.63</td>
<td>0.64</td>
<td>0.64</td>
</tr>
<tr>
<td>Three segments</td>
<td>0.50</td>
<td>0.52</td>
<td>0.54</td>
</tr>
<tr>
<td>NZTA recommendation</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
</tbody>
</table>

8.4.2 Girder relative displacements with SSI

In addition to the experiments of the fixed base case, a field test of a larger single segment bridge was carried out to reveal the influence of SSI on bridge relative displacements. The bridge model was subjected to the spatially varying ground motions of three sequential sites to simulate the response of a three-segment bridge. Pounding was not considered. Figure 8.3 shows the overall average maximum relative opening displacements between any adjacent segments considering excitations from both possible directions along the bridge in comparison with the minimum seating lengths suggested by the NZTA manual. The suggested seating lengths were calculated using Equation 8.9 with the value of $E$ being the experimental relative displacements obtained from uniform ground motions. Segments with different fundamental frequencies were considered. The frequencies of the outer segments were varied from 1.62 Hz to 2.16 Hz for the case of foundations on sand and from 1.7 Hz to 2.3 Hz for the fixed foundations. The dynamic property of the middle segment was kept constant with a fundamental frequency of 1.9 Hz and 2 Hz for the case of foundations on sand and fixed foundations, respectively. The fundamental frequency ratios were defined by dividing the fundamental frequency of the outer segment by that of the middle segment. The maximum relative opening displacements shown in these figures are the overall maxima between those of segments 1 and 2 and those of segments 2 and 3.

The results show that the NZTA Bridge manual is able to predict sufficient seating length for fundamental frequency ratios approximately greater than 1.07 (Figure 8.3(b)) or less than 0.95 (Figure 8.3(a)) regardless of SSI effects. However, for structures subjected to excitations of soft or shallow soil conditions the Bridge manual recommendation can
underestimate the necessary seating length if the adjacent segments have identical or very similar fundamental frequencies, even if this is recommended by many bridge specifications, e.g. CALTRANS and JRA specifications, in order to minimise the out-of-phase movement. Under the earthquake loading, the required seating length for identical segments was underestimated by approximately 50% for the soft soil condition and more than 10% for the shallow soil condition regardless of SSI. Despite the same dynamic properties of segments, they will still move out of phase due to time delay in excitations at different supports and coherency loss of the propagating seismic wave through soil media. It is critical because bridge designers are encouraged to design the bridge segments with the same or similar dynamic characteristics. As shown in all these results, the recommended minimum seating length increases sharply as the frequency ratio drifts away from unity, resulting in uneconomic overestimation of the minimum seating length. Hence the current NZTA Bridge manual needs to be improved.

![Figure 8.3](image)

**Figure 8.3** Comparisons between the maximum relative opening displacements due to spatially varying ground motions of (a) soft soil, (b) shallow soil and (c) strong rock.
conditions and the recommended minimum seating length obtained from uniform ground motions for different fundamental frequency ratios of the segments without pounding.

### 8.4.3 Relative displacement between abutment and girder

The average maximum relative opening displacements between an abutment and a bridge structure were also examined to evaluate the minimum seating length recommended by the NZTA Bridge manual. The excitations of the abutments were considered with spatial variation effects. The abutments were rigidly connected to the ground and moved with the ground as rigid bodies. Fundamental frequencies of the prototype bridge structure ranging from 0.7 Hz to 1.3 Hz were investigated in order to cover a range of possible bridge structures. Pounding was considered. Figure 8.4 compares the recorded maximum girder-abutment separations with the estimate of the minimum seating length according to NZTA Bridge manual (Equation 8.9) with \( E \) being zero. It is because with the assumptions of rigid abutments, zero gap and uniform ground motions, the abutments and the segment will move together, resulting in no relative displacement between any of the abutments and the segment. For the cases considered, the relative opening displacements obtained from the experiments decrease as the bridge structure becomes stiffer regardless of soil conditions of ground motions. As shown in Figure 8.4, the measured relative opening displacements due to spatially varying ground motions of soft soil condition exceeded the seating length recommended by the NZTA manual for all the segment frequencies considered. At least 25% of the seating length was underestimated for the bridge with a fundamental frequency of 1.3 Hz and more than 33% of necessary seating length was underestimated when the bridge fundamental frequency was 0.8 Hz. For the cases considered, the recommendation of the NZTA Bridge manual can cope with the relative displacement between the abutment and the bridge structure for ground motions of shallow soil and strong rock conditions. The maximum relative opening displacement due to ground motions of shallow soil condition is very close to the recommended minimum seating length when the bridge fundamental frequency is 0.7 Hz.
8. Recommendations for minimum seismic seating length of bridge girders

According to the measured relative opening displacements from the experiments, the NZTA Bridge manual may underestimate the relative response of a bridge if spatially varying ground motions are anticipated. The ground motions of the soft soil condition may cause the greater concern in predictions of necessary seating length than shallow soil and strong rock conditions. It is, therefore, suggested that the NZTA Bridge manual should incorporate the effect of spatially varying ground motions, especially for bridge segments with identical dynamic characteristics.

8.4.4 Recommendations for NZTA Bridge manual

To incorporate the effect of ground motion spatial variation, ideally, a set of spatially varying ground motion acceleration time history should be simulated for relevant bridge sites using a coherency loss function similar to Equation 3.1. For high importance bridges, the modelling of spatially varying ground motions in the authors’ opinion should be performed. This recommendation is consistent with the recent CALTRANS practice on long bridges.

However, the application of the coherency loss function is highly specialised and thus it may not be practicable. When the modelling of spatially varying ground motions is not possible, the NZTA Bridge manual should provide designers with design guidance for the adequate minimum seating length. However, the current NZTA Bridge manual may not be able to provide sufficient recommendations. Hence, a procedure is proposed to
estimate the minimum seating length for adjacent structures based on the experimental results from the field test. The formulae are based on the results without SSI effects due to the observation that the relative displacements between any adjacent spans with fixed foundations are, in most of the cases, greater than those with SSI effect (see Figure 8.3). This is the result of the reduced (as is often the case) fundamental frequency of bridge structure with foundation and subsoil being on the ascent curves of the design spectra (Figure 3.5). Therefore, the proposed formulae are only valid for the cases where the SSI effect is beneficial and pounding is excluded.

**Figure 8.5** Comparisons between the normalised average maximum relative opening displacements between segments 1 and 2 due to spatially varying ground motions and uniform ground motion of (a) soft soil, (b) shallow soil and (c) strong rock conditions

Figure 8.5 compares the normalised maximum relative opening displacements between adjacent girders under uniform and spatially varying ground motions without pounding. The relative displacements were normalised by the average of the corresponding maximum displacements of the two participating girders due to the ground motion considered. The results were obtained by considering seismic waves exciting the
8. Recommendations for minimum seismic seating length of bridge girders

segments from both directions along the bridge. By normalising the relative displacement, more general relationships between the relative displacements and the structure responses can be obtained. Figure 8.5 reveals that the normalised relative opening displacements are decreasing as the frequency ratio approaches 1.0. The maximum difference between the results due to the spatially varying ground motions and those due to uniform ground motions occurred when the segments were identical. The differences between the normalised relative movements of the bridge subjected to uniform and spatially varying ground motions are presented in Figure 8.6 with the frequency ratio calculated by dividing the fundamental frequency of the more flexible segment by that of the stiffer segment.

![Graphs showing the relation between the frequency ratio of the structures and the differences between the maximum relative displacements due to spatially varying and uniform ground motions of (a) soft soil, (b) shallow soil and (c) strong rock conditions with high correlation.]

**Figure 8.6** Relation between the frequency ratio of the structures and the differences between the maximum relative displacements due to spatially varying and uniform ground motions of (a) soft soil, (b) shallow soil and (c) strong rock conditions with high correlation.
8. Recommendations for minimum seismic seating length of bridge girders

As shown in Figure 8.6, the differences between the relative opening displacements show approximately a linear relationship with respect to the frequency ratios for ground motions of each soil condition. The lines of best fit are found to have similar gradients and y-intercepts at $f_{\text{flexible}}/f_{\text{stiff}} = 0$. Table 8.3 shows the gradients and the y-intercepts (at $f_{\text{flexible}}/f_{\text{stiff}} = 0$) of the lines of best fit for all the ground motion cases considered.

**Table 8.3** Gradients and y-intercepts (at $f_{\text{flexible}}/f_{\text{stiff}} = 0$) of the line of best fit for the differences between normalised maximum relative displacements due to spatially varying and uniform ground motions with respect to the participating segment fundamental frequency ratios

<table>
<thead>
<tr>
<th>Ground motions of soil condition and coherency loss cases</th>
<th>Soft soil (high)</th>
<th>Soft soil (intermediate)</th>
<th>Soft soil (weak)</th>
<th>Shallow soil (high)</th>
<th>Strong rock (high)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradient</td>
<td>4.4</td>
<td>5.0</td>
<td>5.6</td>
<td>4.7</td>
<td>5.0</td>
</tr>
<tr>
<td>y-intercept</td>
<td>-3.3</td>
<td>-3.8</td>
<td>-4.3</td>
<td>-3.5</td>
<td>-4.0</td>
</tr>
</tbody>
</table>

According to the results shown in Table 8.3, a general equation regardless of the soil condition was established by selecting the largest gradient and the y-intercept, namely 5.56 and 3.29, respectively to ensure the adequacy of the predicted seat length, and defined as follows:

$$SL_{\text{proposed}} = (5.6f_r - 3.3)d_{\text{ave}} + d_{\text{uni}}$$  \hspace{1cm} (8.10)

where $f_r$ is the ratio of the fundamental frequency of the more flexible segment to that of the stiffer segment; $d_{\text{uni}}$ is the maximum relative displacement obtained from applying uniform ground motions with fixed-base foundations, and $d_{\text{ave}}$ is the mean of the maximum displacements of the two girders with fixed-base foundations subjected to the uniform ground motions.

The proposed minimum seating length $SL_{\text{proposed}}$ accounts not only for the influence of different frequencies of the neighbouring bridge structures, but also for the effects of spatially varying ground motions through establishing the relation between the relative displacements due to uniform and spatially varying ground motions, although only empirically. The inclusion of the average of the bridge displacements in the $SL_{\text{proposed}}$ reflects the characteristics of the soil condition of the ground motion.
8. Recommendations for minimum seismic seating length of bridge girders

To derive the minimum seating length for New Zealand bridges, designers are recommended to take the following steps:

1. Determine the design earthquakes which should comply with the requirements of NZS 1170.0 (2002) and NZS 1170.5 (2004).

2. Obtain the maximum relative opening displacement between girders under uniform ground motions considering ground motions of the both possible directions.

3. Determine the average maximum displacements of the bridge structures for span supports under the designated earthquakes.

4. Apply Equation 8.10 for designing the adjacent girder seating length.

5. Run at least three time history analyses to obtain a maximum seating length.

Note that steps 2, 3 and 5 should be achieved with a fixed-base foundation and without pounding.

Table 8.4 Ratios between $d_{sv}/d_{uni}$ the maximum relative girder-to-abutment displacements due to uniform ground motions and spatially varying ground motions of different soil conditions with coherency loss cases

<table>
<thead>
<tr>
<th>Prototype frequency (Hz)</th>
<th>Soft soil (high)</th>
<th>Soft soil (weak)</th>
<th>Shallow soil (high)</th>
<th>Strong rock (high)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7</td>
<td>1.16</td>
<td>1.32</td>
<td>1.19</td>
<td>1.24</td>
</tr>
<tr>
<td>0.8</td>
<td>1.04</td>
<td>1.30</td>
<td>1.12</td>
<td>1.19</td>
</tr>
<tr>
<td>0.9</td>
<td>1.11</td>
<td>1.28</td>
<td>1.20</td>
<td>1.27</td>
</tr>
<tr>
<td>1.0</td>
<td>1.30</td>
<td>1.27</td>
<td>1.25</td>
<td>1.32</td>
</tr>
<tr>
<td>1.1</td>
<td>1.36</td>
<td>1.24</td>
<td>1.36</td>
<td>1.36</td>
</tr>
<tr>
<td>1.2</td>
<td>1.30</td>
<td>1.19</td>
<td>1.25</td>
<td>1.37</td>
</tr>
<tr>
<td>1.3</td>
<td>1.28</td>
<td>1.20</td>
<td>1.23</td>
<td>1.32</td>
</tr>
<tr>
<td>Average</td>
<td>1.22</td>
<td>1.26</td>
<td>1.23</td>
<td>1.30</td>
</tr>
</tbody>
</table>

For a girder-abutment support, the minimum seating length is estimated as a multiple of the maximum relative displacement ($d_{uni}$) between the abutment and the bridge under uniform ground motions considering abutment excitation. Table 8.4 shows the ratios between the maximum relative girder-to-abutment displacements due to spatially varying ground motions ($d_{sv}$) of different soil conditions and uniform ground motions ($d_{uni}$)
8. Recommendations for minimum seismic seating length of bridge girders without SSI. The abutments were assumed to move with the ground as rigid bodies. Pounding was excluded. As shown in Table 8.4, the maximum relative displacements considering spatially varying ground motions were found to be no greater than 1.4 times the maximum relative girder-to-abutment displacements under uniform ground motions. No trend is observed between the fundamental frequency of the bridge segment and the ratio $d_{sv}/d_{uni}$. Therefore, the minimum seat length is recommended to be calculated using Equation 8.11:

$$SL_{proposed} = 1.4 \ d_{uni}$$ (8.11)

Equations 8.10 and 8.11 were derived based on experimental results with particular conditions, e.g. an assumption of the same soil-structure interaction and adjacent bridge structure with the same slenderness. Since all the experiments were conducted using ground motions based on the New Zealand design spectra, the proposed equations may be suitable only for New Zealand bridges. However, it may be valid for similar cases. Further verification is recommended.

The method implemented in this study can also be used to evaluate the adequacy of other bridge specifications. The minimum seating lengths suggested by other design specifications are not compared in this paper since they are based on different design spectra and out of scope of this research.

To avoid serious bridge damage due to pounding, common practice for New Zealand bridges is to use knock-off nibs. After the device is knocked off the joint by impacts from the adjacent structure, pounding would no longer take place. In addition, MEJs are also a potential solution. MEJs can accommodate movements of up to 2 m (WSDOT, 2011). Especially, in near-fault earthquake regions MEJs are suitable for long bridges, where extremely large relative displacements are expected.

8.5 Summary

The provisions of the NZTA Bridge manual for estimating the minimum seating length were reviewed based on the physical results obtained from the three identical segment pounding tests, the segment-abutment pounding tests and the field tests using the shake tables or the inertial actuators. A total of 10,570 experiments were performed. The effects
8. Recommendations for minimum seismic seating length of bridge girders

of spatial variation of ground motions were addressed. By comparing the measured average maximum relative opening displacements with the corresponding estimation of the minimum seating length based on the NZTA Bridge manual, the following conclusions were made:

1. The current NZTA manual is not capable of predicting seating length required for bridge supports if spatially varying ground motions are expected, especially when adjacent segments have identical fundamental frequencies and are excited by ground motions of soft soil condition with weak correlation. Spatial variation of ground motions is critical in determining relative displacement, and hence should be considered in the current NZTA Bridge manual.

2. For bridges of high importance, spatial variation of ground motions should be closely modelled and applied to a suitable numerical bridge model to derive the maximum relative displacements.

3. Two equations were proposed for predicting the minimum seating length for span supports and abutment supports, respectively, based on the New Zealand design spectra.
Conclusions and recommendations for future research

Chapter 9

Conclusions and recommendations for future research

9.1 Conclusions

In reality, seismic ground motions at various supports of extended bridge structures are not the same. The use of uniform ground motions in bridge seismic design analysis is often proved to be insufficient. Pounding and soil-structure interaction (SSI) in strong earthquakes was also considered to have a significant influence on bridge response. Previous studies on the effect of spatially varying ground motions were mainly numerical. Therefore, the objective of this PhD research is to experimentally study the global behaviour of multi-support bridges under spatially varying ground motions considering pounding and SSI effects. The outcome of this research can be considered as an effort in understanding of seismic response of bridges and providing recommendations for updating New Zealand Bridge manual or even the international bridge specifications.

To achieve the objective, a series of shake table (or inertial actuator) experiments were conducted on physical models with different scale (1:125 and 1:22). These bridge models were constructed using Polyvinylchloride based on one segment with a span of 100 m and pier height of 15.5 m. After scaling according to the similitude scaling rules, each model with a scale of 1:125 has a span of 800 mm and pier height of 124 mm while the 1:22 scale model was constructed with a span of 4.55 m and pier height of 0.705 m. All fundamental frequencies of the models were measured to be around 2 Hz. Ground motions of soft soil, shallow soil and strong rock conditions were stochastically simulated based on New Zealand design spectra. Furthermore, the coherency loss effect of the spatially varying ground motions were also studied by considering high correlation for all three soil conditions and intermediate and weak correlation only for ground motions of
9.1.1 Effect of spatially varying ground motions and pounding on elastic bridge response

Chapter 3 studied the seismic response of a three-segment bridge with fixed supports subjected to spatially varying ground motions. The bridge response due to two-sided pounding was compared with those due to one-sided pounding. This study reveals that:

1. The recommendation of having identical or at least similar fundamental frequencies for adjacent bridge segments by current bridge specifications does not necessarily ensure in-phase motion. With the combined effect of spatially varying ground motions and pounding, the relative girder displacement can be as large as 20% of reference value, which was the average maximum absolute girder displacement caused by uniform ground motion. Ignoring spatial variation of ground motions can result in underestimation of relative girder displacements and girder damage potential due to pounding.

2. The bridge model responded more strongly to ground motions of soft soil condition than to those of shallow soil or strong rock conditions. More damage to the bridge structure can be anticipated under the ground motions of soft soil. The maximum opening relative displacement and pounding force due to ground motions of soft soil condition are nearly twice those caused by ground motions derived from strong rock condition.

3. Including pounding effect, the influence of coherency loss of ground motions is not significant. Weakly correlated ground motions did not always cause the strongest response in comparison with the non-pounding cases.

4. Pounding can dissipate energy, resulting in reduction of maximum pier bending moment and the absolute displacement of the segment in most cases considered. However, pounding can increase the relative opening girder displacements in the cases considered, contributing to girder unseating. Pounding from both girder ends...
can confine the movement of the girder and hinder the development of large relative opening displacement compared to one-sided pounding. However, the segment suffering two-sided pounding can gain more energy from the both side impact, resulting in larger pounding force.

9.1.2 Effect of soil flexibility on bridge response with pounding

Chapter 4 addresses the influence of SSI and spatially varying ground motions on bridge response including pounding. The same model described in Chapter 3 was used. SSI was considered by constructing six flexible sand boxes using soft rubber in order to reduce the boundary effect arising from rigid box walls. From this study, it was found that:

1. When pounding was excluded, the spatially varying excitation of soft soil condition led to decrease in the maximum relative girder displacements, while those of shallow soil and strong rock conditions resulted in an amplification of both the maximum girder displacement and the maximum relative opening displacement between girders.

2. SSI can reduce the maximum pounding force regardless the soil condition of the ground motions.

3. With a weak correlation of ground motions, SSI can increase the maximum bending moment or reduce the maximum pounding force with SSI more significantly while the highly correlated ground motions tend to result in more increase in the maximum relative opening displacement.

4. Pounding increased the maximum relative opening displacement regardless of SSI. Therefore, pounding can have a contribution to bridge unseating.

9.1.3 Effect of excitation spatial variation on inelastic bridge response with pounding

In Chapter 5, the influence of spatially varying ground motions and inelastic material behaviour of the model with two identical segments including pounding was considered. Artificial plastic hinges were constructed at each pier base and each column-girder joint. The importance of considering inelastic bridge response is illustrated by comparing the results with elastic bridge response. SSI and abutments were not considered. According to the pushover analysis, an overturning moment of 5.0 Nm will trigger the rotation of the plastic hinges. The soft soil spectrum from the New Zealand design spectra was used to simulate the spatially varying ground motions. High, intermediate and weak correlations
of the ground motions at adjacent supports were used. For each type of correlations, 10 sets of ground motions were considered. This study reveals:

1. Overall, spatially varying ground motions tend to increase the maximum girder displacement relative to the pier base and decrease that of adjacent bridge segment. Pounding may increase the hinge rotations.

2. Pounding can contribute to a girder unseating in the case of considering plastic hinges.

3. Plastic hinge development can reduce possible pounding-induced damage (both the magnitude and the number of large impacts) and unseating potential. Assuming elastic bridge response may overestimate the maximum relative opening girder displacement and the maximum pounding force.

4. Weakly correlated ground motions tend to result in the largest relative opening displacement and pounding forces.

9.1.4 Effect of abutment excitation on bridge response with pounding

In many studies, abutments were assumed to be fixed in their position. In reality, however, abutments will move with the ground. Chapter 6 focuses on the consequence of abutment excitation for seismic response of one-segment bridge allowing pounding. The abutments adopted in this study were made of square steel tubes which were assumed to have an infinite stiffness. The abutments moved with the shake tables as rigid bodies. The bridge segment was loaded with different masses to obtain a range of fundamental frequencies. Measuring heads with stiffnesses of 244 N/mm and 30.5 N/mm were considered as the “stiff spring” and “soft spring”, respectively. The following conclusions were derived:

1. Considering abutment excitations with spatial variability can result in larger pounding force and relative opening displacement between the abutment and the girder, but smaller pier bending moment compared to the fixed abutment case. Therefore, neglecting abutment excitation may underestimate the necessary seating length and pounding-induced damage to girder.

2. In the cases of abutment excitation, the more flexible the bridge structure the stronger the activated pounding force. Abutments with a stiff contact interface led to greater pounding force and relative opening displacement than a softer contact interface.
9.1.5 Field tests of a bridge segment for determining minimum total gap of modular expansion joints

Chapter 7 focuses on deriving the minimum total gap of modular expansion joints (MEJ) for a three-segment bridge through field testing a bridge segment with a scale of 1:22. Two inertia force actuators were placed on the deck to apply the excitations of the bridge structure to account for spatial variation of excitations. The field test was conducted on a natural beach environment. The same bridge model was also tested in the laboratory with fixed foundations. Different fundamental frequencies of the model were considered by loading different numbers of mass blocks on the girder. Different fundamental frequencies of the fixed-base model ranging from 1.7 Hz to 2.3 Hz were considered. A comparison of the average maximum relative girder displacements due to fixed foundations and SSI shows:

1. For the cases considered, SSI can reduce the bridge relative displacements without pounding.

2. Spatially varying ground motions can result in greater relative girder displacements compared to uniform ground motions regardless of SSI.

3. Larger gap is required for an MEJ when the bridge is subject to ground motions of soft soil condition and a weakly correlated ground motion tends to cause a larger minimum MEJ total gap required.

4. If the segment, which experiences the ground motion first, is more flexible than the following segment, the maximum relative displacement between them is very likely to be less than that due to ground excitation from opposite direction.

5. Since the seismic wave propagation direction is unpredictable, the recommendation of designing the adjacent bridge segments with identical or similar fundamental frequencies is still worth following, as it requires the least minimum total gap of MEJ.

9.1.6 Recommendation for minimum seating length of bridges

The recommendation of the New Zealand Transport Agency (NZTA) Bridge manual for minimum seating length required to avoid unseating was reviewed in Chapter 8. The NZTA Bridge manual assumes uniform ground motions, which may underestimate minimum seating length when spatially varying ground motions are anticipated. The
results from the experimental studies described in Chapters 3, 6 and 7 were used to review the recommendation. The conclusions are summarised as follows:

1. Current NZTA Bridge manual does not provide suggestions of sufficient seating length required for bridge supports if spatially varying ground motions are anticipated, especially when adjacent segments have identical fundamental frequencies and are excited by ground motions of soft soil condition with weak correlation. Spatial variation of ground motions is critical in determining relative displacement, and hence should be considered in the current NZTA Bridge manual.

2. Two equations were proposed for estimating the minimum seating length accounting for spatial variation of ground motions. A procedure of using the proposed equations was given.

9.2 Recommendations for future studies

For future studies regarding the effect of spatially varying ground motions, the following is recommended.

1. The bridge model can be designed with different pier height, different spans or even different slenderness to cover a range of possible types of bridges.

2. Skewed bridges and curved bridges are worth researching.

3. When investigating a multi-segment bridge with pounding, abutments could be included. A holistic consideration is useful when the inelastic bridge behaviour and pounding are incorporated.

4. In reality, an earthquake can excite the bridge from different directions. Therefore, excitations in the transvers and vertical directions should to be considered.

5. Pounding-induced damage to girder ends and abutments should be considered in the investigation.

6. Bridge embankment is suggested to be included.

7. It is suggested to present the results in probabilistic manner, such as fragility functions.
References


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


References


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References


Appendix

The parameters of the prototype structure were calculated as follows:

1. PIER CALCULATION:

Pier dimensions – 3440 × 1480 mm

\( f'_c = 30 \text{ MPa} \)

\( E_c = 3320 \sqrt{f'_c} + 6900 = 25084 \text{ MPa} \)

\( f_s = 500 \text{ MPa} \)

\( E_s = 200 \text{ GPa} \)

Longitudinal reinforcement = \( \phi 32 \text{ mm} \)

Due to symmetrical placement of longitudinal reinforcement, \( I_c \) will not be affected by whether there is positive or negative bending occurring in the columns.

Referring to the drawing (02-045) for Newmarket Viaduct-

**SECTION ①:**

\[ A_s' = A_s = 68 \times \left( \frac{\pi \times 32^2}{4} \right) = 54689 \text{ mm}^2 \]

\[ n = \frac{E_s}{E_c} = \frac{200 \times 10^9}{25084 \times 10^6} = 7.97 \]

\[ (n-1)A_s' = 6.97 \times 54689 = 381182 \text{ mm}^2 \]

\[ nA_s = 7.97 \times 54689 = 435871 \text{ mm}^2 \]
Appendix

Find position of neutral axis:

\[ \Sigma M_{\text{NA}} = 0 \]

\[ A_e \cdot y \cdot \frac{y}{2} + (n - 1) A_{s'} (y - 156) = nA_s (1324 - y) \]

\[ 3440 \cdot y \cdot \frac{y}{2} + 381182 \times (y - 156) = 435871 \times (1324 - y) \]

\[ 1720y^2 + 817053y - 636557596 = 0 \]

\[ y = 416 \text{ mm} \]
\[
I_{cr} = \left[ \frac{3440 \times 416^3}{12} + 3440 \times 416 \times \left( \frac{416}{2} \right)^2 \right] + 381182 \times (416 - 156)^2 + 435871 \times (1324 - 416)^2 \\
= 8.255 \times 10^{10} + 2.577 \times 10^{10} + 3.594 \times 10^{11} \\
= 4.677 \times 10^{11} \text{ mm}^4 \\
I_y = \frac{3440 \times 1480^3}{12} = 9.293 \times 10^9 \text{ mm}^4
\]

**SECTION ②:**

\[
A_s' = A_s = 37 \times \left( \frac{\pi \times 32^2}{4} \right) = 29757 \text{ mm}^2
\]

\[
n = \frac{E_s}{E_c} = \frac{200 \times 10^9}{25084 \times 10^6} = 7.97
\]

\[
(n - 1)A_s' = 6.97 \times 29757 = 207406 \text{ mm}^2
\]

\[
nA_s = 7.97 \times 29757 = 237163 \text{ mm}^2
\]

\[
29757x = 25 \times \frac{\pi \times 32^2}{4} \times 660 + 6 \times \frac{\pi \times 32^2}{4} \times 560 + 2 \times \frac{\pi \times 32^2}{4} \times 470 + 2 \times \frac{\pi \times 32^2}{4} \times 280 \\
+ 2 \times \frac{\pi \times 32^2}{4} \times 160
\]

\[
x = 585 \text{ mm}
\]

\[
d' = 1480/2 - 585 = 155 \text{ mm}
\]

\[
d = 1480 - 155 = 1325 \text{ mm}
\]
\( \Sigma M_{NA} = 0 \)

\( A_c \cdot y \cdot \frac{y}{2} + (n - 1)A_s \cdot (y - 155) = nA_s(1325 - y) \)

\[3440 \cdot y \cdot \frac{y}{2} + 207406 \times (y - 155) = 237163 \times (1325 - y)\]

\[1720y^2 + 444569y - 34638905 = 0\]

\( y = 338 \text{ mm} \)

\[I_{cr} = \left[ \frac{3440 \times 338^3}{12} + 3440 \times 338 \times \left( \frac{338}{2} \right)^2 \right] + 207406 \times (338 - 155)^2 + 237163 \times (1325 - 338)^2\]

\[= 3.324 \times 10^{10} + 6.95 \times 10^9 + 2.31 \times 10^{11}\]

\[= 2.71 \times 10^{11} \text{ mm}^4\]

\(I_{cr}\) of total column:

SECTION ①: \( I_{cr} = 4.677 \times 10^{11} \text{ mm}^4, L = 4900 \text{ mm}\) (based on centreline measurement)

SECTION ②: \( I_{cr} = 2.71 \times 10^{11} \text{ mm}^4, L = 6400 \text{ mm}\) (based on centreline measurement)

SECTION ③: \( I_{cr} = 4.677 \times 10^{11} \text{ mm}^4\) (same as SECTION ① - ignoring flared sections),

\( L = 4200 \text{ mm}\) (based on centreline measurement)

Total column height = 15500 mm

\[I_{cr\text{total}} = \frac{4900}{15500} \times 4.677 \times 10^{11} + \frac{6400}{15500} \times 2.71 \times 10^{11} + \frac{4200}{15500} \times 4.677 \times 10^{11}\]

\[= 3.86 \times 10^{11} \text{ mm}^4\]

For the total column section:

\[I_c = \frac{3440 \times 1480^3}{12} = 9.293 \times 10^{11} \text{ mm}^4\]

\[I_{cr} = 3.86 \times 10^{11} \text{ mm}^4\]
Appendix

\[ f_r = 0.6\sqrt{f_c} = 0.6 \times 1 \times \sqrt{30} = 3.29 \text{ MPa} \]

\[ y_t = d - y \]

SECTION ①: \( y = 416 \text{ mm}, d = 1324 \text{ mm}, y_t = 1324 - 416 = 908 \text{ mm}, L = 4900 \text{ mm} \)

SECTION ②: \( y = 338 \text{ mm}, d = 1325 \text{ mm}, y_t = 1325 - 338 = 987 \text{ mm}, L = 6400 \text{ mm} \)

SECTION ③: \( y = 416 \text{ mm}, d = 1324 \text{ mm}, y_t = 1324 - 416 = 908 \text{ mm}, L = 4200 \text{ mm} \)

For total column: \( y_t = \frac{4900}{15500} \times 908 + \frac{6400}{15500} \times 987 + \frac{4200}{15500} \times 908 = 940 \text{ mm} \)

\[ M_{cr} = f_r \frac{I_y}{y_t} = 3.29 \times \left( \frac{9.293 \times 10^{11}}{940} \right) = 3250 \text{ kNm} \]

\[ I_c = \left( \frac{M_{cr}}{M_a} \right)^3 I_a + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \]

\[ I_c = \left( \frac{3250}{24834} \right)^3 \times 9.293 \times 10^{11} + \left[ 1 - \left( \frac{3250}{24834} \right)^3 \right] \times 3.86 \times 10^{11} = 3.871 \times 10^{11} \text{ mm}^4 \]

Newmarket Viaduct - Pier Bending Capacity:

Pier Dimensions: \( 3440 \text{ mm} \times 1480 \text{ mm} \)

30 MPa concrete

SECTION ①:

\[ A_s = A_s' = 68 \times \left( \frac{\pi \times 32^2}{4} \right) = 54689 \text{ mm}^2 \]

Assume all steel is yielding

\[ \frac{1480}{2} = 740 \text{ mm} \]
Appendix

\( f_y = 500 \text{ MPa (HD bars)} \)
\( T = A_s f_y = 54689 \times 500 = 27345 \text{ kN} \)
\( C_s = A_s f'_s \gamma_s \text{ eff} \)
\( \gamma_s \text{ eff} = f'_s - \alpha f'_c \)

Assume compression reinforcement yields \( f'_s = 500 \text{ MPa} \)

For 30 MPa concrete \( \alpha = 0.85 \)
\( f'_s \text{ eff} = 500 - 0.85 \times 30 = 474.5 \text{ MPa} \)
\( C_s = A_s f'_s \gamma_s \text{ eff} = 54689 \times 474.5 = 25950 \text{ kN} \)
\( C_c = \alpha f'_c ab = 0.85 \times 30 \times a \times 3440 = 87.72a \text{ kN} \)
\( N_n = 10244 \text{ kN} \) (assuming self-weight only of structure)
\( C_c = T + N_n - C_s = 27345 + 10244 - 25950 = 11639 \text{ kN} \)
\( 11639 = 87.72a \therefore a = 132.7 \text{ mm} \)
\( c = \frac{a}{\beta} = \frac{132.7}{0.85} = 156.1 \text{ mm} \)

\[ \varepsilon_s = 0.003 - \frac{0.003}{c} d' = 0.003 - \frac{0.003}{156.1} \times 156 = 1.92 \times 10^{-6} \leq 0.0025 \]
\( (\varepsilon_y = \frac{500 \text{ MPa}}{200 \text{ GPa}} = 0.0025) \)
NG \( \rightarrow \) not yielding
\( \varepsilon_s = \frac{0.003}{156.1} \times 1324 - 0.003 = 0.022 \geq 0.0025 \) \( \text{OK} \rightarrow \) yielding
Refer Excel spread sheet for iterations to find consistent values.

**Moment Capacity of Newmarket Viaduct Piers – SECTION ①**

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<th>Cc (kN)</th>
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\[ T = 27345 \text{ kN} \]
\[ Cs = 14704 \text{ kN} \]
\[ Cc = 22885 \text{ kN} \]
\[ Nn = 10244 \text{ kN} \]
\[ a = 261 \text{ mm} \]

**Force Equilibrium:**
\[ T + Nn - Cc - Cs = 0 \quad \text{OK} \]

**SECTION ②:**
\[ A_s = A_s' = 37 \times \left(\frac{\pi \times 32}{4}\right) = 54689 \text{ mm}^2 \]
\[ d = 1325 \text{ mm} \]
\[ d' = 155 \text{ mm} \]
Assume all steel is yielding:
Appendix

\[
\frac{1480}{2} = 740 \text{ mm}
\]

\[
\begin{align*}
T &= A_s f_y = 29757 \times 500 = 14879 \text{ kN} \\
C_s &= A_s' f_{s' \text{ eff}} \\
f_{s' \text{ eff}} &= f_{s'} - \alpha f_c' \\
&= 500 - 0.85 \times 30 = 474.5 \text{ MPa} \\
N_n &= 10244 \text{ kN (assuming self-weight only of structure)} \\
C_c &= T + N_n - C_s = 14879 + 10244 - 14120 = 11003 \text{ kN} \\
11003 &= 87.72a \\
\therefore a &= 125.4 \text{ mm}
\end{align*}
\]

\[
\begin{align*}
\varepsilon_s' &= 0.003 - \frac{0.003}{146.5} \times 155 = -1.74 \times 10^{-4} \leq 0.0025 (\varepsilon_y = \frac{500 \text{ MPa}}{200 \text{ GPa}} = 0.0025) \\
\text{NG} &\rightarrow \text{ not yielding} \\
\varepsilon_s &= \frac{0.003}{146.5} \times 1325 - 0.003 = 0.024 \geq 0.0025 \quad \text{OK} \rightarrow \text{ yielding}
\end{align*}
\]
Refer Excel spread sheet for iterations to find consistent values.

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| T=     | 14879  kN |
| Cs=    | 6058   kN |
| Cc=    | 19065  kN |
| Nn=    | 10244  kN |
| a=     | 217    mm |

Force Equilibrium:

\[ T+Nn-Cc-Cs = 0 \quad \text{OK} \]

Pier Moment Capacities:

SECTION ①:

Referring to diagram under SECTION ① and the spread sheet

\[ \Sigma M^+ = 27345 \times (1324 - 740) + 14704 \times (740 - 156) + 22885 \times (740 - 274/2) = 38356 \text{kNm} \]

SECTION ②:

Referring to diagram under SECTION ② and the spread sheet

\[ \Sigma M^+ = 14879 \times (1325 - 740) + 6058 \times (740 - 155) + 19065 \times (740 - 232/2) = 24145 \text{kNm} \]

Using a weighted average of each section to find overall moment capacity:

SECTION ①: \[M_n = 38356 \text{kNm}; \quad L = 4900 \text{ mm} \]

SECTION ②: \[M_n = 24145 \text{kNm}; \quad L = 6400 \text{ mm} \]
SECTION ③: $M_n = 38356 \text{kNm}; L = 4200 \text{mm}$

Total column height = 15500 mm

$$\Phi M_{n,\text{total}} = 0.85 \times \left( \frac{4900}{15500} \times 38356 + \frac{6400}{15500} \times 24145 + \frac{4200}{15500} \times 38356 \right) = 27615 \text{kNm}$$
Appendix

Newmarket Viaduct – Structure self-weight per segment:

**Deck self-weight:**

![Diagram of deck self-weight](image)

- \( A_1 = 14850 \times 240 = 3564000 \text{ mm}^2 \)
- \( A_2 = 0.5 \times 1900 \times 310 = 294500 \text{ mm}^2 \)
- \( A_3 = 310 \times 200 = 77500 \text{ mm}^2 \)
- \( A_4 = 0.5 \times 1200 \times 310 = 186000 \text{ mm}^2 \)
- \( A_5 = 200 \times \sqrt{2550^2 + 775^2} = 533034 \text{ mm}^2 \)
- \( A_6 = 5100 \times 200 = 1020000 \text{ mm}^2 \)
- \( A_7 = 0.5 \times 450 \times 450 = 101250 \text{ mm}^2 \)

Note: The area of \( A_7 \) was approximated as it is small compared to large sections.

\[
A_{\text{total}} = 3564000 + 2 \times (294500 + 77500 + 186000 + 533034 + 101250) + 1020000 \\
= 6.969 \text{ m}^2
\]

Span length = 100 m
Density of concrete = 24 kN/m$^3$

Concrete box-girder deck weight = $6.969 \times 24 = 167$ kN/m

For one span deck weight = $100 \times 167 = 16700$ kN

**Column self-weight:**

Height = 15500 mm

Volume of concrete = $15500 \times 3440 \times 1480 = 78.91$ m$^3$

Each column has total weight = $78.91 \times 24 = 1894$ kN

For one complete bridge segment, structure weight = $2 \times 1894 + 16700 = 20488$ kN

Seismic weight of the structure (deck + $\frac{1}{2}$ column) = $16700 + 1894 = 18594$ kN

**Newmarket Viaduct – Deck Section Stiffness**

$I_{total} = I + Ay^2$

Using the section areas calculated under the Structure self-weight calculations:

$I_{total} = I_1 + A_1 y_1^2 + 2 \times \left( \sum_{n=2}^{5} I_n + A_n y_n^2 \right) + (I_6 + A_6 y_6^2) + 2 \times (I_7 + A_7 y_7^2)$

Deck centroid:

$6.969 \times 10^6 \bar{y} = 3564000 \times \left( 3100 \times \frac{240}{2} \right) + 2 \times 294500 \times \left( 2550 + \frac{2}{3} \times 310 \right) + 2 \times 77500 \times \left( 2550 + \frac{310}{2} \right)$

$+ 2 \times 18600 \times \left( 2550 + \frac{2}{3} \times 310 \right) + 2 \times 533034 \times \frac{2550}{2} + 1020000 \times \frac{200}{2}$

$+ 2 \times 101250 \times \left( 200 + \frac{450}{2} \right)$

$\bar{y} = 2052$ mm (from deck soffit)
Appendix

\[ I_{\text{total}} = \frac{14850 \times 240^3}{12} + 3564000 \times (2980 - 2052)^2 + 2 \times \left[ \frac{1500 \times 310^3}{36} + 294500 \times (2757 - 2052)^2 \right] + 2 \times \left[ \frac{200 \times 310^3}{12} + 77500 \times (2705 - 2052)^2 \right] + 2 \times \left[ \frac{1200 \times 310^3}{36} + 186000 \times (2757 - 2052)^2 \right] \]

\[ + 2 \times \left[ \frac{200 \times 2550^3}{12} + 533034 \times (2052 - 1275)^2 \right] + 2 \times \left[ \frac{5100 \times 200^3}{12} + 1020000 \times (2052 - 100)^2 \right] \]

\[ + 2 \times \left[ \frac{450 \times 450^3}{36} + 101250 \times (2052 - 350)^2 \right] \]

\[ = 9.337 \times 10^{12} \text{ mm}^4 \]

Newmarket Viaduct – Bending stiffness:

Using Equation 1.3.5 from “Dynamics of Structures” (Chopra, 2007)

\[ k = \frac{24EI_c}{L^3} \frac{12\rho + 1}{12\rho + 4} \]

where \( \rho = \frac{I_b}{4I_c} \)

N.B.: This accounts for both deck and pier stiffness contributions to the structural bending stiffness.

For concrete used in the Newmarket Viaduct bridge, \( E = 30 \text{ GPa} \)

\( I_c = 3.871 \times 10^{11} \text{ mm}^4 \)

\( I_b = 9.337 \times 10^{12} \text{ mm}^4 \)

\( \rho = \frac{9.337 \times 10^{12}}{4 \times 3.871 \times 10^{11}} = 6.03 \)

\[ k = \frac{24 \times 30000 \times 3.871 \times 10^{11}}{15500^3} \frac{12 \times 6.03 + 1}{12 \times 6.03 + 4} = 71851 \text{ kN/m} \]
Appendix

Fundamental frequency of the structure (for each segment)

\[ \omega = \sqrt{\frac{k}{m}} \]

Structure self-weight = 18594 kN

\[ m = \frac{18594000}{9.81} = 1895413 \text{ kg} \]

\[ \omega = \sqrt{\frac{71851}{1895413}} = 6.16 \text{ rad/s} \]

\[ T = \frac{2\pi}{\omega} = \frac{2\pi}{6.16} = 1.02 \text{ s} \]

\[ f = 0.98 \text{ Hz} \]

Newmarket Viaduct – Summary of parameters

- \( I_{\text{column}} = 3.871 \times 10^{11} \text{ mm}^4 \)
- \( I_{\text{deck}} = 9.337 \times 10^{12} \text{ mm}^4 \)
- \( k_{\text{total}} = 71851 \text{ kN/m} \)
- Total weight of one segment: 20488 kN
- Total seismic weight of one segment: 18594 kN
- Fundamental frequency: \( f = 0.98 \text{ Hz} \)