Strong motion records from the Thorndon Overbridge in the 2013 Cook Strait and Lake Grassmere earthquakes

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ABSTRACT: The 1.35 km long Thorndon Overbridge is located on State Highway (SH) 1 about 2 km north of the central business area of Wellington. It crosses Aotea Quay and the Wellington Railway Yard and is on a critical lifeline route into Wellington.

Sixteen tri-axial accelerometers were installed on the Overbridge during 2010-11 as part of the GeoNet Structures Instrumentation Programme. The instruments recorded both the 21 July 2013 Cook Strait and the 16 August 2013 Lake Grassmere earthquakes and the more recent 20 January 2014 Eketahuna Earthquake. These are the first strong-motions recorded on a New Zealand highway bridge and add to the very limited database of strong-motions recorded worldwide on long bridges.

The paper presents comparisons between the amplification of the response along the two principal directions of the bridge and the ground motions and superstructure response accelerations at two locations spaced 400 m apart. Periods of vibration evident in the recorded response of the bridge are compared with computed periods and estimates of the damping were made from the acceleration response. The amplifications in the transverse direction of the bridge, based on peak accelerations, varied between 2.8 to 3.6 and were much greater than in the longitudinal direction where amplifications were less than 1.4. Although spatial variability apparently did not reduce the transverse response it clearly had a significant influence on the longitudinal response.

1 INTRODUCTION

The Thorndon Overbridge is one of three bridges that have recently been instrumented with strong motion accelerometers as part of the GeoNet Structures Instrumentation Programme. The other two bridges are on SH 74 which links Christchurch city to the Port of Lyttelton. The installation of the instruments on these two bridges was completed in August 2013.

The structural array programme of the GeoNet project aims to install multiple seismic instruments in approximately 30 representative buildings (commercial and residential) and bridges throughout New Zealand to gain insight into the earthquake performance of typical structures. One of the main objectives of the instrumentation of structures is to improve analysis procedures so that they provide more reliable predictions of the influence of such factors as non-linear structural behaviour and soil-structure interaction on the response. Typical and representative samples of buildings and bridges have been selected for instrumentation in areas of high seismic hazard. In addition to the three bridges installations have been completed on nine buildings in Wellington, Levin, Christchurch, Nelson, and Napier. (http://info.geonet.org.nz/)

Major bridges are often extended structures greater than 100 m long, are often located on recent alluvium in river valleys, and frequently have deep foundations. In contrast to typical buildings, their response under earthquake loading is strongly influenced by the variability in ground motion inputs along their length and the effects of soil-structure interaction at the abutments and in the pier foundations (Monti et al, 1994).

Spatial variability of ground motions may have a significant influence on response of bridges in earthquakes. Under uniform soil conditions, these variations arise mainly because of the wave-passage effect, resulting from the time lag in the arrival of seismic waves at separate locations, and the incoherence effect, resulting from random differences in the amplitudes and phases of seismic waves at separate locations, caused by reflections and refractions and by the differential super-positioning of waves originating from different parts of the source. In the case of varying soil conditions, there is additionally the site-response effect, which results in variations in amplitudes and frequency contents of surface motions caused by the propagation of bedrock motions through soil profiles with different stiffness characteristics (Konakli and Der Kiureghian, 2011a). The effects of spatial variability of input motions on the response of long bridges has been investigated by a number of recent numerical studies but there do not appear to be any previous records of the response of long bridges in strong ground shaking (Sextos et al, 2003; Wang et al, 2003).

2 THORNDON OVERBRIDGE

2.1 Structure

The 1.35 km long Overbridge consists of twin three-lane parallel structures with similar construction details for the 36 simply supported spans of each structure. Constructed between 1967 and 1972 along the original foreshore of Wellington Harbour it carries SH 1 over the main trunk railway, an extensive area of rail yards, the Inter Islander Ferry Terminal and two important access roads into Wellington City; Aotea Quay and Thorndon Quay. The southbound and northbound carriageways are typically 11.5 m wide and carry 3 x 3.5 m traffic lanes plus 0.5 m shoulders. On and off ramps provide access to Aotea Quay midway along the bridge (see Figure 1). The ramp structures are continuous prestressed concrete box structures essentially isolated from the main bridge by flexible elastomeric bearings at the end span support locations on the main bridge.

The superstructure of the overbridges consists of simply supported precast prestressed concrete I-beams with a monolithic reinforced concrete deck. Most of the spans are supported using half joints on large cellular reinforced or prestressed concrete box type pier caps (umbrellas) having single circular column piers (see Figure 2). The heights of the structures vary with the northern most spans ramping down to reclamation level. Seven of the most northern spans on the southbound structure and four on the northbound structure are supported on multi-column portal frames. Apart from these northern spans, most of the northern half of the structures have a height from the top of the pile caps to the deck surface of approximately 10 m. The corresponding height on the southern sections (over the rail yards) is appproximately 14 m with a maximum height of 14.3 m.

On the northern half of the structures the beam spans vary from 19.8 to 28.7 m with an overall structure depth of 1.82 m. On the southern half beam spans vary from 19.8 to 41.5m with an overall structure depth of 2.55 m. The span of the pier caps between the beam support points varies but is typically about 9 m. The maximum centre to centre span between the pier columns is 51 m. Long linkage bolts tie the beam spans onto the pier caps. A single pier cap located near the centre of the bridge is the only point along the length where the northbound superstructure is structurally connected to the southbound superstructure. Wider pier caps at the junctions of the on- and off-ramps have two circular columns instead of the typical single column.

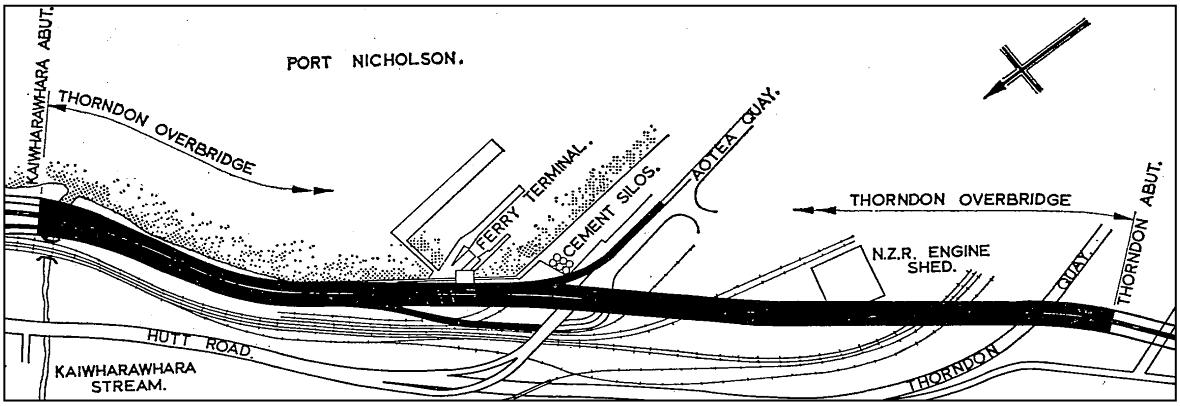

Figure 1. General layout of Thorndon Overbridge.

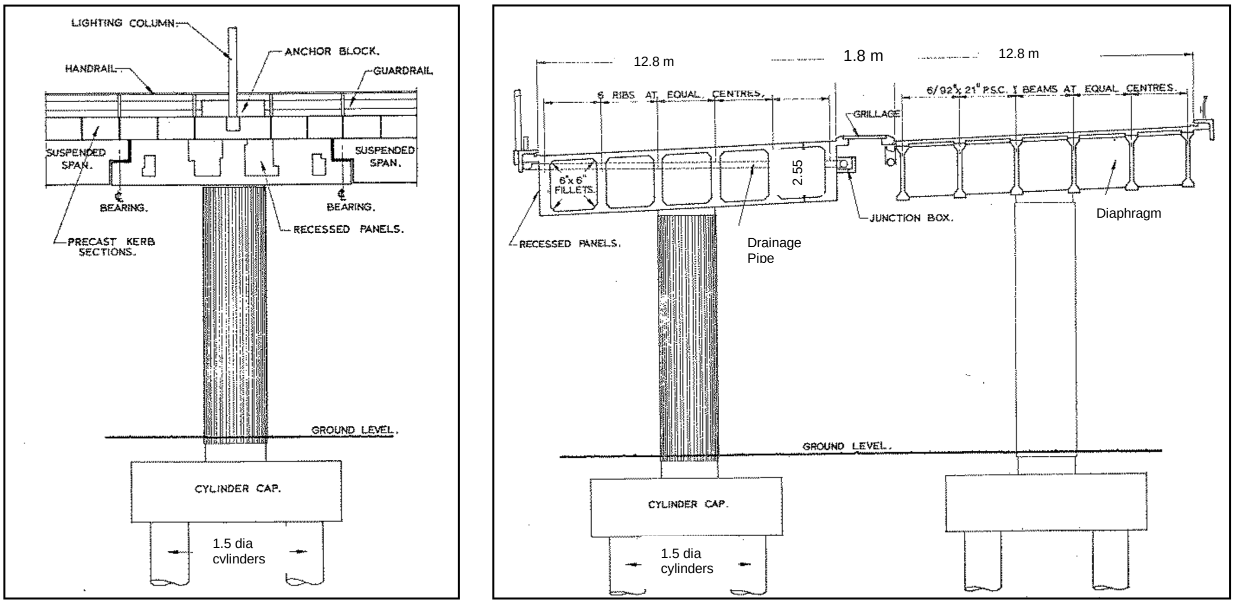

Figure 2. Elevation of pier and sections of pier and beam spans on 2.55 m deep superstructure section.

2.2 Foundations and Seismicity

The shallower spans on the northern half of the superstructure are founded on driven 600 mm diameter steel shell piles with most of the single column piers supported on 12 piles. The single column piers on the deeper southern section spans are supported on four 1.5 m diameter bored piles (cylinders).

The Overbridge is constructed over reclaimed land which is considered to be prone to liquefaction in strong earthquake shaking. Reclamations on the harbour perimeter were carried out in stages, primarily in 1882, 1904, 1924-32, 1960's and 1970. The reclamations typically consist of 4 m to 16 m of gravel rockfill or pumped hydraulic fill and overly a 1 to 2 m layer of silty / gravelly sand Holocene beach and marine sediments (Arianpour, 2012).

The bridge is located in an area of high seismicity and crosses the Wellington-Hutt Valley segment of the Wellington Fault. Between 110 to 310 years have elapsed since the last rupture of this segment, which has a median recurrence interval of approximately 1100 years. (Langridge et al, 2011). Magnitude estimates for earthquakes on the fault vary from 7.3 to 7.9 and ground displacements of up to 5 m horizontal and 1 m vertical are predicted (Stirling et al, 2002; Little et al, 2010).

2.3 Strengthening Work

Extensive seismic strengthening work was carried out on the Overbridge during 1996 to 1998. The work was directed at preventing collapse of the bridge in a Wellington Fault event.

All 72 pile caps were strengthened to ensure that plastic hinging forms in the pier columns rather than in the pile caps and piles. The pile caps adjacent to the sea wall near the Ferry Terminal were tied together using post tensioned foundation side beams supported on two additional 1.5 m diameter concrete bored piles with belled ends. This strengthening was designed to provide gravity support to one of the pier columns as well as preventing the eastern most pile cap from moving towards the sea in the event that the old sea wall collapsed (Arianpour, 2012).

Analysis of the columns showed inadequate confinement reinforcement in the lower sections of the columns and indicated that premature shear failures could occur at the termination points of the longitudinal reinforcement. All pier columns were jacketed with a 12 mm thick x 2.4 m high steel lower section. An additional 10 mm thick steel jacket, welded to the top of the lower section and extending to within 300 mm of the pier caps, was installed on most of the columns.

Linkage improvements involved replacement of some of the existing bolts linking the spans across the pier caps with slack high strength bolts. These were designed to reduce the risk of an unseating collapse from distributed ground movements expected in the reclamation soils following rupture of the Wellington Fault.

Steel catch frames were attached to the pier caps on eight piers adjacent to where the Wellington Fault crosses the axis of the bridge to support the beam spans if they are unseated by the large permanent ground movements expected in a fault rupture (Arianpour, 2012).

3 INSTRUMENT LOCATIONS

The accelerometers were located in the two zones shown in Figure 3, with eight tri-axial accelerometers in each zone. The zones were approximately 400 m apart. Within each Zone, seven accelerometers were located on the superstructure and one accelerometer on the concrete base slab of the control cabinet which was at ground level and separated from the adjacent pier column by a gap of 50 mm. In each Zone the accelerometers were located on two adjacent piers and on two adjacent spans. The accelerometer locations in Zone II are summarised in Table 1.

Instruments were located at similar positions on Piers 25 and 26, and Spans 24 - 25 and 25 - 26 in Zone I. Further details of the instrument locations are shown in Figure 4.

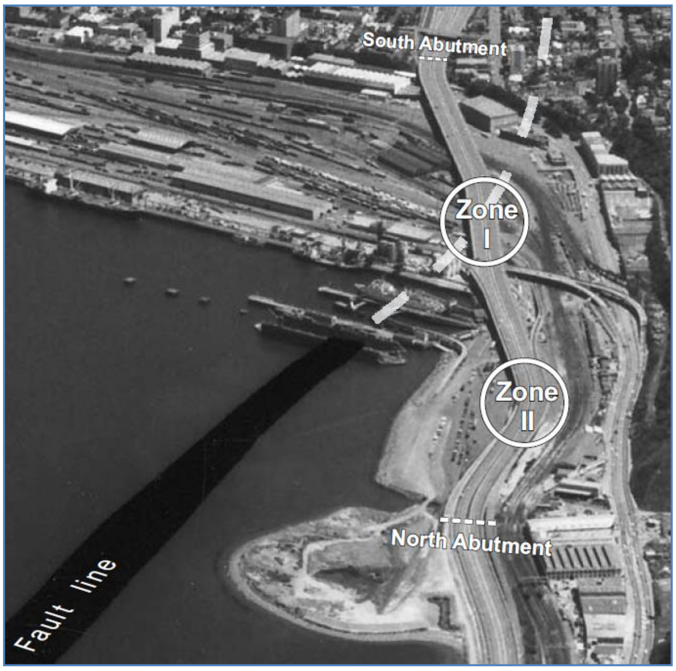

Figure 3. Location of instrument zones and Wellington Fault.

Figure 4. Accelerometers and GPS instruments on Pier 14 cap.

Table 1. Instrument Locations in Zone II

Instrument	Pier or Span	Location Description
1	Pier 13	Pier cap - centre
2	Span 13-14	Centre of span
3	Span 13-14	End of span - W side
4	Pier 14	Pier cap - E side
5	Pier 14	Pier cap - W side
6	Span 14-15	End of span - E side
7	Span 14-15	Centre of span
8	Pier 14	Ground level

4 RECORDED ACCELERATIONS

4.1 Transverse Response Accelerations

Acceleration time-histories for the transverse response of the bridge in the Cook Strait Earthquake are shown in Figure 5. The response accelerations were very similar at all the superstructure positions within each Zone although there were significant differences between each Zone with different modes of vibration dominating at the two locations. Peak response accelerations, based on the average over the seven instruments on the superstructure, were 0.36 g and 0.34 g in Zone I and II respectively. The periods of vibration in the transverse direction were in the range 0.5 to 0.9 seconds.

4.2 Longitudinal Response Accelerations

Acceleration time-histories for the longitudinal response of the bridge in the Cook Strait Earthquake are shown in Figure 6. The average peak response in the Zone II longitudinal direction was 0.17 g or 50% of the transverse peak response acceleration of 0.34 g. Although there were differences in the transverse and longitudinal ground motion inputs (see Figure 8 and Section 4.4) these differences were small in comparison to the difference in the response accelerations in the two principal directions. The response in the longitudinal direction appeared to be more influenced by lack of coherency and time lag in the input motions along the length of the bridge than was the case for the transverse response.

4.3 Comparison of Ground and Response Accelerations

A comparison between the ground motions and the response time-histories recorded at Zone I is shown in Figure 7 for both the longitudinal and transverse bridge directions. The amplification of the transverse ground motions was significantly greater than for the longitudinal direction with amplification in the transverse direction of 2.8 compared to 1.4 in the longitudinal direction (based on the average of the peak accelerations recorded by superstructure instruments).

4.4 Comparison of Ground Motions at the Two Zones

A comparison of the ground motion components recorded in each Zone for the two bridge axes directions is shown in Figure 8. In the transverse direction there was some loss of coherency between the recording points located 400 m apart. The peaks were generally lower in Zone II. In the longitudinal direction there was a significant loss of coherency with larger peaks occurring at Zone I than at Zone II in the latter part of the record. In the longitudinal direction the input records may have been influenced by interaction between the pier columns and the concrete cabinet bases. Because the cabinets housing the ground instruments were more aligned with the pier columns in the longitudinal direction this effect would probably be greater in this direction.

4.5 Peak Accelerations and Amplifications

The peak response and ground accelerations recorded in each direction of the bridge axes at the two zones in both the Cook Strait and Lake Grassmere Earthquakes are summarised in Table 2. The range of the response accelerations on the superstructure is given for each zone together with the average from the seven accelerometers on the superstructure recording in each principal direction. An amplification factor was obtained by dividing the average response acceleration in each of the two directions by the corresponding maximum ground acceleration.

Table 2. Peak Response and Ground Accelerations

Parameter	Cook Strait Earthquake				Lake Grassmere Earthquake			
	Transverse		Longitudinal		Transverse		Longitudinal	
	Zone I	Zone II	Zone I	Zone II	Zone I	Zone II	Zone I	Zone II
Max response acceln, g	0.36	0.34	0.21	0.17	0.24	0.20	0.10	0.16
Min response acceln, g	0.25	0.24	0.16	0.12	0.17	0.13	0.07	0.12
Ave response acceln, g	0.32	0.29	0.17	0.14	0.21	0.18	0.09	0.13
Max ground acceln, g	0.12	0.08	0.12	0.11	0.06	0.06	0.12	0.08
Amplification factor	2.8	3.6	1.4	1.2	3.7	2.9	0.8	1.6

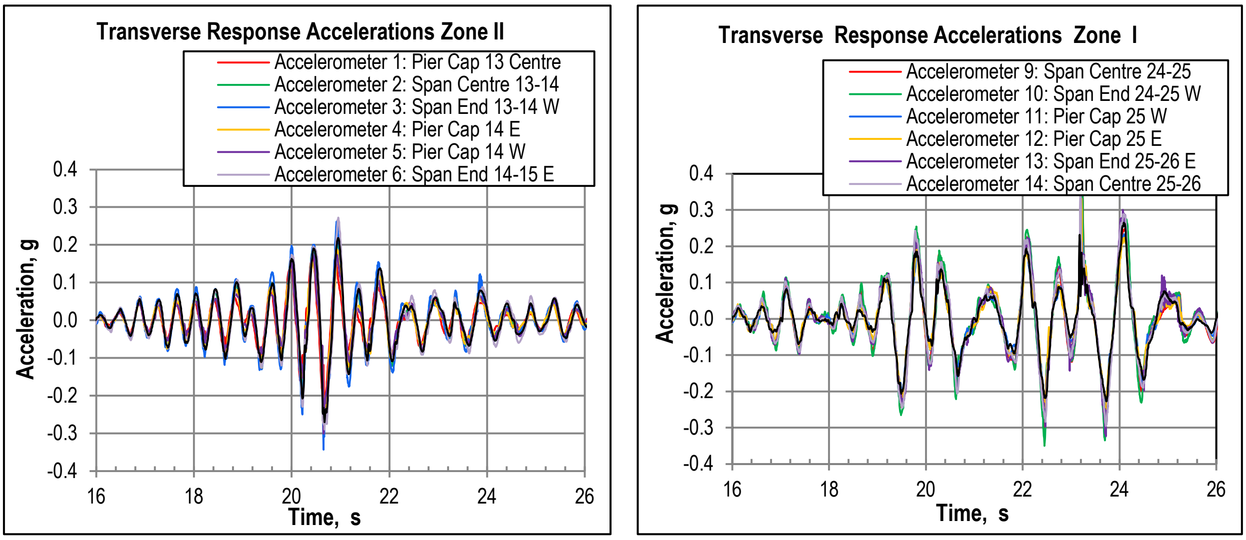

Figure 5. Transverse response accelerations on superstructure at Zone II (left) and Zone I (right).

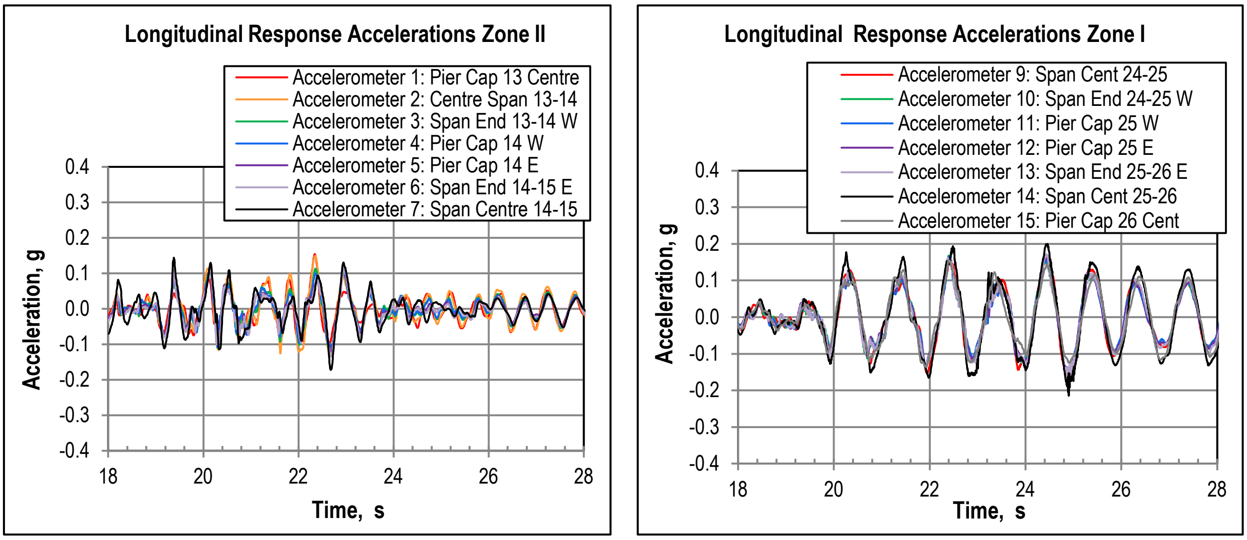

Figure 6. Longitudinal response accelerations on superstructure at Zone II (left) and Zone I (right).

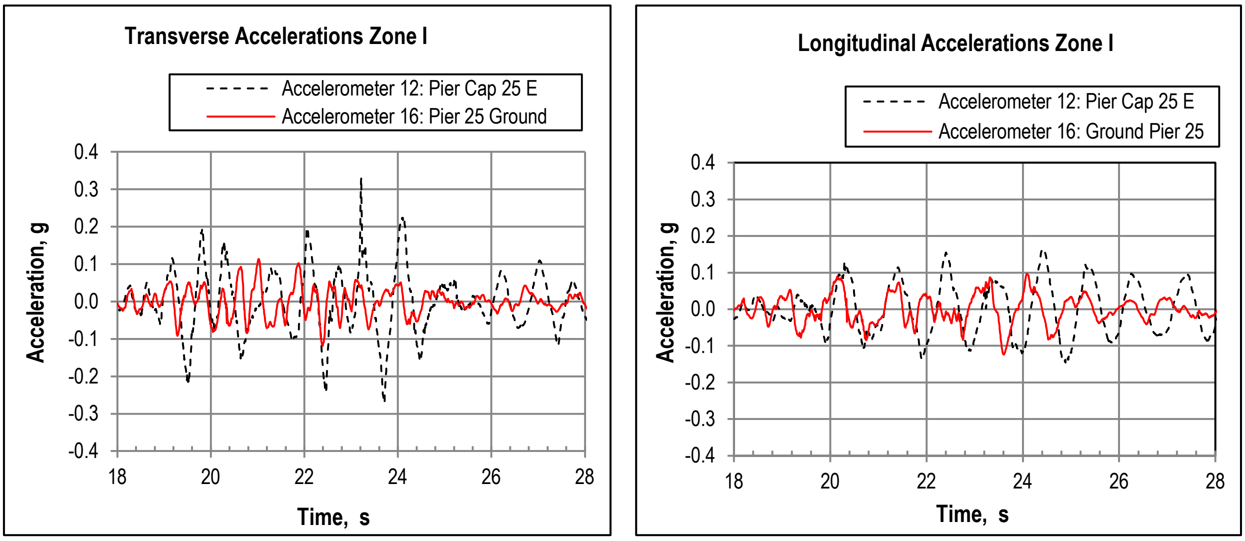

Figure 7. Comparison of response with ground motions for Zone I. Transverse (left) and longitudinal (right).

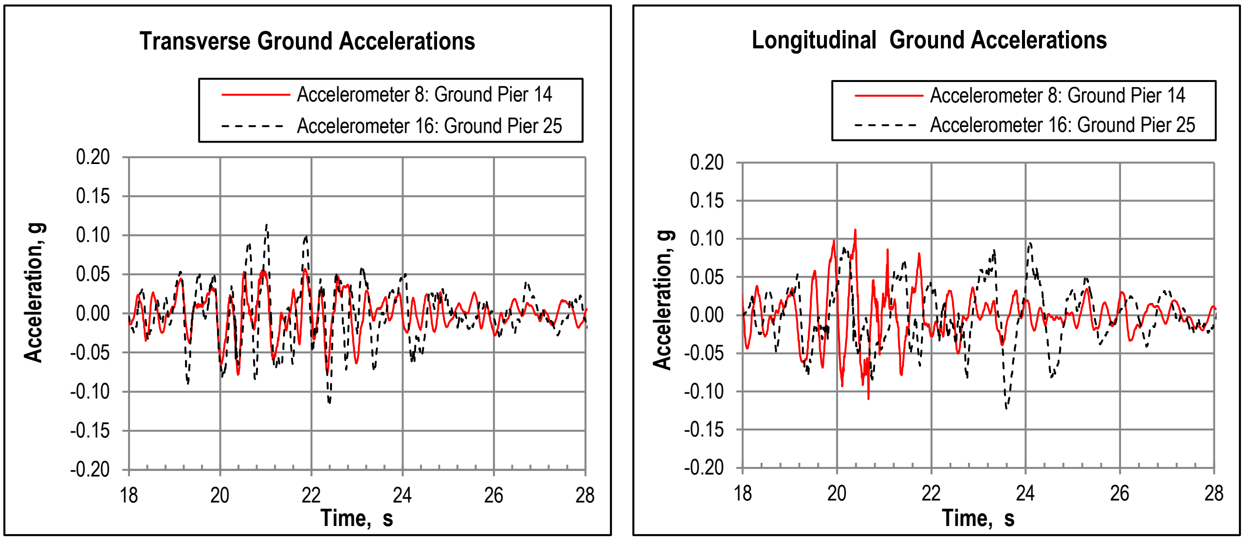

Figure 8. Comparison of ground motions in Zones I and II (400m apart). Transverse (left) and longitudinal (right).

In the transverse direction the ground motions are strongly amplified by the structure with the amplification factors varying from 2.8 to 3.6. By calculating the response spectra from the input motions for various levels of damping and computing the amplification over the period range covering the lowest transverse modes of vibration it was found that the amplifications corresponded to damping values of between 2 to 3%. These are quite low values for a bridge founded on soft soils and the strong amplification suggested that spatial variability in the ground motions was not resulting in a significant reduction in the transverse response.

In the longitudinal direction the ground motions were only weakly amplified by the structure with the amplification factors varying from 0.8 to 1.4. These low values indicate that the variability in the longitudinal ground motions between the two zones was resulting in a large reduction in the longitudinal response.

5 STRUCTURAL MODEL AND PERIODS OF VIBRATION

A detailed three-dimensional model of the bridge is currently being developed. Several simplified two-dimensional models have been set-up to investigate the influence of the steel jackets, foundation flexibility and the variation in the mass and stiffness along the length of the bridge.

The retrofitted steel jackets were found to increase the stiffness of the piers by 25%. The flexibility of the pile foundations was estimated using the elastic continuum methods outlined by Pender (1993) and the pile test results carried out during the design of the bridge (Huizing et al, 1968). On the northern section of the bridge deformations of the 600 mm diameter piles was found to reduce the overall pier stiffness by a factor of 0.3. A large part of this reduction resulted from the translation flexibility of the piles in the soft reclamation soil. On the southern section of the bridge deformations of the 1.5 m bored piles reduced the overall pier stiffness by a factor of 0.6. Both the rotational and translation components of the pile cap displacement were significant but the effective rotational stiffness (calculated at the centre of mass of the superstructure) of the foundation was six times greater than the translational stiffness.

The predominant periods of vibration in the response acceleration time histories were found by computing Fourier spectra of the records and dividing the modulus of the superstructure spectrum by the modulus of the ground motion spectrum. A spectrum ratio plot obtained by this method for the transverse response at Pier 14 (Zone II) in the Cook Strait Earthquake is shown in Figure 9. First and second mode transverse periods of 0.66 and 0.49 seconds are clearly evident in the ratio plot and these are within 11% of the corresponding values of 0.74 and 0.54 seconds computed using simplified structural models.

6 RESPONSE SPECTRA

Response spectra computed from the Overbridge site longitudinal ground motion records are compared in Figure 10 with spectra computed from ground motions recorded on deep layers of soft reclamation soil at building sites on Aotea Quay. The two recording sites, PIPS and BNZ were located approximately 330 m and 1,400 m respectively from the bridge. The acceleration records from these sites were resolved to be in the same directions as the bridge instruments at the Zone I recording location.

There is significant scatter in the spectral accelerations indicating a loss of coherency in the ground motions over moderate distances. Differences in ground motions will also arise from changes in the local geology and soils. There is a significant peak in the Thorndon Overbridge spectrum for accelerometer 8 (Zone II) at a period of 0.45 seconds. There are also similar peaks in the other spectra and it is therefore not possible to assess from the spectra how much the Thorndon Overbridge ground motion records were affected by soil-structure interaction that was considered likely because of the position of the recorders near the base of the piers.

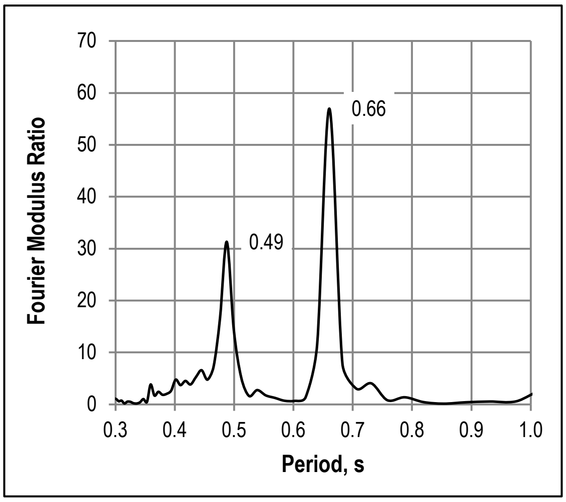

Figure 9. Modulus ratio for Pier 14 transverse motion.

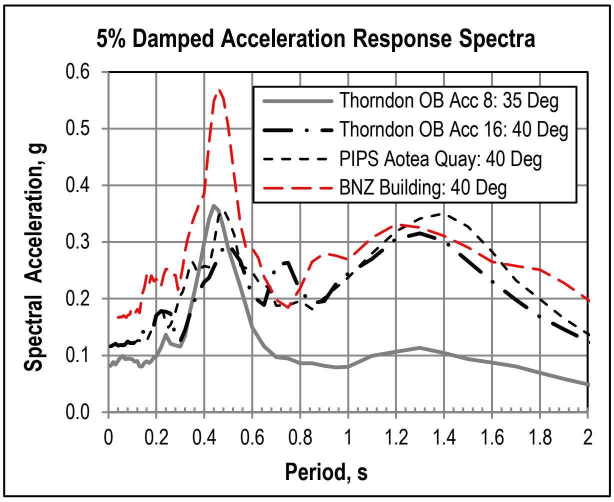

Figure 10. Response spectra for longitudinal direction.

7 CONCLUSIONS

Excellent records of the bridge response and site ground motions have been obtained from three large earthquakes which produced peak ground accelerations at the site of greater than 0.1 g. These are the first strong-motion recordings from a highway bridge in New Zealand and add to the very limited worldwide data available for the response of long bridges in earthquakes.

Preliminary studies of the bridge response to the earthquakes have been completed and will form the basis for a proposal to undertake detailed time-history analyses to compare the recorded response with numerical predictions. Further work is also required to compare the response with simplified design methods of estimating the effects of spatial variability (Der Kiureghian and Neuenhofer,1992; Konakli, and Der Kiureghian, 2011b). The records of the response of the Thorndon Overbridge in the strong shaking at the site in the three recent earthquakes will be valuable for verification and further development of theoretical predictions.

The amplifications in the transverse direction of the bridge were much greater than in the longitudinal direction and although it did not appear that spatial variability had reduced the transverse response it clearly had a significant influence on the longitudinal response. Because of the complexity added by abutment soil-structure interaction, the longitudinal response of long bridges has received little attention in previous studies. The preliminary results from the present study and observations made by Wood et al (2012) on the performance of highway bridges in the 2010-11 Canterbury Earthquakes indicate that this is an area clearly requiring further investigation.

The ground motion instruments are located very close to the piers of the bridge and their records may have been affected by displacements of the bridge foundations estimated to be between 5 mm to 10 mm. A recommendation is to be made to the NZ Transport Agency (NZTA) and GeoNet to install two further instruments located at approximately 10 m from the piers near the present instrumentation clusters.

8 ACKNOWLEDGEMENTS

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