PERFORMANCE OF HIGHWAY STRUCTURES DURING THE DARFIELD AND CHRISTCHURCH EARTHQUAKES OF 4 SEPTEMBER 2010 AND 22 FEBRUARY 2011

Prepared for:

NEW ZEALAND TRANSPORT AGENCY

Final Revision: 26 February 2012

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Report prepared for New Zealand Transport Agency by:

J H Wood, H E Chapman and P Brabhaharan

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ABBREVIATIONS

The following abbreviations are used in this report:

BM	Bridge Manual (see list of References)
BSN	Bridge structure number
CCC	Christchurch City Council
DHC	Double hollow core deck units
DSA	Detailed Seismic Assessment
GeoNet	Geological hazard monitoring system operated by GNS Science http://www.geonet.org.nz/
HBDB	Highway Bridge Design Brief (see list of References)
MMI	Modified Mercalli Scale
M_W	Moment magnitude
M_L	Local magnitude
NZS 1170.5	NZS 1170.5:2004 – Structural Design Actions Part 5: Earthquake Actions - New Zealand
NZDT	New Zealand daylight time
NZST	New Zealand standard time
NZTA	New Zealand Transport Agency
PC	Prestressed concrete
PGA	Peak ground acceleration
RC	Reinforced concrete
RP	Route position on State Highway
S_a	Spectral acceleration
SDOF	Single degree of freedom
SH	State highway
SMA	Strong motion accelerograph

EXECUTIVE SUMMARY

The Earthquakes

The Darfield earthquake, of moment magnitude M_w 7.1, occurred at 04.35 on 4 September 2010 (NZST) and was centred approximately 40 km west of Christchurch city centre at a depth of approximately 11 km. The Christchurch earthquake, of moment magnitude M_w 6.2, occurred at 12.51 on 22 February 2011 (NZDT) and was centred approximately 8 km south-east of Christchurch city centre at a depth of approximately 6 km. Two aftershocks have reached magnitude M_w 6.0. The first of these was recorded at 14.20, 13 June 2011 (NZST) and the second at 15.18, 23 December 2011 (NZDT). The 13 June event was centred approximately 9 km south-east of Christchurch city centre and the 23 December event approximately 9 km to the east of the city centre. Both were at a depth of approximately 7 km.

At the sites of the bridges inspected following the earthquakes the overall averages of the return period levels in the Darfield earthquake, as measured by the equivalent return period of the spectral ordinates averaged over the 0.1 to 0.9 second period range, were between 140 and 580 years. These values indicate, *in theory*, that the pre-1970 bridges probably experienced shaking equivalent to their elastic design intensity, whereas the post-1970 ductile bridges may or may not have reached their design elastic limits, depending on the degree of ductility they were designed for and their corresponding actual elastic strength.

The shaking intensity at the eastern sites, as measured by the equivalent return period of the spectral ordinates averaged over the 0.1 to 0.9 second period range, was greater in the Christchurch earthquake than in the Darfield earthquake. The average return period levels ranged from 280 to 390 years respectively in one area and from 1400 to 5000 years further south, nearer to the epicentre. These values indicate, *in theory*, that the pre-1971 bridges experienced shaking significantly exceeding their elastic design intensity, and the post-1971 ductile bridges most likely reached, and probably exceeded, their design elastic limits.

Purpose of the Investigation

The performance of the highway structures and the ground during the earthquakes is of interest for reviewing the assumptions that are made during the design or seismic assessment and retrofit of structures and their approaches. The authors were therefore asked by New Zealand Transport Agency to visit the area and examine the bridges and the geotechnical effects in detail to identify how they had responded to the shaking.

The Investigation

There are about 50 state highway bridges located within 50 km of the Darfield earthquake epicentre and 30 located within 20 km of the Christchurch earthquake epicentre. Of the total of 50 bridges, 27 individual bridges were inspected as part of the study of the performance of the state highway bridges in the earthquakes.

To assess the strength and performance of the critical components of each of the bridges simple static analyses were carried out. These were generally based on spreadsheet computations. Because of the approximations the results of these analyses could differ from those obtained by more detailed dynamic or non-linear push-over analyses by about ±30%.

To provide for a comparison to be made between the expected and actual performance of the bridges, three values, expressed in terms of gravity acceleration g, were calculated for each structure for both longitudinal and transverse directions:

- The limiting capacity of the bridge's various elements in flexure and shear.
- The minimum capacity that the bridge was assessed to require in terms of the design code current at the date of design.
- The response acceleration predicted for the bridge, taking account of its estimated period of vibration and the spectral accelerations computed from the two closest strong motion accelerograph records shown in Figures 5 and 7 for the Darfield and Christchurch earthquakes respectively.

From these three values, the following ratios were calculated and listed in Tables 5 and 6 for the Darfield and Christchurch earthquakes respectively:

- The minimum ratio of limiting calculated capacity to the required design capacity. This is expressed as "Minimum Ratio Capacity/Design Coefficient".
- The minimum ratio of limiting capacity to response acceleration. This is expressed as "Minimum Ratio Capacity/Demand".

Tables 5 and 6 also contain a summary of data used for these calculations and the assessed return period of the ground motion each bridge was estimated to have experienced.

In addition to the bridges, the ground associated with the bridges and other areas along highways and local roads were also visited and inspected to obtain a full appreciation of the geotechnical issues.

Liquefaction and ground damage behaviour were assessed using current methods at selected bridge sites, and compared with the behaviour observed. From the observations and assessments, lessons are drawn for future investigation, assessment and liquefaction risk mitigation for highway structures and routes.

Contents of the Report

The following main text presents key information in sections on:

- The ground motions assessed to have affected each bridge;
- The damage identified during the inspections;
- The results of approximate structural analyses of each structure and a comparison of the theoretical capacity and assessed demand on each structure;
- Comments on the likely reasons for the damage, and locations of possible damage that could be out of sight below ground;
- Geotechnical engineering aspects including liquefaction;
- Conclusions;
- Recommendations for further inspections that require excavation, and for some more detailed analyses to more closely relate theory and observed performance.

Appendices A, B and C of the report contain detailed information on:

- Ground motions and response spectra;
- Details of the bridges and field observations of the structures' and the ground's performance;
- Assessment of bridge strengths.

Conclusions

1. Analysis of the ground motions in both the Darfield and Christchurch earthquakes indicated that the 1000-year return period design level used for many state highway bridges designed after 1971 was only exceeded at six of the 23 sites inspected (counting the twin bridge sites as a single site). These sites were all close to the epicentre of the Christchurch earthquake. At the 12 sites further than 15 km from the Christchurch earthquake epicentre the ground shaking intensities were greater in the Darfield earthquake than the Christchurch earthquake. Average intensities in the Darfield earthquake ranged from 140 to 580 year return period levels (excluding the HVSC records), so many state highway bridges did not receive the current design level intensity of shaking although all the sites of bridges designed prior to 1971 were subjected to shaking intensities greater than the bridges had been designed for.

2. There was extensive liquefaction in eastern Christchurch and Kaiapoi in the 4 September 2010 Darfield earthquake, and in central and eastern Christchurch in the 22 February 2011 earthquake event. This was generally expected from the liquefaction maps published by Environment Canterbury, but may not have been fully appreciated when most of the bridges and highway structures were designed and built.

3. Rockfall was triggered in the Port Hills area by the September 2010 and the February 2011 earthquakes, but was more extensive in the latter 2011 event. This affected access roads in the Port Hills area and in particular the western portal area of Lyttelton road tunnel. Landslides affected roads in the Port Hills area and near the epicentre of the Darfield earthquake.

4. Earthquake induced liquefaction and associated ground damage, and in particular lateral spreading, was the principal cause of damage to bridges in the earthquake events. Liquefaction generally caused extensive damage to abutments of bridges, but also to the piers closest to the abutments, where lateral spread loads from the river banks imposed loads on the pier foundations.

5. Bridges with abutment walls supporting existing ground underlain by liquefaction prone ground, and supported by many slender piles are particularly prone to damage from liquefaction and consequent lateral spreading, which imposes loads on the abutment structure and foundations.

6. All of the inspected multi-span bridges at sites where significant liquefaction spreading did not occur performed better than predicted by simple analyses based on response spectra computed from the SMA records of the two closest stations and assuming 5% critical damping, which is generally accepted as representative for structural response.

7. It is likely that soil/structure interaction introduces significant energy dissipation between the ground and the foundation members on multi-span structures. For shorter bridges (say up to three spans) the energy absorbed at the abutment/soil interface is likely to be considerable. These effects were intentionally not included in the simple analysis calculations but the apparent better-than-expected performance of the bridges supports this likelihood. The current NZTA Bridge Manual includes a reduction factor (S_p), which is linked to the Site Soil Category and its degree of firmness, in recognition of the likely energy dissipation. For Site Soil Categories C and D, on which most of the bridges are founded, the current values of S_p are 0.8 and 0.7 respectively. There is currently limited evidence of the "accuracy" of the specified values of S_p and more detailed analysis of some of the bridges could provide useful information for this factor.

8. Three of the older multi-span bridges (Old Waimakariri, SH1 Selwyn River and SH77 Selwyn River Bridges) performed significantly better than would be expected, even after allowing for foundation/soil energy dissipation. All three were long multi-span structures (90 to 354 metres). The response of these bridges in the longitudinal direction would be strongly influenced by travelling ground wave effects that result in a phase lag between the seismic input motions at the piers along the length. Travelling wave effects might also reduce the response in the transverse direction by interference between adjacent spans.
9. The heavily skewed, four-span Railway Overbridge, designed in 1962, performed remarkably well, showing no visible evidence of damage, except linkage bolt yielding, despite having apparently been subjected to shaking intensity of 1300 to 1500 years' return period during the Christchurch event. The foundations were not visible for inspection but from ground displacements it seemed unlikely that significant damage was done. It would be useful to inspect some of the critical foundations and to undertake a detailed analysis of the bridge.
10. Thirteen of the 25 state highway bridges inspected received significant structural damage. Two of the eleven suffered serious damage from liquefaction lateral spreading which would be costly repair. With the exception of the Kaiapoi Railway River Bridge (damaged in the Darfield earthquake), the visible damage to the other bridges was relatively minor and not difficult to repair. Possible damage could not be confirmed on some bridges as the bases of the columns and the pile tops were not visible.
11. Excluding the liquefaction-damaged bridges, the actual flexural damage threshold was only exceeded on four of the 22 pre-1972 bridges inspected and this was a factor that limited the extent of the damage. Stronger shaking exceeding the damage threshold levels at more of the pre-1972 bridge sites would have had more serious consequences because of the limited ductility of these older bridges.
12. It seems certain that the retrofitted Chaneys Road, Port Hills Road and Horotane Valley Overpass bridges benefitted from the linkage bars and shear keys that were installed between 2003 and 2010.
13. A number of the bridges' critical members (usually the piles) were not visible for inspection and it is therefore not certain that they were undamaged. It would be useful for a number of the structure foundations to be uncovered and more closely inspected. In some cases damage may be sufficient to require repairs to ensure long term security and durability.
14. The 14 older bridges inspected (constructed in the period 1920 to 1962) were well detailed and constructed, and generally well maintained. This contributed to their overall good performance.
15. There were no indications that current design standards need to be revised to provide an acceptable standard of earthquake performance so perhaps the principal lesson for bridge owners from the earthquake events was the importance of quality of detailing and construction and good maintenance. An exception to this is the need to reinforce, in the design standards, the consideration of liquefaction and lateral spreading as an integral part of bridge design, and incorporation of measures to achieve good performance.
16. The damage caused to abutments by lateral spreading/liquefaction of the soils around them demonstrates the importance of making realistic provision for this risk during the design. The problem is that liquefaction and lateral spreading mitigation measures can be an expensive addition to the cost of the bridge.

17. The approach embankments to the bridges generally performed well with some settlement, despite liquefaction of the surrounding areas, for example, at the CCC Chaneys Road Overpass. Assessment indicates that the overburden pressure from the embankments generally improves the resistance to liquefaction of the shallow soils, and therefore reduces the potential for more extensive damage from lateral spreading. However, the overburden pressure effect from the approach embankments is not always sufficient to prevent extensive lateral spreading, as observed at the Bridge Street Bridge. It is therefore important to appropriately consider the overburden pressure effects from approach embankment in the seismic assessment and design of embankments and associated highway structures.

Recommendations

1. Further inspection of members of some of the bridges should be undertaken, as listed in Table 8. This will require shallow excavations, and dewatering in some cases.
2. Consideration should be given to installing robust inter-span linkages on the City Council Waimakariri River (Main North Road) Bridge. The drawings show only two holding down bolts per span end, with no linkages between the spans. Although a detailed inspection of the bridge was not undertaken no evidence was seen of added security.
3. Detailed analyses of the following bridges would be valuable to better relate the predicted structural performance to the assessed ground motions and observed behaviour of the structures. Investigation and analyses of liquefaction should be included, where appropriate:
   - SH1 Waimakariri River Bridge;
   - SH1 Kaiapoi Railway and River Bridge
   - SH1 Selwyn River Bridge;
   - SH1 Styx Overbridges Nos 1 and 2;
   - SH73 Heathcote River Bridge (Opawa);
   - SH74 Heathcote River Bridge;
   - SH74 Railway Overbridge;
   - SH74A Rutherford Street Bridge (back analysis of abutment pressures);
   - SH74 Port Hills Road Overpasses and SH74 Horotane Valley Overpasses (abutment and embankment stability);
   - SH77 Wairiri Stream Bridge (investigate soil-strain analysis for locked-in bridge);
   - SH77 Selwyn River Bridge (investigate travelling ground wave effects);
   - Anzac Drive and Bridge Street bridges (effects of liquefaction and lateral spreading).

   To gain full value from this report and provide better design information it is important that such detailed analyses are undertaken.
4. Investigation of ground conditions and assessment of the liquefaction at the Bridge Street (CCC), Anzac Drive and Ferrymead (CCC) bridges and the Horotane Valley Overpass to relate the ground damage to the performance of these bridges. Installation of vibrating wire electrical piezometers and inclinometers with data logging capability would be valuable at locations susceptible to liquefaction, as part of the investigations, so that the onset of liquefaction and timing of lateral spreading can be recorded during aftershocks that continue to affect Christchurch.

5. The repairs required on the five significantly damaged SH bridges are:

   - The Kaiapoi Railway and River Bridge diaphragms and linkage bolts. These will be straightforward but nevertheless costly, with complete breakout and reinstatement required on at least all the eight diaphragms at the northern abutment. It may be necessary to strengthen the other diaphragms (88 in total) or upgrade the linkage system to avoid loading them. Further repairs may be necessary if damage to the bases of the pier columns and pier piles is found.
   - The Old Waimakariri River Bridge needs repairs to the abutment walls (backwalls, wing walls and underpinning wall at the south end) and some of the piers. Although the bridge was not inspected by the authors following the Christchurch earthquake significant lateral spreading damage to the southernmost pier was reported. A linkage retrofit is required as mentioned above.
   - The Anzac Drive Bridge and Bridge Street Bridge abutment structures, including their piles, may need to be replaced, with measures to mitigate future liquefaction and lateral spreading. This will be a costly and difficult operation.
   - The Rutherford Street Bridge will require major repairs to the abutments to restore the joint gaps.
   - The Halswell River Bridge will probably need to be replaced. The present bridge is 6.6 m long by 8.3 m wide and has an estimated replacement cost of the order of $400,000.

6. The repairs required on the eight more lightly damaged bridges are:

   - The Ohoka Road Undercrossing needs repairs to the spalling and cracking damage at the bases of pier columns.
   - The Chaneys Road Overpass requires repairs to the north abutment linkage bolts, south abutment deck joint and paved abutment slopes.
   - The Styx Overbridge No 2 needs repairs to the spalling and cracking damage on several pier columns.
   - The twin Port Hills Road Overpass bridges need repairs to the spalling and cracking at the base of the piers.
   - The twin Horotane Valley Overpass bridges need repairs to the abutments and abutment linkage bolt system. Improvements to the soil embankments may also be required.
   - The Hawkins River Bridge needs repairs to the spalling and cracking in the tops of the piles at the piers.

PERFORMANCE OF HIGHWAY STRUCTURES DURING THE DARFIELD EARTHQUAKE OF 4 SEPTEMBER 2010 AND THE CHRISTCHURCH EARTHQUAKE OF 22 FEBRUARY 2011

Report prepared for New Zealand Transport Agency (NZTA) by:

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1. INTRODUCTION

The Darfield (Canterbury) earthquake, of moment magnitude M_w 7.1, occurred at 04.35 on 4 September 2010 (NZST) and was centred approximately 40 km west of Christchurch city centre at a depth of approximately 11 km. Over the next six months it was followed by numerous aftershocks that were centred closer to the city and of which 13 were of magnitude 5 or greater, at a depth of 15 km or less. Significant damage and some collapses, mainly of older masonry structures, occurred within the city.

The Christchurch earthquake, of moment magnitude M_w 6.2, occurred at 12.51 on 22 February 2011 (NZDT) and was centred approximately 8 km south east of Christchurch city centre at a depth of approximately 6 km. Over the next 12 months this event was followed by 25 aftershocks of magnitude 5 or greater, at a depth of 11 km or less. Two reinforced concrete buildings in the Christchurch city centre collapsed in the main shock and a number of unreinforced masonry buildings partially collapsed. Serious damage occurred to many buildings in the city area. One of the aftershocks at 14.20 on 13 June 2011 (NZST) was of magnitude M_w 6.0 centred approximately 9 km south-east of Christchurch city centre at a depth of approximately 7 km. A similar magnitude aftershock occurred at 15.18 on 23 December 2011 (NZDT) and was located approximately 9 km east of the city centre. Both these events increased the damage to many of the buildings that had previously been damaged in the main shock. Although these two aftershocks were of similar magnitude to the main shock the intensity of shaking at most of the state highway bridge sites was less than in the main shock.

Following each of the two main shocks and the M_w 6.0 aftershocks, the highways were inspected by the network consultants (Opus International Consultants) to establish safety conditions and repairs that were required to enable traffic to flow. Surprisingly the bridges experienced little structural damage in the Darfield earthquake, although some approaches had settled and some stretches of highway were significantly affected by ground displacement due to the ground shaking, and lateral spreading and ground subsidence due to soil liquefaction. Rock falls also affected the highways and posed an ongoing risk to road users. Several bridges experienced more significant structural damage in the Christchurch main shock and additional settlement and lateral spreading occurred in this event. Aftershocks necessitated repeated inspections but their effects on the highways were generally minimal. Temporary or more permanent repairs to the roads were effected within the next few days or weeks following the main events but no significant immediate structural repairs were necessary to the bridges.

The performance of the highway structures during the earthquakes is of interest for reviewing the assumptions that are made during the design or seismic assessment and retrofit of

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structures and their approaches. The authors were therefore asked by New Zealand Transport Agency (NZTA) to visit the area and examine the bridges and the geotechnical effects in detail to identify how they had responded to the shaking. The visits were made by Brabhaharan during 20 to 23 September 2010 and during 24 February to 4 March 2011 and on 24 August 2011, and by Wood and Chapman on 14 and 15 October 2010 and on 16 March 2011.

The three main elements in gaining information from the project are:

- The ground motions to which each of the structures was subjected. Appendix A contains details of how the ground motions at each of the bridge sites were estimated.
- Observed structural damage or indications of structural response to the shaking, for comparison with an understanding of the expected behaviour of the structures. Appendix B contains individual sections that relate to these for each of the inspected bridges.
- Observed ground damage from ground shaking and liquefaction, both at the sites of state highway and some local authority road structures as well as in the general areas affected by the earthquakes.

2. ASSESSING THE GROUND MOTIONS AT THE BRIDGES

2.1 Intensity of Shaking

The intensity of ground shaking at the bridge sites was of interest in planning the investigation as one of the factors used to decide which bridges to inspect within the limited time available. More significantly, it was important to estimate as closely as possible the intensity of shaking each bridge had experienced so that the theoretical expected damage could be compared with the observed damage. This could indicate the validity of some of the assumptions made in the seismic analysis procedures used for design of new bridges and the assessment and retrofit of existing structures.

The area around Christchurch is well instrumented with strong motion accelerographs (SMA's) and the records obtained therefore enable estimates to be made of the shaking experienced by structures and the ground. Table 1 lists the SMA's and Figures 1 to 4 show the locations of the highways, bridges and recording stations. Figures 1 to 3 also show the peak ground accelerations (PGA's) recorded at the SMA's during the Darfield earthquake and Figure 4 the PGA's recorded during the Christchurch earthquake.

Two different methods to obtain a best estimate of the intensity of shaking at the sites of the inspected bridges and the response accelerations of the bridges were explored. A more detailed description of the procedures used is included in Appendix A.

Method 1 Using Recorded Ground Motions

The recorded ground motions at the two SMA stations nearest to each bridge were investigated. Bridges located quite close together resulted in their having the same two nearest SMA stations and were therefore treated as a group. Each of the eight groups is identified in Table 2. Grouping in this way reduced the number of areas that needed to be investigated for site shaking intensity. One exception to all bridges in the group having the same two nearest SMA's was Group 5 where two of the four bridges had HVSC as one of their nearest two SMA's but it was considered that the SMA record from CCCC, which was also quite close to all bridges in the group, was more likely to represent the bridge site ground motions. Because of the quite small separations between the bridge sites and the SMA's, it was assumed that a mean of the ground motions recorded by the nearest two SMA's would provide a best estimate of the shaking intensity experienced by the bridges in each group. The results in terms of PGA's and the 0.2, 0.4, 0.6 and 0.8 second spectral ordinates for the eight bridge groups are plotted in Figures 5 and 7 for the Darfield and Christchurch earthquakes respectively. In addition, 1.0 second spectral ordinates are plotted in Figure 7 for the Christchurch earthquake as the stronger shaking at some sites was expected to result in longer periods of vibration.

Method 2 Using Attenuation Functions

The second method explored for assessing the intensity of shaking at the sites of the eight groups of bridges was based on the McVerry *et al* (2006) attenuation functions for peak ground acceleration (PGA) and spectral ordinates that underlie the hazard model used to develop the NZS 1170.5 design spectra. This method was only applied to the Darfield earthquake observations, as the results were not adopted for subsequent assessments of the bridges' performance.

The geometric means of the two horizontal components of the PGA and of the spectral ordinates for structure periods of 0.2, 0.4, 0.6 and 0.8 seconds were computed using the attenuation relationships for each of the eight bridge groups assuming a strike-slip mechanism for the Darfield earthquake. The shortest distance between the fault rupture and the bridges in each group was taken as the average of the distance for each individual bridge in the group, as the variation of this distance within each group was small. The results are plotted in Figure 6 for the eight bridge groups.

Comparison of Results from Methods 1 and 2

Comparing the plotted results for the Darfield earthquake, there is less variation of the spectral ordinates over the range 0.2 to 0.8 seconds from Method 2 (Figure 6) than from Method 1 (Figure 5). The attenuation function approach, although not adopted further for this investigation, does smooth out the variation in the spectral response, as the attenuation functions are based on regression analysis of a large number of similar magnitude events. It reduces the influence of fault directivity and local site effects, which were evident in the SMA records used in the first method for Groups 6 and 8. Results from both methods can be used to obtain a range of likely acceleration response values for the bridges in each group. Because of the averaging procedures inherent in both methods they give a best estimate and do not predict the minimum or maximum possible response values. Lower and upper bounds to the response accelerations for each bridge group can be estimated by the error bars plotted in Figures 5 and 6. The figure captions indicate the method used to determine the error magnitudes.

2.2 Return Periods for Shaking Intensity

Estimation of Return Periods

The seismic design standard NZS1170.5 specifies seismic hazard response spectra for the whole country. These are derived for a particular location from the product of the Spectral Shape Factor ($C_h(T)$), the Hazard Factor (Z) for the location, and the Return Period Factor (R). The return period for the ground shaking intensity in both the Darfield and Christchurch earthquakes associated with each bridge group was estimated by averaging the spectral ordinates of the processed acceleration/period spectra available from GeoNet (http://www.geonet.org.nz/) over spectral ordinates for the period ranges 0.1 to 0.5 seconds and 0.5 to 0.9 seconds and comparing these with the NZS 1170.5 Spectral Shape Factor for the site subsoil class at the recording station, adjusted using the NZS 1170.5 Return Period

Factor. For the comparison the NZS 1170.5 Hazard Factor appropriate for each of the bridge group locations was applied. While it has recently been proposed to increase the Hazard Factor for Christchurch the present analyses were based on the current Code values. The spectral ordinates were averaged over the four components of ground motion associated with each bridge group. The 0.1 to 0.5 second range was chosen for the comparison as this corresponds to the plateau of the NZS 1170.5 shape factor for Site Subsoil Classes D and E. The range 0.5 to 0.9 seconds was selected as most of the bridges inspected had estimated periods in the 0.1 to 0.9 second range. More details of the process used are contained in Appendix A.

Results of the Estimation

The return periods calculated for the shaking intensity corresponding to each bridge group by the above method are listed in Table 4. Very strong shaking was recorded at the HVSC station, the closest recorder to the Group 6 Bridges, in the Darfield earthquake and both the closest recorders to this group in the Christchurch earthquake. The very strong spectral ordinates from the HVSC records in the 0.1 to 0.5 second range of the spectra from the Darfield earthquake indicated a return period of about 4,000 years, and the 0.1 to 0.9 second range of the spectra from the Christchurch earthquake indicated a return period of greater than 10,000 years. This very strong short period intensity was also apparent in the spectra from the Christchurch earthquake at the LPCC station, the second closest to the Group 6 bridges, but was not found in the records from the other recording stations. Very strong shaking would be expected at the HVSC and LPCC stations in the Christchurch earthquake because they were both only a few kilometres from the epicentre. However, the very strong shaking at HVSC during the Darfield earthquake was much higher than indicated by the McVerry *et al* attenuation functions. Because of this anomaly, separate return periods are shown in Table 4 for the records from the two recorders closest to the Group 6 bridges. The return periods of shaking intensity varied considerably between the groups, as is also indicated by the spectral ordinates plotted in Figures 5, 6 and 7. The return periods are also sensitive to the period range of interest.

2.3 Comparison of Return Periods and Intensity of Shaking with Design Levels

When comparing the intensity of the shaking experienced by a structure with its design strength it is important to also consider the basis of its seismic design.

Post-1971: Since 1971 seismic design has provided for stronger earthquakes than previously and has taken account of the structure’s importance, foundation soil type, natural period, and the seismic risk associated with the structure’s location. As an example, a bridge in Christchurch on an important route (which includes bridges in Groups 1 to 6), on soft soils and of a short natural period (up to 0.5 seconds) would be designed by the present Bridge Manual (2003) for a 2,500-year return period event, represented by a coefficient of 0.83 g, reducing gradually to 0.55 g if its natural period were between 0.5 and 1.0 second. Structures on less important routes (which include bridges in Groups 7 and 8) would be designed for a 1000-year event, with coefficients about 70% of those quoted above. A key requirement for post-1971 bridges is that ductility must also be built into them. Provided sufficient ductility is included, a structure’s design strength may be reduced by up to a factor of 6, although shorter period structures and serviceability considerations can limit this to a lesser value. Assuming a ductile structure, the yield strength of a bridge designed to the 1000-year return period level is often likely to be similar to that of a pre-1971 design, but the structure will possess significantly more resilience.

Pre-1971: Before 1971 a design horizontal seismic coefficient of 0.1 g at working stresses was specified, irrespective of the structure's natural period or location. This equates to a probable flexural ultimate strength value of 0.18 g when safety factors and probable steel yield strength are taken into account. For such structures no requirement for ductile capability was specified, and therefore such structures may or may not be at risk of brittle failure, depending on their proportions and details. On the basis of return period factors in NZS1170.5:2004 the pre-1970 design loading represents an elastic limit resistance equivalent to resisting an earthquake of approximately a 60 to 150-year return period event for the Christchurch area.

Darfield Earthquake Records: Excluding the Group 6 HVSC record, the overall averages of the return period levels in the Darfield earthquake, as measured by the equivalent return period of the spectral ordinates averaged over the 0.1 to 0.9 second period range, were between 140 and 580 years. These values indicate, *in theory,* that the pre-1970 bridges probably experienced shaking equivalent to their elastic design intensity, whereas the post-1970 ductile bridges, may or may not have reached their design elastic limits, depending on the actual elastic strengths for which they were designed.

Christchurch Earthquake Records: The shaking intensity at the Groups 3 to 6 sites, as measured by the equivalent return period of the spectral ordinates averaged over the 0.1 to 0.9 second period range, was greater in the Christchurch earthquake than in the Darfield earthquake. For Groups 3 and 4 the average return period levels were 280 and 390 years respectively. For Groups 5 and 6 the average return period levels were 1400 and 5000 years (excluding the severe HVSC records) respectively. These values indicate, *in theory,* that the pre-1971 bridges experienced shaking significantly exceeding their elastic design intensity, and the post-1971 ductile bridges most likely reached, and probably exceeded, their design elastic limits.

2.4 Information Sources

- Best estimate values and possible ranges of ground shaking at the bridges, and their equivalent return periods relative to the hazard set out in NZS1170.5, can be obtained for each bridge group by reference to Figures 5, 6 and 7 and Table 4.
- Comparison of ground motion displacement spectra with NZS1170.5 spectra with return periods for 250 and 1000 years can be found in Figures A9 and A10 in Appendix A.
- Peak ground accelerations can be estimated by reference to Table 2 and the time/history plots included in Figures A19 to A44 in Appendix A, which also contains more detailed information on the assessment of ground motions at the bridge sites.
- For assessing the demand on the bridges listed in Tables 5 and 6 the values of PGA and spectral accelerations shown in Figures 5 and 7 were used. The information in Figures 5 and 7 is thought to give the best estimate of the structure response accelerations since Method 1 used to derive these figures is based on actual recorded motions. It therefore includes the influence of the source characteristics of the particular earthquake and local ground conditions better than does Method 2.
- Method 2 and Figure 6 provide useful information on the uncertainty in the best estimates of the structure response accelerations.

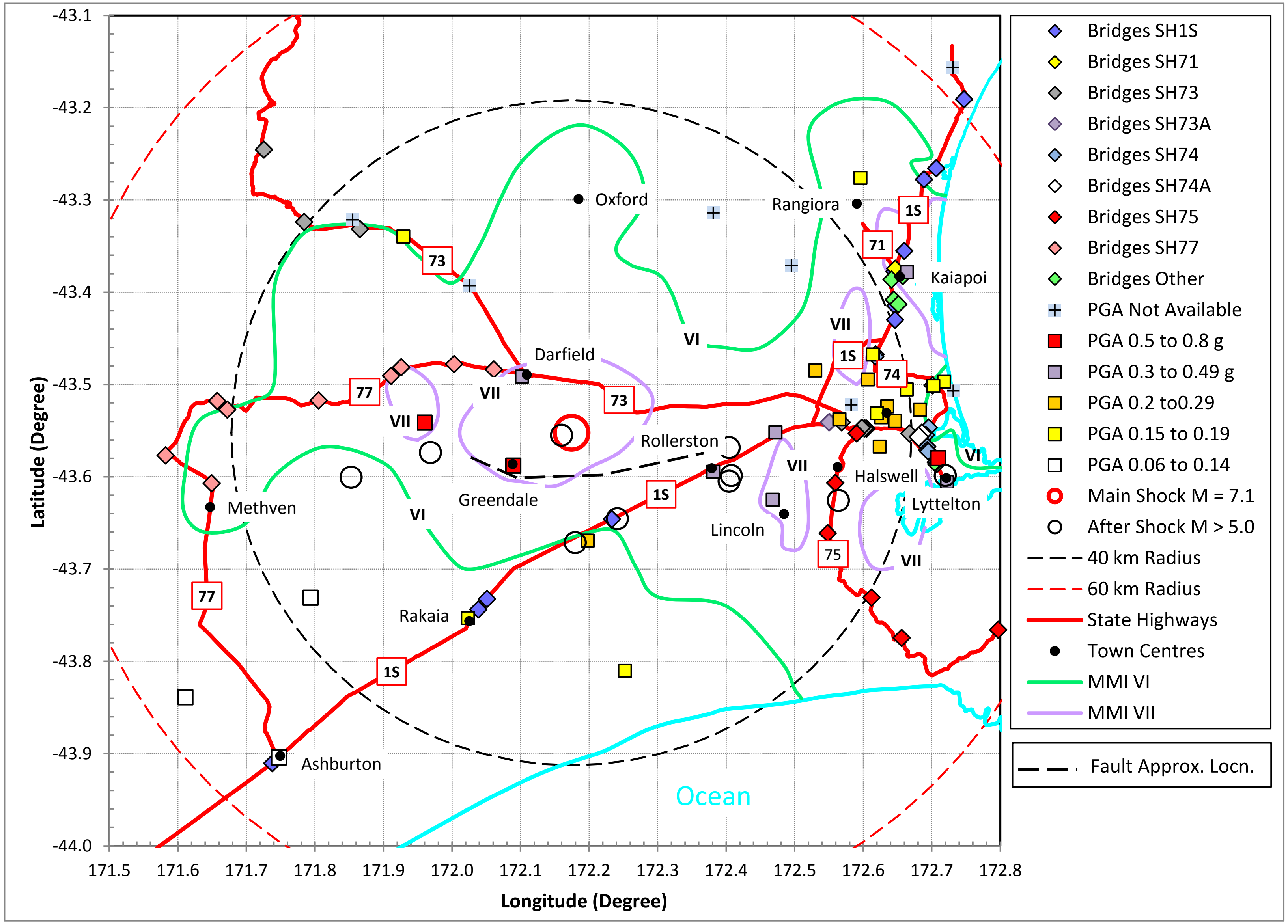

Figure 1. Darfield Earthquake. Location of state highway bridges in relation to SMA stations and MMI iso-seismals. All state highway bridges within 50 km of the epicentre are shown.

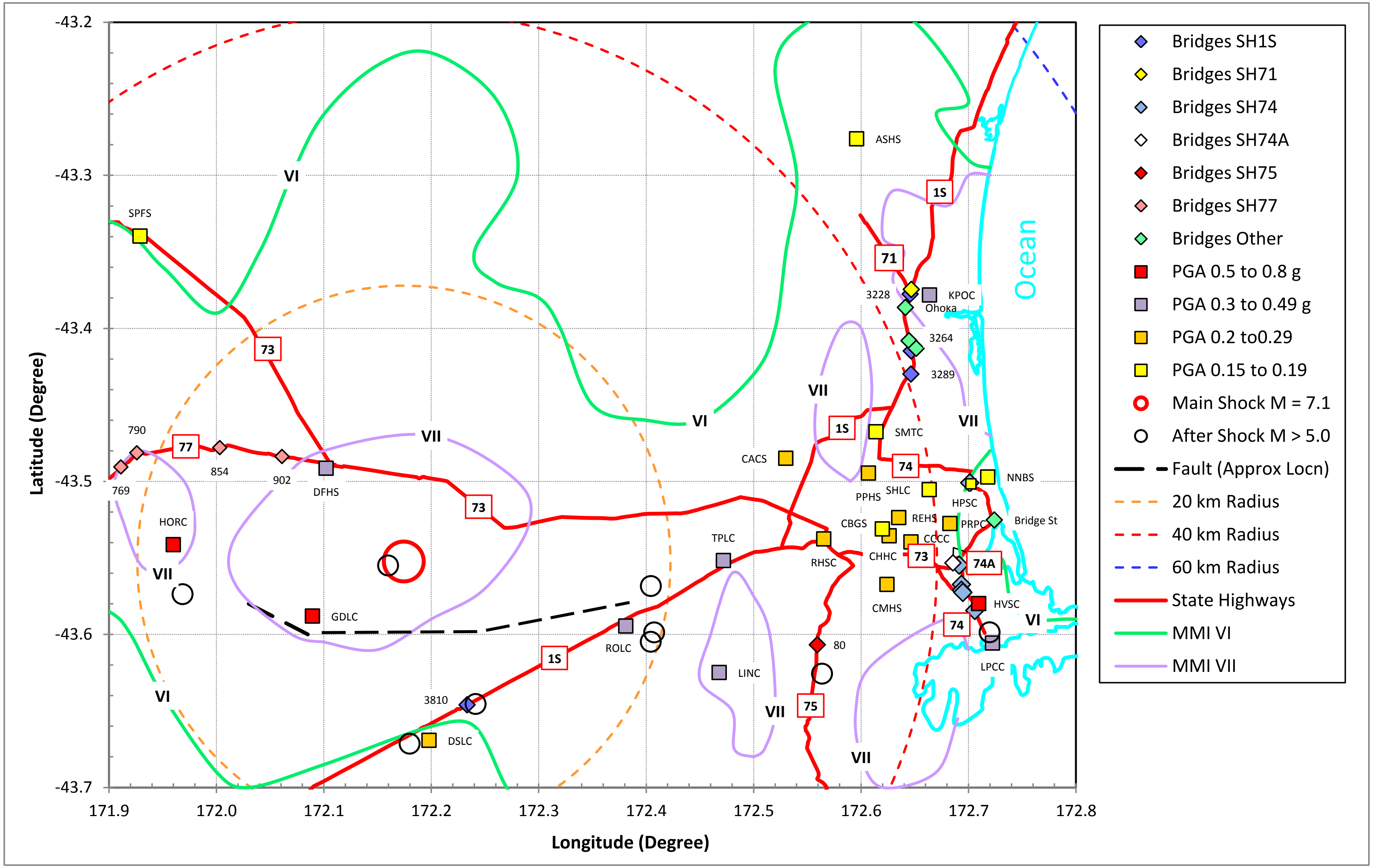

Figure 2. Darfield Earthquake. Location of inspected state highway bridges in relation to SMA stations and MMI iso-seismals. Labels indicate SMA abbreviated name and bridge BSN (where shown).

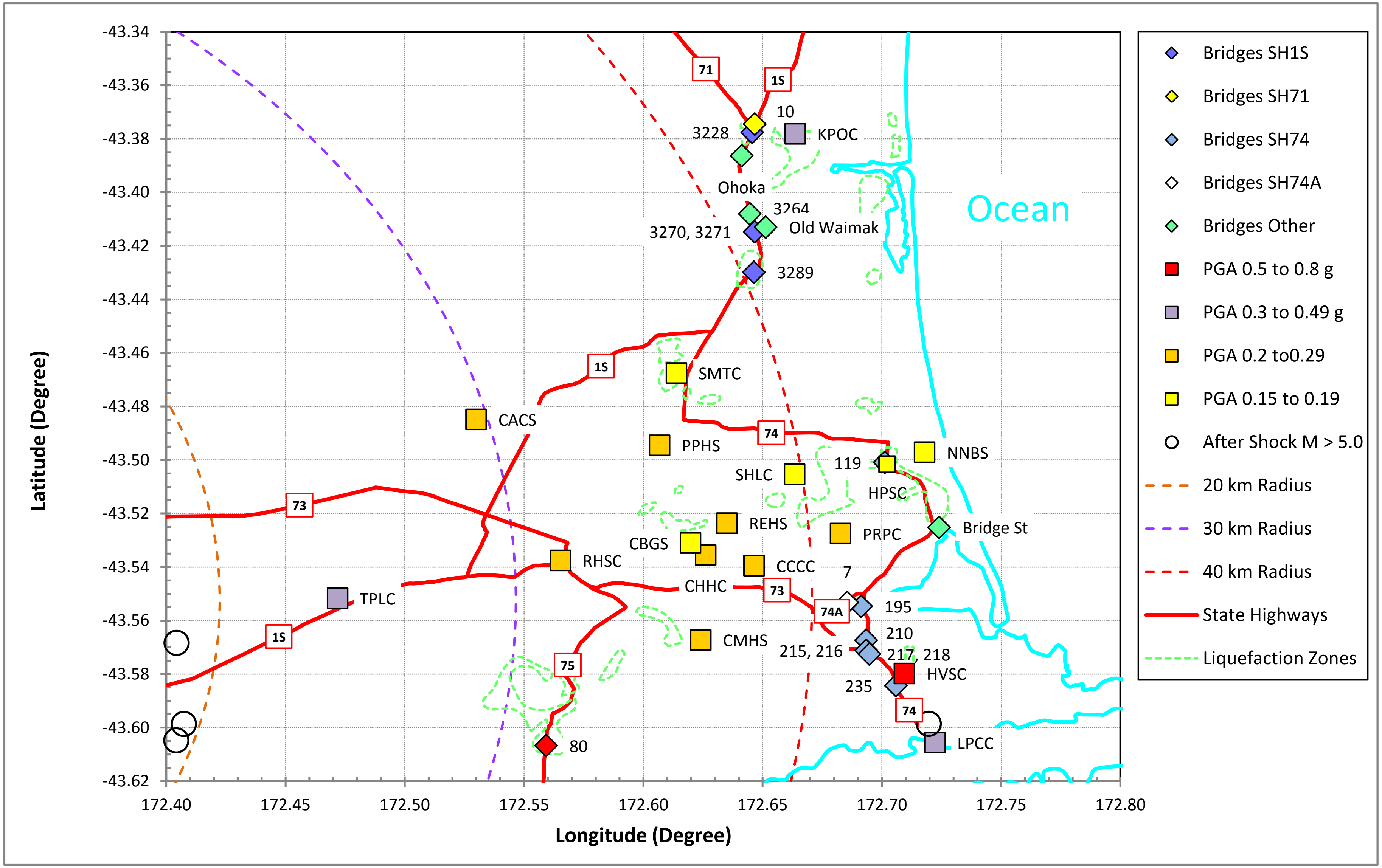

Figure 3. Darfield Earthquake. Location of inspected state highway bridges in relation to SMA stations and liquefaction areas. SH77 and SH1 Selwyn River Bridge are not shown (see Figure 2).

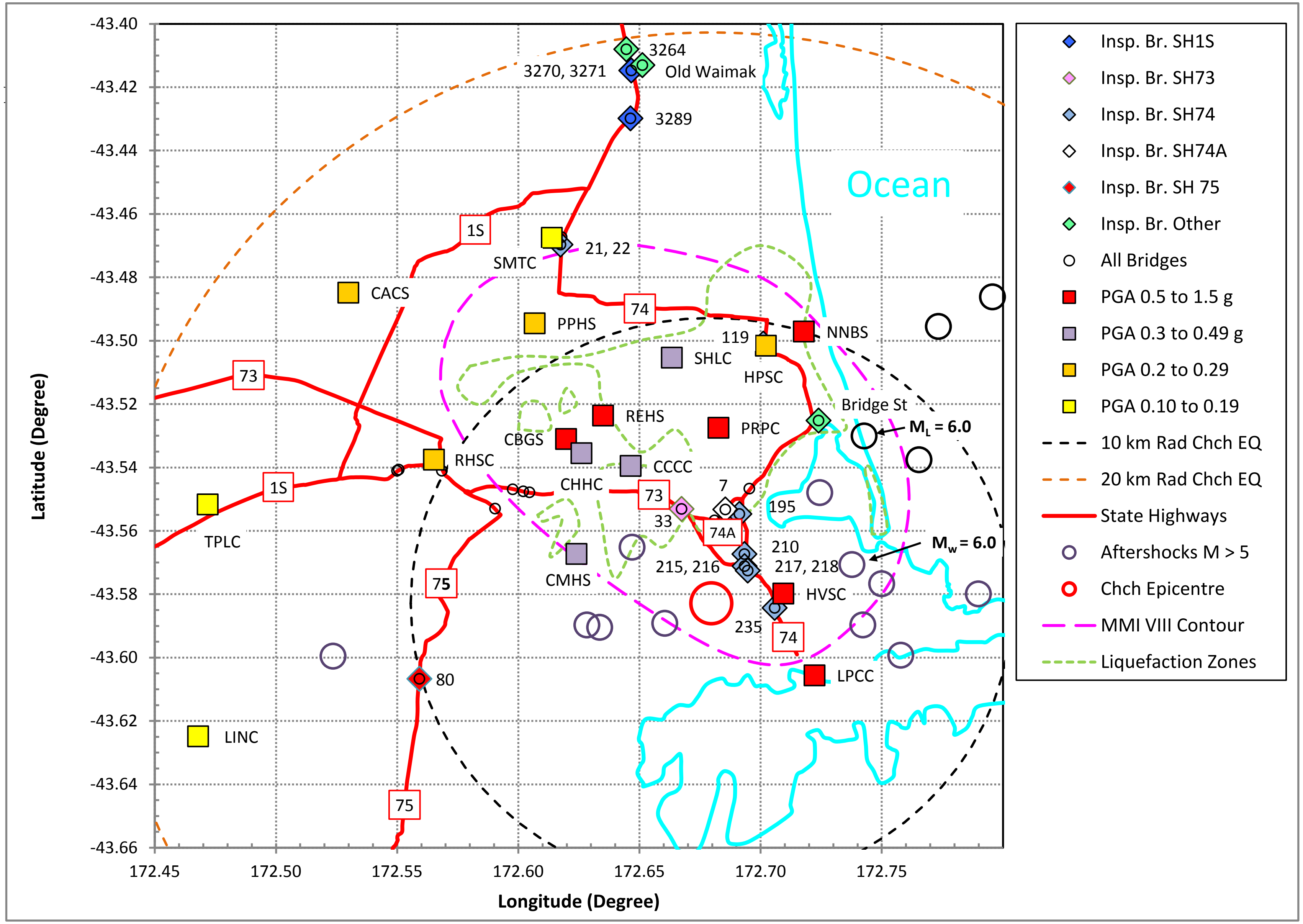

Figure 4. Christchurch Earthquake 22 Feb 2011 and aftershock epicentres in relation to inspected state highway bridges and SMA stations.

Table 1. Strong Motion Recorders within 80 km of Darfield Earthquake Epicentre

	Name of SMA Recording Station	SMA[1] Subsoil Category	Epicentral and Fault Distances, km			PGA's - Horizontal Components, g			
						Darfield		Christchurch	
			Darfield Epicent.	Darfield Fault	Chch. Epicent.	H 1	H2	H 1	H2
GDLC	Greendale	D	8	1.3	-	0.72	0.68	-	-
DFHS	Darfield High School	D	9	6	49	0.49	0.46	0.05	0.06
DSLC	Dunsandel School	D	13	8	42	0.24	0.25	0.05	0.07
ROLC	Rolleston School	D	17	2	26	0.30	0.35	0.18	0.19
HORC	Hororata School	D	18	7	60	0.47	0.44	0.06	0.04
TPLC	Templeton School	D	24	8	19	0.28	0.19	0.12	0.10
LINC	Lincoln Crop and Food Research	D	25	9	19	0.43	0.40	0.16	0.08
RKAC	Rakaia School	D	26	18	58	0.19	0.17	0.04	0.02
CACS	Canterbury Aero Club	D	29	16	18	0.18	0.19	0.19	0.22
SBRC	Southbridge School	D	29	24	44	0.15	0.15	0.04	0.07
RHSC	Riccarton High School	D	31	16	12	0.19	0.23	0.29	0.25
SPFS	Springfield Fire Station	D	31	28	68	0.16	0.16	0.05	0.08
PPHS	Christchurch Papanui High School	D	35	21	12	0.21	0.18	0.21	0.20
CBGS	Christchurch Botanic Gardens	D	36	20	9	0.15	0.18	0.53	0.43
CMHS	Chch. Cashmere High School	D	36	20	6	0.25	0.23	0.35	0.40
CHHC	Christchurch Hospital	D	36	21	8	0.21	0.15	0.33	0.36
SMTC	Styx Mill Transfer Station	D - E	36	23	14	0.17	0.17	0.18	0.14
REHS	Christchurch Resthaven	D	37	22	8	0.24	0.26	0.72	0.37
LSRC	Lauriston	D	37	26	-	0.11	0.07	-	-
CCCC	Christchurch Cathedral College	D	38	22	6	0.23	0.19	0.48	0.37
DORC	Dorie	D	39	33	60	0.09	0.08	0.03	0.05
SHLC	Shirley Library	D – E	39	25	9	0.17	0.19	0.31	0.35
PRPC	Pages Road Pumping Station	E	41	25	6	0.19	0.22	0.67	0.59
HVSC	Heathcote Valley Primary School	C	43	27	1	0.56	0.62	1.46	1.19
HPSC	Hulverstone Drive Pumping Station	E	43	28	9	0.17	0.11	0.15	0.25
NNBS	Chch. North New Brighton School	E	44	29	11	0.20	0.20	0.77	0.61
KPOC	Kaiapoi North School	E	44	32	23	0.31	0.34	0.21	0.19
LPCC	Lyttelton Port Company	B	44	28	4	0.34	0.22	0.78	0.88
ASHS	Ashley School	D	45	38	35	0.16	0.13	0.09	0.04
CSHS	Castle Hill Station	B	51	46	88	0.09	0.08	0.05	0.03
ADCS	Ashburton District Council	D	53	43	85	0.08	0.11	0.03	0.07
WSFC	Westerfield	D	56	45	92	0.06	0.07	0.02	0.02
MAYC	Mayfield School	D	68	56	-	0.07	0.06	-	-
WAKC	Waikari	C	78	73	68	0.15	0.13	0.08	0.07

Notes: 1. Subsoil Category at recording station as defined in NZS 1170.5:2004
2. Nine recording stations from which Darfield earthquake main shock records were not available are not included.

Table 2. Location of Nearest Recorders to Bridges Inspected

SH	BSN	Bridge Name	Bridge Group	Nearest SMA Recorder			2nd Nearest SMA Recorder			Bridge to Epicentre Distance[2], km	Bridge to Fault Rupture, Distance, km
				SMA Code	Dist km	Mean PGA[1] g	SMA Code	Dist km	Mean PGA[1] g		
1S	3228	Kaiapoi Railway River[3]	1	KPOC	1.5	0.33 0.20 0.10	SMTC	10.3	0.17 0.16 0.09	43 22 21	31
1S	3270 3271	Waimakariri River South & North Bound		KPOC	4.3	0.33 0.20 0.10	SMTC	6.4	0.17 0.16 0.09	41 19 18	28
1S	3289	Chaneys Rd Overpass		SMTC	4.9	0.17 0.16 0.09	KPOC	5.9	0.33 0.20 0.10	40 17 17	27
71	10	Cam Road Underpass		KPOC	1.4	0.33 0.20 0.10	SMTC	10.7	0.17 0.16 0.09	43 23 23	31
MIS	3264	Tram Road Underpass		KPOC	3.7	0.33 0.20 0.10	SMTC	7.1	0.17 0.16 0.09	41 20 19	29
MIS	3239	Ohoka Road U/P		KPOC	2.0	0.33 0.20 0.10	SMTC	9.3	0.17 0.16 0.09	42 22 21	30
MIS	OB7B	Old Waimakariri River		KPOC	4.0	0.33 0.20 0.10	SMTC	6.8	0.17 0.16 0.09	42 19 18	29
1S	3810	Selwyn River	2	DSLC	3.8	0.25 0.06 0.03	ROLC	13.2	0.33 0.18 0.05	11 38 42	6
74	21 22	Styx Overbridges No 1 & 2	3	SMTC	0.4	0.17 0.16 0.09	PPHS	2.9	0.20 0.20 0.13	37 14 15	23
74	119	Anzac Drive	4	HPSC	0.1	0.14 0.20 0.29	NNBS	1.4	0.20 0.69 0.18	43 9 8	28
CCC	-	Bridge Street		HPSC	3.2	0.14 0.20 0.29	NNBS	3.2	0.20 0.69 0.18	44 7 5	29
73	33	Heathcote River (Opawa)	5	CCCC	2.3	0.21 0.43 -	PRPC	3.1	0.22 0.63 0.40	42 2 4	24
74	195	Heathcote River		PRPC	3.1	0.22 0.63 0.40	HVSC[4]	3.1	0.61 1.32 0.82	42 3 4	26
74	210	Railway Overbridge		HVSC[4]	1.9	0.61 1.32 0.82	PRPC	4.5	0.22 0.63 0.40	42 2 4	26
74A	7	Rutherford Street		PRPC	2.9	0.22 0.63 0.40	CCCC	3.5	0.21 0.43 -	41 3 5	25
74	215 216	Port Hills Overpasses No 1 & 2	6	HVSC	1.6	0.61 1.32 0.82	LPCC	4.5	0.28 0.83 0.57	42 1 4	26
74	217 218	Horotane Valley Overpasses No 1 & 2		HVSC	1.4	0.61 1.32 0.82	LPCC	4.3	0.28 0.83 0.57	42 1 4	26
74	235	Heathcote Valley O/P		HVSC	0.6	0.61 1.32 0.82	LPCC	2.7	0.28 0.83 0.57	43 0.5 4	27
75	80	Halswell River (Landsdown)	7	CMHS	6.8	0.24 0.38 0.19	LINC	7.6	0.41 0.12 0.07	32 12 16	15

Table 2. Continued

SH	BSN	Bridge Name	Bridge Group	Nearest SMA Recorder			2nd Nearest SMA Recorder			Bridge to Epicentre Distance[2], km	Bridge to Fault Rupture, Distance, km
				SMA ID	Dist km	Mean PGA[1] g	SMA ID	Dist km	Mean PGA[1] g		
77	769	Wairiri Stream	8	HORC	6.9	0.45 0.05 0.03	DFHS	15.4	0.47 0.06 0.03	22 65 68	14
77	790	Selwyn River		HORC	7.2	0.45 0.05 0.03	DFHS	14.3	0.47 0.06 0.03	22 64 67	14
77	854	Waianiwaniwa River		HORC	7.9	0.45 0.05 0.03	DFHS	8.1	0.47 0.06 0.03	16 58 60	12
77	902	Hawkins River		DFHS	3.4	0.47 0.06 0.03	HORC	10.4	0.45 0.05 0.03	12 53 56	11

Notes:
1. Mean PGA is the mean PGA of the two horizontal components.
   PGA's listed in order for 4-Sep-10, 22-Feb-11 and 13-Jun-11 events.
2. Epicentral distance listed in order for 4-Sep-10, 22-Feb-11 and 13-Jun-11 events.
3. Kaiapoi Railway River Bridge comprises twin superstructures on a common substructure and is designated as one bridge.
4. Strong Motion Recorder Station CCCC adopted as better indicator of ground motions than HVSC for these bridges.

Table 3. Bridge Site and SMA Station Subsoil Categories

SH	BSN	Bridge Name	Bridge Group	Bridge Site Subsoil Category (NZS 1170)	Source[1] for Bridge Subsoil Category	Nearest SMA Recorder		2nd Nearest SMA Recorder	
						SMA ID	Station Subsoil Category	SMA ID	Station Subsoil Category
1S	3228	Kaiapoi Railway River	1	D	Drawings	KPOC	E	SMTC	D - E
1S	3270 3271	Waimakariri River South & North Bound	1	D	Drawings	KPOC	E	SMTC	D - E
1S	3289	Chaneys Rd Overpass	1	D	Drawings	SMTC	E	KPOC	E
71	10	Cam Road Underpass	1	D	DSA	KPOC	E	SMTC	D - E
MIS	3264	Tram Road Underpass	1	D	Estimated	KPOC	E	SMTC	D - E
MIS	3239	Ohoka Road U/P	1	D	Estimated	KPOC	E	SMTC	D - E
MIS	OB7B	Old Waimakariri River	1	D	Estimated	KPOC	E	SMTC	D - E
1S	3810	Selwyn River	2	D	Estimated	DSLC	D	ROLC	D
74	21 22	Styx Overbridges No 1 & 2	3	D	Estimated	SMTC	D – E	PPHS	D
74	119	Anzac Drive	4	D	Drawings	HPSC	E	NNBS	E
CCC	-	Bridge Street	4	D	Drawings	HPSC	E	NNBS	E
73	33	Heathcote River (Opawa)	5	D	Drawings	CCCC	D	PRPC	E
74	195	Heathcote River	5	D	Drawings	PRPC	E	CCCC[2]	D
74	210	Railway Overbridge	5	D	Drawings	CCCC[2]	D	PRPC	E
74A	7	Rutherford Street	5	D	Drawings	PRPC	E	CCCC	D
74	215 216	Port Hills Overpass No 1 & No 2	6	D	DSA	HVSC	C	LPCC	B
74	217 218	Horotane Valley O/P No 1 & No 2	6	D	DSA	HVSC	C	LPCC	B
74	235	Heathcote Valley O/P	6	B	DSA	HVSC	C	LPCC	B
75	80	Halswell River (Land.)	7	D	Drawings	CMHS	D	LINC	D
77	769	Wairiri Stream	8	D	Estimated	HORC	D	DFHS	D
77	790	Selwyn River	8	D	Estimated	HORC	D	DFHS	D
77	854	Waianiwaniwa River	8	D	Estimated	HORC	D	DFHS	D
77	902	Hawkins River	8	D	Estimated	DFHS	D	HORC	D

Notes:
1. Abbreviations: Drawings = Drawing Files; DSA = Detailed Seismic Assessment
2. Strong Motion Recorder Station CCCC adopted as better indicator of ground motions than HVSC for these bridges, although HVSC nearer (see Table 2).

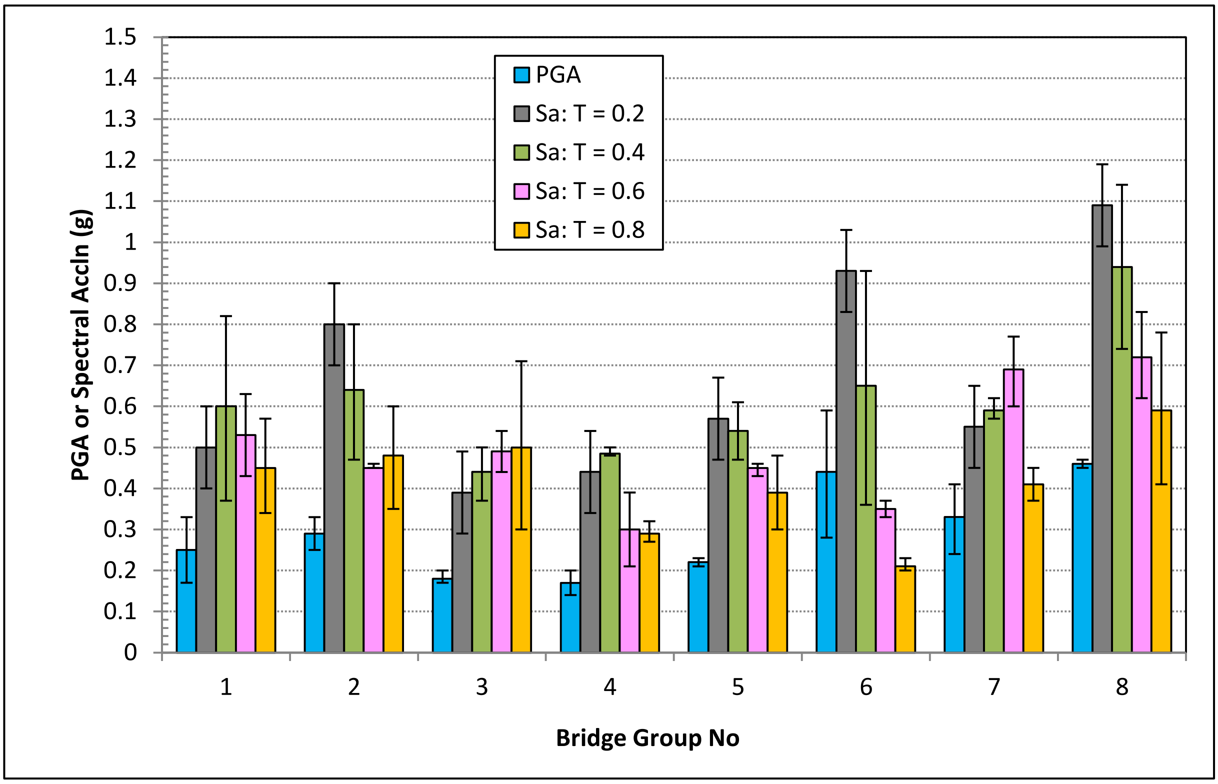

Figure 5. Darfield Earthquake. Mean PGA and S_a's from the nearest two SMA's to Bridge Groups. The error bars show the variation between the two stations used for each group with the upper and lower limits being the mean of the two horizontal components at each station.

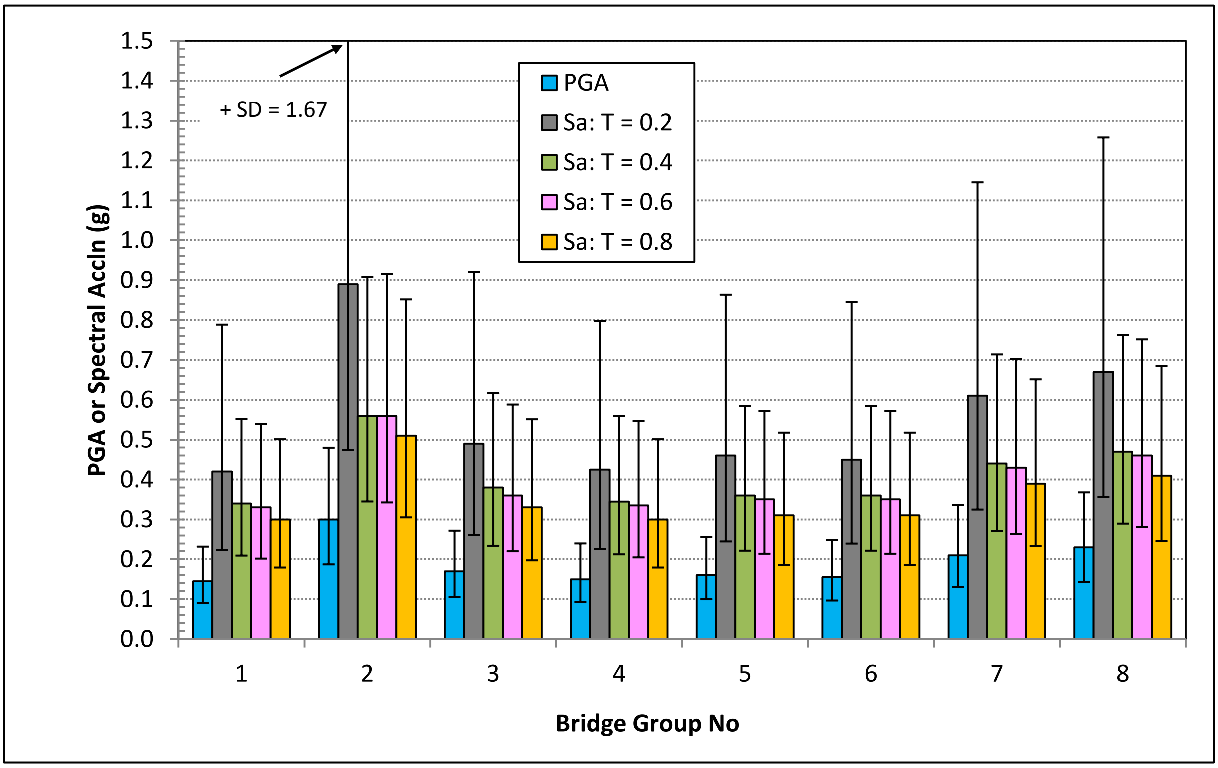

Figure 6. Darfield Earthquake. Mean PGA and S_a's from McVerry *et al* attenuation functions. Error bars show ± one standard deviation of the median predicted values.

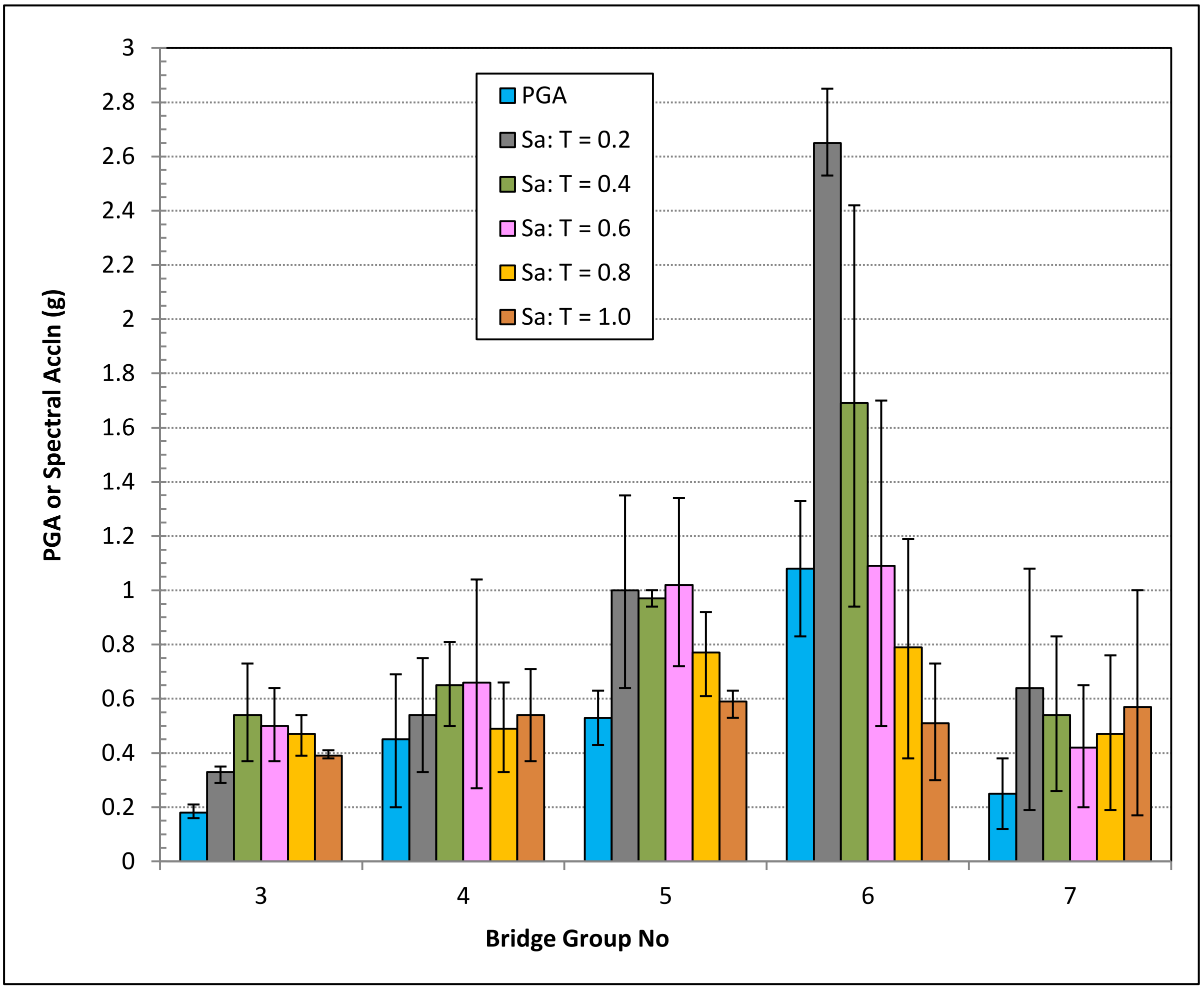

Figure 7. Christchurch Earthquake. Mean PGA and S_a's from the nearest two SMA's to Bridge Groups. The error bars show the variation between the two stations used for each group with the upper and lower limits being the mean of the two horizontal components at each station.

Table 4. Ground Motion Return Periods from Strong Motion Accelerograph (SMA) Records. The return periods shown in [ ] are for the 22-Feb-11 Christchurch earthquake. The unbracketed values are for the 4-Sep-10 Darfield earthquake.

Bridge Group	Bridge Group Zone factor (Z)	SMA Station Subsoil Class (NZS1170.5)	Return Period , years		
			S_a 0.1 to 0.5 seconds	S_a 0.5 to 0.9 seconds	Average S_a 0.1 to 0.5 & 0.5 to 0.9 seconds
1	0.26	E (KPOC & SMTC)	220	180	200
2	0.22	D (DSLC & ROLC)	520	320	420
3	0.22	D (SMTC & PPHS)	170 [180]	350 [390]	260 [280]
4	0.22	E (HPSC & NNBS)	190 [420]	80 [350]	140 [390]
5	0.22	E (PRPC); D (CCCC)[1]	290 [1,300]	220 [1,500]	250 [1,400]
6	0.22	C (HVSC)	4,000 [> 10,000]	540 [> 10,000]	2,300 [> 10,000]
		B (LPCC)	650 [> 5,000]	510 [1,600]	580 [5,000]
7	0.22	D (CMHS & LINC)	450 [250]	600 [300]	530 [270]
8	0.30	D (HORC & DFHS)	710	340	520

Notes: 1. Strong Motion Recorder Station CCCC was adopted as better indicator of ground motions although HVSC was nearer for these bridges (see Table 2).

2. Return periods for the effects of the Christchurch earthquake on bridges in Groups 1, 2 and 8 were not calculated, as they were not critical.

3. THE BRIDGES – INSPECTION AND ASSESSMENT

3.1 General

There are about 50 state highway bridges located within 50 km of the Darfield earthquake epicentre and 30 located within 20 km of the Christchurch earthquake epicentre. Of the total of 50 bridges, 27 individual bridges were inspected as part of the study of the performance of the state highway bridges in the earthquakes. Included in the total inspected were four sets of twin bridges with each pair in three of the sets having similar but not identical details. The fourth set of twin bridges, Styx Overbridges Numbers 1 and 2, were constructed at different times and differed significantly in structural form and detail. The total also included two Christchurch City Council bridges that were damaged in the Darfield earthquake. One of these was the Old Waimakariri River Bridge, which was of interest because of its age and proximity to the Waimakariri River state highway bridge, and the other was the Bridge Street Bridge, which was of interest because it sustained serious damage from liquefaction lateral spreading.

Twenty two of the inspected bridges were designed to requirements that were in force before 1971 (see Table 5), when more stringent design standards were introduced. In particular, while the designated strength was less than current requirements, no special measures were taken to providing the structures with ductile resilience, as is current practice. Any resilience possessed by these older structures is therefore coincidental, rather than intentional.

A site "walk-over" was carried out at each of the inspected bridges with the time spent on site generally varying between 30 minutes to one hour. Particular attention was focused on checking for evidence of movements at the piers and abutments. On many bridges the most critically loaded components, such as the abutment footings and piles, and pier bases and piles, were covered by water or soil so it was not possible to clearly establish whether there had been damage to these items. However, the extent of gapping between the piers and abutments at ground level gave some indication of the likelihood of foundation damage. Design drawings or as-builts were available for all of the bridges and were used to identify the critical components in the foundations and for the follow-up assessment work. Appendix B contains details of each bridge that was inspected, together with the observations made of any earthquake effects noted.

3.2 The Figures

Figure 1 shows the location of the SMA's within 50 km of the Darfield earthquake epicentre and their relationships to the state highway bridges within this radius. The SMA locations have been colour coded to indicate the intensity of ground shaking as measured by the recorded PGA's in the Darfield earthquake. Also shown are approximate Modified Mercalli Intensity (MMI) iso-seismals for intensities VI and VII. These have been derived from a colour graded MMI plot available from the GeoNet website and reproduced in Appendix A.

Figures 2 and 3, which are drawn to a larger scale than Figure 1, show the locations of the bridges that were inspected and the locations of the SMA's. To simplify the figures the bridges not inspected have been omitted. The bridges are labelled in Figure 3 with their Bridge Structure Number (BSN) as recorded in the NZTA Descriptive Inventory, and the SMA's with their station abbreviation. Bridges outside the area covered by Figure 3 are labelled on Figure 2. The names of the inspected bridges, their state highway number, and their BSN are listed in Table 2. All the SMA stations within the areas covered by Figures 2 and 3 are labelled with the station abbreviation listed in Table 1.

Figure 4 shows the location of the SMA's within 20 km of the Christchurch earthquake epicentre and all the state highway bridges within this radius except for two small bridges to the south of the epicentre on SH75 (BSN's 140 and 261). BSN's and SMA identifiers are shown as described above for Figure 3. The SMA locations have been colour coded to indicate the intensity of ground shaking as measured by the recorded PGA's in the Christchurch earthquake. Also shown is the approximate MMI iso-seismal for intensity VIII. This was estimated from the MMI's assigned to observed damage and shown on a ground motion intensity plot available on the GeoNet website.

3.3 Approximate Analyses of Bridges

To assess the strength and performance of the critical components of each of the bridges simple static analyses were carried out. These were generally based on spreadsheet computations but in some cases the piles were analysed with a simple two-dimensional computer model employing Winkler springs. For the transverse direction, the analyses were based on a tributary mass assumption for the tallest or most critically loaded pier. For the longitudinal direction, the relative stiffness of the piers and abutments was considered, with passive pressures on the abutments assumed to resist the appropriate load level based on the overall estimated displacement. Generally all the piers of similar height were assumed to be of the same stiffness although in some cases the height between the tops of the piles at the underside of the cap and the bed level varied along the length of the bridge. A more detailed description of the basis of the approximate analyses is contained in Appendix C. The results of these analyses could differ from those obtained by dynamic or non-linear push-over analyses by about ±30%.

To provide for a comparison to be made between the expected and actual performance of the bridges, three values, expressed in terms of gravity acceleration g, were calculated for each structure for both longitudinal and transverse directions:

- The limiting capacity of the bridge's various elements in flexure and shear.
- The minimum capacity that the bridge was assessed to require in terms of the design code current at the date of design.
- The response acceleration predicted for the bridge, taking account of its estimated period of vibration and the spectral accelerations computed from the two closest SMA records shown in Figures 5 and 7 for the Darfield and Christchurch earthquakes respectively.

From these three values, the following ratios were calculated and listed in Tables 5 and 6 for the Darfield and Christchurch earthquakes respectively:

- The minimum ratio of limiting calculated capacity to the required design capacity. This is expressed as "Minimum Ratio Capacity/Design Coefficient".
- The minimum ratio of limiting capacity to response acceleration. This is expressed as "Minimum Ratio Capacity/Demand".

Tables 5 and 6 also contain a summary of data used for these calculations and the assessed return period of the ground motion each bridge was estimated to have experienced.

3.4 Discussion and Comments on Results

Following is a summary of the more detailed coverage contained in Appendices B and C:

Design Coefficients: With the exception of the bridges designed before 1931, Tables 5 and 6 show the equivalent ultimate strength seismic design coefficients for the inspected bridges. The coefficients vary from 0.18 for the older bridges to 0.24 for the Anzac Drive Bridge designed in 1999. This range is not very great, indicating that the damage threshold of flexural yield would not be expected to vary greatly between the inspected bridges. If the damage threshold were exceeded, bridges designed before 1971 would be expected to be more seriously damaged than more recent designs because the older bridges generally were not specifically detailed to provide ductility in their lateral load resisting elements after reaching yield level displacements.

Capacity to Design Coefficient Ratios: The ratios of capacity over design coefficient listed in Tables 5 and 6 indicate that most of the inspected bridges had strengths that exceeded by more than 50% the minimum design provisions of the codes that were likely to have been used for their design. Adoption of probable, rather than minimum specified steel yield strengths for the assessments would have contributed to this margin. In several cases, eccentric live loading may have been more critical than seismic loading and in some two cases the bridges were designed for additional lanes to be added by widening the superstructure. Rather surprisingly the ratio was only about 1.0 for the Waimakariri River Bridge, which, however, may have been designed using the 1956 Bridge Manual provisions rather than the NZSS 1900 provisions assumed in this assessment. A further reason for the strengths of the bridges exceeding the design standard levels is that in past design the passive resistance at the abutments was probably neglected, whereas in these simple analyses it was included and provided significant resistance in the longitudinal direction for many of the bridges.

Capacity to Demand Ratios: The ratios of capacity over demand listed in Tables 5 and 6 are less than 1.0 for 22 of the inspected bridges, indicating that based on the simple analyses many of the inspected bridge performed better than anticipated in either one or both main events. However, for a number of reasons these ratios are very approximate – for example:

- The capacity analyses are based on approximate methods that may contain conservatism in the simplifying assumptions.
- The response accelerations have been predicted from accelerographs located at some distance from the bridge sites and using periods of vibration that depend on structure and soil stiffness parameters that cannot be precisely predicted.
- Six of the inspected bridges are more than 100 m long. The response of these bridges in the longitudinal direction would be strongly influenced by travelling ground wave effects that result in a phase lag between the seismic input motions at the piers along the length. Travelling wave effects might also reduce the response in the transverse direction by interference between adjacent spans.
- On some of the shorter bridges inspected there would have been significant soil-structure interaction at the abutments that could have resulted in damping greater than the 5% critical assumed in estimating the response accelerations.
- Damping may have been higher than assumed for some of the bridges on sites with soft surface layers where non-linear soil response may have significantly affected the pile-soil interaction.

Structural Damage and Repairs: Thirteen of the inspected bridges received significant visible structural damage, as summarised in Table 7. The visible damage on five of these (Chaneys Road Overpass, Styx Overpass No 2, Port Hills Overpass No's 1 and 2 and the Hawkins River Bridge) was relatively minor and not difficult to repair. On some bridges possible damage could not be confirmed as the bases of the columns and the pile tops were not visible. Excavation or dewatering is necessary to complete these inspections and the bridges where this work is recommended are listed in Table 7.

Table 5. Results of Simple Analyses of Bridges for the Darfield Earthquake

State H/W	BSN	Group No	Bridge Name	Year of Design	EQ Design Standard	Design Equiv. ULS Coeff.	Estimated Periods of Vibration, s		Response Acceleration g		Ground Motion Return Period, yrs	ULS Capacity From Simple Analysis, g		Min Ratio Capacity/ Design Coeff.	Min Ratio Capacity/ Demand	Critical Components	
							Long	Trans	Long	Trans		Long	Trans			Longitudinal	Transverse
1S	3228		Kaiapoi Railway River	1968	NZSS 1900?	0.22	0.60	0.70	0.5	0.5	180	0.35	0.55	1.6	0.7	Abutment diaphragms	Pier cap, pier piles & columns
1S	3270/1		Waimakariri River	1965	NZSS 1900?	0.22	0.93	0.56	0.25	0.5	180	0.22	0.36	1.0	0.7	Pier walls	Pier piles
1S	3289	1	Chaneys Rd Overpass	1971	HBDB?	0.20	0.31	0.24	0.6	0.5	220	0.3	0.5	1.5	0.5	Shear keys	Pier & abut. piles
71	10		Cam Road Underpass	1967	NZSS 1900?	0.22	0.64	0.65	0.5	0.5	180	0.45	0.45	2.0	0.9	Pier columns	Pier columns
MIS	3264		Tram Road Underpass	1966	NZSS 1900?	0.22	0.7?	0.7?	0.5	0.5	180	-	-	-	-	Not analysed	Not analysed
MIS	3239		Ohoka Road Underpass	1966	NZSS 1900?	0.22	0.6?	0.6?	0.5	0.5	180	-	-	-	-	Not analysed	Not analysed
CCC	-		Old Waimakariri River	1929	None	-	0.65	0.15	0.5	0.4	180/220	0.2	0.6	-	0.4	Abut. back walls	Pier piles
1S	3810	2	Selwyn River	1920	None	-	0.50	0.25	0.5	0.6	520	0.25	0.35	-	0.5	Pier piles	Pier piles
74	21	3	Styx Overbridge No 1	1936	0.1g?	0.18	0.3?	0.3?	0.4	0.4	170	-	-	-	-	Pier columns	Pier columns
74	22		Styx Overbridge No 2	2006	BM 2003	0.20	0.4	0.5	0.4	0.4	170/350	0.4	0.4	2.0	1.0	Pier columns	Pier columns
73	33		Heathcote River (Opawa)	1989	HBDB	0.20	0.2	0.3	0.55	0.55	190	1.5	0.4	2.0	0.7	Abutment piles?	Pier columns
74	119	4	Anzac Drive	1999	BM 1994	0.24	0.30	0.15	0.5	0.3	190	0.55	0.55	2.3	1.1	Abut. linkage rods	Pier columns
CCC	-		Bridge Street	1978	HBDB	0.20	0.33	0.31	0.5	0.5	190	0.4	0.3	1.5	0.6	Pier columns	Pier columns
74	195		Heathcote River	1962	BM 1956	0.18	0.35	0.35	0.5	0.5	290	0.5	0.42	2.3	0.8	Abutment Piles	Pier columns
74	210	5	Railway Overbridge	1962	BM 1956	0.18	0.4?	0.26	0.5	0.55	290	0.35	0.33	1.8	0.6	Abutment piles	Pier columns
74A	7		Rutherford Street	1983	HBDB	0.20	0.63	0.53	0.4	0.45	220	0.45	> 0.5	2.3	1.1	Pier columns	Not analysed
74	215/6		Port Hills Rd Overpass's	1962	BM 1956	0.18	0.25	0.50	0.8	0.5	650/510	0.6	0.3	1.7	0.6	Abutment sliding	Pier columns
74	217/8	6	Horotane Valley O/P	1962	BM 1956	0.18	0.45	0.92	0.5	0.2	650/510	0.25	0.25	1.4	0.5	Abutment sliding	Pier columns
74	235		Heathcote Valley O/P	1962	BM 1956	0.18	-	-	0.4	0.4	650	> 0.4	> 0.4	> 2.2	> 1.0	Not analysed	Not analysed
75	80	7	Halswell River (Lands.)	1935	0.1g?	0.18	-	-	0.35	0.35	450	< 0.32	> 0.32	-	< 1.0	Abut. walls & piles	Abutment piles
77	769		Wairiri Stream	1962	BM 1956	0.18	-	-	0.5	0.5	710	> 0.5	> 0.5	> 2.8	> 1.0	Abutment walls	Abutment piles
77	790		Selwyn River	1929	None	-	0.44	0.34	0.9	0.9	710	0.5	0.35	-	0.4	Pier & abut. piles	Pier piles
77	854	8	Waianiwaniwa River	1933	0.1g?	0.18	0.16	0.17	0.9	0.95	710	> 0.9	0.6	3.3	0.6	Abutment walls	Pier piles
77	902		Hawkins River	1939	0.1g?	0.18	0.27	0.22	0.9	0.9	710	0.5	0.5	2.8	0.6	Abut. piles. Pier & abut. walls	Pier piles

Table 6. Results of Simple Analyses of Bridges for the Christchurch Earthquake

State H/W	BSN	Group No	Bridge Name	Year of Design	EQ Design Standard	Design Equiv. ULS Coeff.	Estimated Periods of Vibration, s		Response Acceleration g		Ground Motion Return Period, yrs	ULS Capacity From Simple Analysis, g		Min Ratio Capacity/ Design Coeff.	Min Ratio Capacity/ Demand	Critical Components	
							Long	Trans	Long	Trans		Long	Trans			Longitudinal	Transverse
74	21	3	Styx Overbridge No 1	1936	0.1g?	0.18	0.3?	0.3?	0.5	0.5	180	-	-	-	-	Pier columns	Pier columns
74	22		Styx Overbridge No 2	2006	BM 2003	0.20	0.4	0.5	0.45	0.4	180/390	0.4	0.4	2.0	0.9	Pier columns	Pier columns
73	33		Heathcote River (Opawa)	1989	HBDB	0.20	0.2	0.3	1.0	1.0	420	1.5	0.4	2.0	0.4	Abutment piles?	Pier columns
74	119	4	Anzac Drive	1999	BM 1994	0.24	0.30	0.15	0.6	0.5	420	0.55	0.55	2.3	0.9	Abut. linkage rods	Pier columns
CCC	-		Bridge Street	1978	HBDB	0.20	0.33	0.31	0.6	0.6	420	0.4	0.3	1.5	0.5	Pier columns	Pier columns
74	195		Heathcote River	1962	BM 1956	0.18	0.35	0.35	0.9	0.9	1300	0.5	0.42	2.3	0.5	Abutment Piles	Pier columns
74	210	5	Railway Overbridge	1962	BM 1956	0.18	0.4?	0.26	0.9	1.0	1300	0.35	0.33	1.8	0.4	Abutment piles	Pier columns
74A	7		Rutherford Street	1983	HBDB	0.20	0.63	0.53	1.0	1.0	1500	0.45	> 0.5	2.3	0.5	Pier columns	Not analysed
74	215/6		Port Hills Rd Overpass's	1962	BM 1956	0.18	0.25	0.50	1.9	0.6	> 5000/ 1600	0.6	0.3	1.7	0.3	Abutment sliding	Pier columns
74	217/8	6	Horotane Valley O/P	1962	BM 1956	0.18	0.45	0.92	0.7	0.3	> 5000/ 1600	0.25	0.25	1.4	0.4	Abutment sliding	Pier columns
74	235		Heathcote Valley O/P	1962	BM 1956	0.18	-	-	1.0	1.0	> 5000	> 0.4	> 0.4	> 2.2	> 0.4	Not analysed	Not analysed
75	80	7	Halswell River (Lands.)	1935	0.1g?	0.18	-	-	0.4	0.4	250	< 0.4	> 0.4	-	< 1.0	Abut. walls & piles	Abutment piles

Note: Simple calculations were not carried out for Groups 1, 2 and 8 for the Christchurch earthquake.

Settlement of Approaches: There was significant settlement of the approaches on 10 of the inspected bridges. On these bridges steps at the interface between the ground at the abutment and the abutment backwalls varied in height between 25 and 100 mm.

- Chaneys Road Overpass was reduced to a single lane and speed restricted to 30 km/h for about four days following the Darfield earthquake and these restrictions were mainly required to allow the pavement to be reinstated on the approach ramps and at the step that formed at the abutment.
- Bridge Street Bridge was closed for approximately 10 days following the Darfield earthquake due to the differential settlement at the east abutment (Cubrinovski et al, 2010). A photograph taken after the earthquake showed a step of at least 100 mm at the soil interface with the backwall.
- The abutments of Chaneys Road and Bridge Street Bridges were not constructed with settlement or friction slabs, which, if fitted, may have reduced the height of the abutment discontinuities and perhaps eliminated the need to close or restrict the bridges.
- The four bridges inspected on State Highway 77 were closed for a period of about six weeks but this was because of a slip that occurred a few kilometres west of the Wairiri Stream Bridge. Differential settlement was significant on the approaches of the Wairiri Stream and Selwyn River bridges and if the slip had not closed the highway these bridges may have been closed until the approach pavement was repaired. The abutments of these two bridges were not fitted with settlement or friction slabs.

Effects of Liquefaction: Significant liquefaction occurred near at least six of the inspected bridges reported in Appendix B. However, only the Anzac Drive, Bridge Street and Halswell River Bridges were seriously damaged by lateral spreading resulting from liquefied soil layers or, alternatively, failure of weak soil layers. Liquefaction caused significant differential settlement of the Halswell River Bridge approaches and steps of about 50 mm developed at the soil-abutment wall interfaces but there are no reports indicating that the bridge was closed by the settlement or other damage following the earthquake. Like the other bridges constructed prior to the 1960's the abutments of this bridge were not constructed with settlement or friction slabs. In addition, a number of Christchurch City Council bridges, only inspected by Brabhaharan and not included in Appendix B, were damaged due to liquefaction and lateral spreading in the 22 February 2011 Christchurch earthquake.

Effects of Friction or Settlement Slabs at Abutments: Of the bridges inspected, six were constructed with friction slabs at the abutments and two were constructed with settlement slabs. With the exception of the Styx Overpass No 2 no significant differential settlement steps developed at the abutments of these bridges. Five of the inspected bridges performed satisfactorily without friction or settlement slabs, so although the abutment slabs appeared to prevent settlement steps it is not possible to draw firm conclusions about their effectiveness in earthquakes.

Six of the inspected bridges without abutment slabs had abutments on spread footings, which would be less susceptible to differential settlement than abutments on piles. Although there was insufficient evidence to draw firm conclusions about the benefits of abutment slabs it appeared that on several bridges they prevented significant differential settlement steps. For example, on the Cam Road Underpass a differential settlement step of more than 50 mm developed in the footpath that was outside the area covered by the friction slab, and settlements of about 100 mm were evident on the front of the abutment seatings above the top of the aprons.

Linkages and Retrofits: Except for the Old Waimakariri Bridge, all the multi-span bridges inspected were fitted with linkages of various types at both the abutments and piers. Six of the bridges had continuous decks at the piers and the older bridges were generally linked by inclined bars between the beams and piers or by horizontal bars in the bottom of the beams.

Seven of the inspected bridges (counting twin bridges as two bridges) had been retrofitted with linkage systems. A summary of the strengthening components installed on each of these bridges, earthquake damage to the bridges and the performance of the retrofits is given in Table 9. Six retrofitted bridges sustained light to moderate structural damage and there was clear evidence that the strengthening was effective in reducing the level of damage. The SH1 Selwyn River Bridge was undamaged but it was not clear how effective the linkages had been in preventing damage.

Two of the bridges that were not retrofitted with linkages are of interest:

*SH1 Kaiapoi Railway and River Bridge**:*** The severe damage to the diaphragms anchoring the linkage bolts at the northern abutment and the failure of two of the span linkage bolts at one of the piers illustrates that on long bridges and under unfavourable circumstances linkage bolts can be heavily loaded. Evidence from this and other overseas earthquakes has demonstrated that in strong shaking, and particularly on sites with weak layers of soils, linkage systems at abutments and piers can distribute loads to reduce the damage to critical components and prevent simply supported spans from sliding off their seatings. However, it is essential that the elements that resist the forces in the linkage bolts can do so without damage, which was not the case for the abutment diaphragms on this bridge. Current practice is to ensure that the diaphragms can readily resist the yield forces from the linkage bolts. It is also important that the linkage bolts can stretch in a ductile manner without sudden breakage, as happened to the two failed linkage bolts at the piers. The reasons for the failure of these two linkages are still being investigated.

Old Waimakariri River (Main North Road) Bridge: This bridge has minimal interspan connection in the form of holding down bolts only through the outer flanges of the simply supported outer beams, and no linkage system at either the piers or abutments. In the Darfield earthquake the southern abutment backwall was damaged and lateral spreading resulting in cracking of the pier nearest to the abutment. Further spreading damage occurred at this pier during the Christchurch earthquake. In the Darfield earthquake the intensity of shaking at this site was estimated to be at the 180-year return period level and in stronger shaking large inelastic deformations would occur in the pier walls and pile foundations that would lead to a high risk of spans falling from their seatings. This bridge should be retrofitted with a linkage system to reduce the risk of collapse in large earthquakes.

Overall Performance of the Bridges:

Single Span Bridges: Three of the bridges inspected (Halswell River Bridge, Heathcote Valley Overpass and the Wairiri Stream Bridge) comprised simply supported single spans on high abutment walls. These bridges are essentially locked into the soil and the abutments are loaded by the shearing deformations in the soil over the height of the walls in addition to the inertia forces from the superstructures.

The intensity of shaking experienced by the Heathcote Valley Overpass was greater in the Christchurch earthquake than in the Darfield earthquake but the other two bridges experienced stronger shaking in the Darfield earthquake.

- The Halswell River Bridge was seriously damaged by lateral spreading from liquefaction in the Darfield earthquake. The shaking was estimated as being of 450-year return period.

Had the bridge not been subjected to lateral spreading pressures it is unlikely that it would have suffered significant damage.

- At the Heathcote Valley Overpass site the equivalent return period level of shaking intensity was estimated to be greater than 5000 years in the Christchurch earthquake and about 650 years in the Darfield event. The bridge was undamaged.
- At the Wairiri Stream Bridge the equivalent return period level of shaking intensity was estimated to be 710 years in the Darfield event. The bridge was undamaged.

The simple structural analysis calculations indicated that if they were founded on competent ground, all three bridges would perform satisfactorily in events with return periods in excess of 1000 years and be unlikely to collapse in a 2500-year return period event.

Multi-span Bridges Constructed Prior to 1940: Six of the multi-span bridges inspected were constructed before 1940 (Old Waimakariri, Selwyn River SH1, Styx No. 1, Selwyn River SH77, Waianiwaniwa River and Hawkins River). Except for Styx No 1 all these bridges have wall type piers with both the piers and abutments founded on vertical reinforced concrete piles. The pile tops are the critical sections under transverse loading, and the pile sections at about 1 metre below ground level and the base of the pier walls the most critical sections for longitudinal loading.

Three of the bridges are less than 100 m long and on these bridges the passive soil resistance of the relatively high abutment walls provides a significant part of the longitudinal resistance. For the three longer bridges, which exceed 175 m in length, the simple analyses and their good performance in the earthquake indicated that travelling wave effects might reduce the longitudinal inertia loads to significantly less than those calculated assuming synchronised input motions along the length of the bridge.

Except for the Styx Overpass No 1 Bridge, the pre-1940 multi-span bridges received stronger shaking in the Darfield earthquake than in the Christchurch earthquake. Two of the six bridges received moderately significant visible structural damage in the Darfield earthquake.

- At the Old Waimakariri River Bridge the equivalent return period level of shaking intensity was estimated to be 180 years in the Darfield event. Abutment backwall damage and cracking occurred in some of the piers.
- At the Hawkins River Bridge the equivalent return period level of shaking intensity was estimated to be between 340 and 710 years in the Darfield event. Pile top spalling and cracking occurred.
- At the Styx Overbridge site the equivalent return period level of shaking intensity was estimated to be between 170 years in the Darfield event and 180 years in the Christchurch event. No structural damage to the Styx No 1 Bridge was found, although the bridge is on short stiff multi-column piers on spread footings and some local areas of liquefaction were observed.

The simple structural calculations indicated that, with the exception of the Old Waimakariri River Bridge, the bridges would perform satisfactorily in events with return periods of 1000 years. The wall type piers loaded out-of-plane and the spirally wound cages in the octagonal piles have moderate levels of ductility (giving overall ductility factors of at least 2.0) so although there would be significant damage at the 1000-year return period level there would be a very low risk of collapse. There would be serious damage to the bridges and a moderate risk of collapse at the 2500-year return period level. If the Old Waimakariri Bridge were subjected to a 1000-year return period event the abutment backwall and the abutment and pier holding down bolts would fail, exposing the beams to a very high risk of falling from their seatings on both the piers and abutments.

Multi-span Bridges Constructed Between 1962 and 1972: Ten of the multi-span bridges inspected (counting twin bridges as one) were constructed between 1962 and 1972 (Group 1: Kaiapoi Railway/River, Waimakariri, Chaneys, Cam, Tram, Ohoka; Group 5: Heathcote River, Railway Overbridge; Group 6: Port Hills and Horotane Valley). They are all well detailed and constructed. Their only obvious shortcoming is that the primary lateral load resisting elements (piers and pile foundations) are not specifically detailed to have high levels of ductility, which were first specified in the 1971 Highway Bridge Design Brief. A variety of different pier shapes and foundation types were used but most of the substructure elements would have moderate levels of ductility or provide higher damping from soil-structure interaction than is usually assumed. With the exception of the Kaiapoi Railway River and Waimakariri River Bridges, all the bridges were less than 80 m long. Passive resistance at the abutments provided significant resistance to longitudinal inertia loads on all the bridges in this age category except for the two longer bridges.

- At the Kaiapoi Railway River Bridge the equivalent return period level of shaking intensity was estimated to be between 180 and 220 years in the Darfield event. Significant structural damage occurred to the abutment diaphragms and two of the pier linkage bolts failed on this bridge.
- At the Chaneys Road Overpass the equivalent return period level of shaking intensity was estimated to be between 180 and 220 years in the Darfield event. Apparent yield extension occurred to the abutment linkages on this bridge.
- In the Darfield earthquake the Ohoka Road Underpass received a similar intensity of shaking to the other Group 1 bridges. No significant structural damage was observed at Ohoka in this event and there were no reports of damage following the 22 February 2011 Christchurch earthquake but spalling damage was observed at ground level in the south column of the eastern pier following the M_L 6.0 aftershock on the 23 December 2011. The epicentre of this 7 km deep event was located 18 km to the south of the bridge. Removal of the cover concrete from the damaged column revealed poorly compacted concrete surrounding the main bars. A circumferential crack of 0.5 mm in width extended around the column base.
- The remaining seven bridges built between 1962 and 1972 and subjected to shaking intensity estimated to be between 180 and 220 years were essentially undamaged by the Darfield earthquake, and no further significant damage was reported to the Group 1 bridges during the Christchurch earthquake.
- Of particular interest was the performance of the two Group 5 and the Group 6 twin Port Hills Road and Horotane Overpass bridges. Excluding the very long return periods indicated by the HVSC records for short period spectral accelerations, the equivalent return period level of shaking intensity was estimated to be 250 to 580 years over the average of the 0.1 to 0.9 second spectral ordinates in the Darfield event, during which the bridges were essentially undamaged. However, during the Christchurch earthquake, the Group 6 bridges were moderately damaged, with cracking and spalling damage occurring in the abutments and piers of the two twin bridges. The equivalent return period level of shaking intensity was estimated to be between 1300 and > 5000 years over the average of the 0.1 to 0.9 second spectral ordinates in this event. Surprisingly, the Group 5 bridges received no significant visible damage.

The simple structural analysis calculations indicated that if founded on competent ground, all ten of the multi-span bridges would perform satisfactorily in events with return periods up to at least 1000 years. In longer return period events damage to the piers of the Waimakariri River Bridge could be severe but the other bridges would probably survive a 2500-year return period event although some of them might be seriously damaged. The Chaneys Road Overpass might be damaged by liquefaction in a 500-year return period event as there was evidence of significant liquefaction at the site in the 220-year return period shaking intensity of the Darfield earthquake.

Multi-span Bridges Constructed Between 1973 and 2006: Five of the multi-span bridges inspected were constructed between 1973 and 2006 (Styx No. 2, Heathcote River (Opawa), Anzac Drive, Bridge Street and Rutherford Street). They were all detailed to provide high levels of ductility in the critical plastic hinge zones in the piers and with overstrength in the foundations to prevent damage to their piles.

- At the Styx Overbridge No. 2 the equivalent return period levels of shaking intensity were estimated to be between 170 and 350 years in the Darfield event and 180 and 390 years in the Christchurch event, during which flexural cracking and minor spalling occurred in the tallest piers.
- At the Heathcote River (Opawa) Bridge the equivalent return period levels of shaking intensity were estimated to be between 80 and 190 years in the Darfield event and 350 and 420 years in the Christchurch event. The bridge was undamaged but there were subsequent reports of some settlement of the piled abutments having occurred.
- At the Anzac Drive Bridge the equivalent return period levels of shaking intensity were estimated to be between 80 and 190 years in the Darfield event and 350 and 420 years in the Christchurch event. Whilst undamaged during the Darfield event, lateral spreading or sliding of the approach fills caused a 5º rotation of the abutments about its long transverse axis during the Christchurch event, which also caused extensive spalling in the bridge piers. There was displacement of the independently supported walkways under the bridge alongside the abutments during both events. Had liquefaction or failure in weak soil layers not occurred the bridge would probably not have been seriously damaged, although cracking and spalling would probably have occurred in the piers.
- At the Bridge Street Bridge the equivalent return period levels of shaking intensity were estimated to be between 80 and 190 years in the Darfield event and 350 and 420 years in the Christchurch event. Lateral spreading or sliding of the approach fills caused some rotation of the abutments with displacement of the elastomeric bearings during the Darfield event, and this was significantly increased by the Christchurch event. Had liquefaction or failure in weak soil layers not occurred the bridge would probably not have been seriously damaged although cracking and spalling may have occurred in the zones detailed for plastic hinging at the base of the octagonal piers.
- At the Rutherford Street Bridge the equivalent return period levels of shaking intensity were estimated to be between 220 and 290 years in the Darfield event and 1300 and 1500 years in the Christchurch event. The bridge was undamaged during the Darfield event but the high abutment walls rotated or displaced horizontally closing the abutment joint gaps during the Christchurch event. Lateral spreading may have been a factor causing this damage.

Of particular interest was the performance of the Anzac Drive and Rutherford Street bridges, which were undamaged in the Darfield earthquake but sustained moderate structural damage in the stronger intensity Christchurch earthquake. A more detailed analysis of their performance in the two events would provide valuable information on design requirements to limit damage from lateral spreading and soil pressures on abutment walls.

The simple calculations indicated that all the multi-span bridges constructed between 1973 and 2006 would perform satisfactorily in events with return periods up to at least 1000 years provided that their foundations were not subjected to large forces from lateral spreading. In the absence of large liquefaction induced lateral spreading deformations, all three bridges would be unlikely to collapse in a 2500-year return period event.

4. GEOTECHNICAL ENGINEERING ASPECTS

4.1 Introduction

The magnitude 7.1 Darfield earthquake led to a wide range of geotechnical effects over the affected area, with widespread liquefaction in eastern Christchurch and Kaiapoi. Geotechnical reconnaissance of the affected area was carried out by P Brabhaharan (20-23 September) and Janet Duxfield (20-22 September) of Opus International Consultants, some two weeks after the main earthquake event of 4 September 2010. Because the state highways happened to be located generally in areas of better ground conditions in this earthquake and were less affected, the geotechnical engineering reconnaissance extended to cover not only a representative section of the state highways, but also local roads in the affected areas of eastern Christchurch and the epicentral area of Greendale, Hororata and Darfield. This was to ensure that the lessons brought together for NZTA are as much as possible representative of the full range of effects from the earthquake.

Brabhaharan also carried out reconnaissances of the Christchurch area bridges and roads, during 24 February to 4 March 2011 and on 24 August 2011, following the 22 February 2011 Christchurch earthquake. These included inspections at state highway and Christchurch City Council bridges.

Geotechnical engineering aspects in relation to the bridges are included in the description of each bridge in Appendix B. An overview of the geotechnical engineering issues is presented in this section, and covers:

- Ground shaking
- Rock fall
- Earthquake induced slope failures
- Liquefaction and associated ground damage
- Geotechnical effects on bridges
- Retaining walls
- Tunnel

4.2 Ground Shaking

Darfield Earthquake

The Darfield earthquake occurred on a fault, the location of which was unknown. However, the presence of hidden earthquake fault sources was expected by geologists based on the general seismicity of the area (Opus, 2004).

The performance of the ground in earthquakes is generally considered to be governed by the peak ground accelerations and the duration of shaking.

The variation of the recorded peak ground accelerations are summarised in Table 7, see Figures 1 and 2 for locations.

Geologically the strongest shaking was in areas of predominantly coarse river gravels close to the foothills of the mountain range. The ground shaking was relatively lower (PGA 0.2g to 0.3g) in areas of finer sands and silts close to the coast in Christchurch and Kaiapoi (0.35g).

The duration of shaking was about 40 seconds overall. However, strong shaking lasted only 10 to 20 seconds.

Table 7. Overview of Recorded Peak Ground Accelerations from the Darfield earthquake

Location	Distance from Epicentre (km)	Peak Ground Accelerations	Comments
Epicentral Area – Darfield, Greendale and Hororata	Within 15 km	0.5g to 0.8g	
Rollerston, Lincoln	20 km to 30 km	0.3g to 0.5g	
Christchurch	30 km to 50 km	0.2g to 0.3g	
Lyttelton	50 km	0.5g to 0.8g?	Directionality effects
	60 km	0.3g to 0.4g	Local amplification?

Christchurch Earthquake (22 February 2011)

Larger peak ground accelerations of 0.3 g to 0.7 g and up to 1.5 g were recorded in central and eastern Christchurch during the 22 February 2011 earthquake. Large peak ground accelerations including high vertical accelerations were recorded in the Port Hills area, reflecting the location very close to the epicentre and probably topographical amplification effects.

4.3 Rock Fall

There were localised rock falls in the area, in particular the Port Hills area adjacent to the western portal of the Lyttelton tunnel, which posed a significant risk to the tunnels and required remedial work. The rock falls originated in a bluff at the top of the hills, with large boulders landing on the state highway on the approach to the tunnel portal and on the Summit Road over the Port Hills. More extensive rock falls occurred in the Port Hills area in the 22 February 2011 earthquake. These blocked some of the local roads in the Port Hills area, and also affected residential areas – see Figure 8.Rocks mobilised by the earthquake may continue to pose a hazard to the highway in after-shocks and rainfall events for a long period, and could require mitigation measures.

4.4 Earthquake Induced Slope Failures

There were a few earthquake induced slope failures affecting the highways in the Darfield earthquake. Notable examples are the failure of the downhill lane of the highway SH77 in Glentunnel area, and a failure of the steep approach embankment south of the Kaiapoi Railway River Bridge. There were a number of other overslips onto the highway which were cleared quickly after the earthquake.

In the Christchurch earthquake of 22 February 2011, there were a number of notable earthquake induced landslides that affected and closed local roads in the Port Hills area.

Figure 8. Rockfall on the Summit Road over the Port Hills

Figure 9. Earthquake induced landslide affecting road (SH77)

4.5 Liquefaction and Associated Ground Damage

Darfield Earthquake

The Darfield earthquake caused widespread liquefaction in the eastern parts of Christchurch, Kaiapoi and localised areas in Halswell. The liquefaction occurred generally in areas susceptible to liquefaction as mapped and published by Environment Canterbury (2004). However, the area affected in the Darfield event was less extensive than the area predicted to be susceptible to liquefaction in the Christchurch area.

Liquefaction may have been present in areas where it was not obvious at the surface due to the depth of the liquefied layer and the presence of a thick liquefaction resistant crust. In these areas there would still be ground subsidence and an impact on deeper members of structures such as piles.

Liquefaction and associated ground subsidence led to damage to roads in Christchurch and Kaiapoi in particular. Where there was lateral spreading, the damage to roads was more extensive. Uplift of manholes due to buoyancy in the liquefied soils led to obstructions in many roads, particularly in eastern Christchurch.

Christchurch Earthquake (22 February 2011)

The Christchurch earthquake, which was closer to the Christchurch urban area, caused extensive liquefaction in the eastern and central Christchurch in particular. The area affected was generally in areas mapped and published by Environment Canterbury, but locally was also in areas outside the mapped liquefaction hazard areas.

There was widespread liquefaction in the Christchurch central business district and this contributed to extensive damage to the roads and bridges.

Liquefaction may have contributed to damage to embankments, such as north of the Chaneys Road Overpass. The damage to the embankment may have been reduced by the higher overburden pressures compared to the surrounding area, where there was significant liquefaction, sand boils and lateral spreading. It would informative to carry out further ground investigations and analyses to verify this, as this may be important for future design and security of highway embankments.

It would also be prudent to install piezometers in liquefiable areas, with automatic loggers, so that the rise in porewater pressures during earthquakes and dissipation over a period of time after the main shock can be understood. This will help understand the performance and access along critical highways after large earthquakes.

4.6 Geotechnical Effects on Bridges

Where there was no liquefaction the bridges on state highways and local roads performed well in the Darfield earthquake. Even in the epicentral area near Darfield, Hororata and Greendale, there was little ground damage. Where there was significant liquefaction, there was damage to bridges, particularly local road bridges that were located in these vulnerable locations.

The most severe damage was to the CCC Bridge Street Bridge near New Brighton, where the approach embankments and abutment slopes underwent significant displacement, probably caused by lateral spreading due to liquefaction of the underlying ground. The embankment as well as gabion walls were severely damaged due to lateral spreading, and it is understood that the lateral spreading was ongoing for at least two weeks after the Darfield earthquake of 4 September 2010. The lateral spreading led to severe rotation of the abutments, exposing the raked octagonal piles. There is likely to be damage to the piles, immediately below the pile cap, and at the interface between the liquefied and underlying non-liquefied stiffer layer.

There was also some displacement of the abutment at the Anzac Drive bridge, where there was extensive liquefaction in the surrounding area. Probably the displacement was limited because of the short duration of shaking. Liquefaction was apparently recognised during the design of this bridge and the foundations were designed accordingly. It is reported that the ground liquefied during the construction of the piled foundations for the bridge (Greenfield, pers. comm.).

Liquefaction also affected the Smiths Bridge in the outskirts of Kaiapoi, and observations indicated that there was a lean on the first pier from the southern abutment.

There was damage to a number of the abutment walls in the Christchurch area. The damage in the Darfield earthquake was generally limited to cracking, mainly between the abutment and wing walls due to differential displacement. The extent of damage was probably reduced by the short duration of ground shaking in this earthquake. The damage was much more extensive in the Christchurch earthquake, for example the CCC Ferrymead and Avondale Bridges - see Figures 10 and 11.

The pattern of damage to the bridges indicates that bridges were most vulnerable where the ground was poor and particularly where there was liquefaction and associated ground damage. The most vulnerable parts of the bridges were the abutments, and in some cases the piers closest to the abutments, where lateral spreading has a significant effect. There may be damage to piles at the underside of the pile caps and at the interface between liquefied and non-liquefied layers, but these locations are difficult to inspect. The upper parts of the piles in liquefied areas should be inspected where there is access to carry out such investigations.

Figure 10. CCC Ferrymead Bridge. Lateral spreading damage to abutment wall and piers in the 22 February 2011 Christchurch earthquake.

Figure 11. CCC Avondale Bridge. Damage to bridge abutment in the 22 February 2011 earthquake event.

Note: The Ferrymead and Avondale Bridges were not included in the detailed structural inspections but are of particular interest for the effects of liquefaction on the abutments.

4.7 Retaining Walls

No large retaining walls were observed during the earthquake reconnaissance after the September 2010 Darfield earthquake. The walls observed were mainly at relatively short bridge abutments. The walls appear to have experienced cracking where there was liquefaction and lateral spreading. On short bridges the walls appear to have been propped by the bridges. There was cracking observed in some walls, particularly at the interface with wing walls. Given the large liquefaction and displacements, it is difficult to differentiate retaining wall displacements. However, given the short duration of shaking, significant retaining wall displacements cannot be expected.

A large number of retaining walls in the Port Hills and Lyttelton area were severely affected by the 22 February 2011 Christchurch earthquake. The majority of these walls were generally poorly designed and constructed.

The recently constructed reinforced soil retaining walls on the Christchurch Southern Motorway performed well but these were not inspected in detail. The bridge abutment walls were constructed on ground improved with stone columns.

4.8 Tunnel

The Lyttelton tunnel was located in an area of moderate ground shaking in the Darfield earthquake and very strong shaking in the Christchurch earthquake. There was no significant damage to the tunnel. However, a quick drive-through reconnaissance indicated some diagonal cracks in the lining. This needs to be inspected in greater detail.

Table 8. Results of Inspection of Bridges and Recommendations – see Appendix B for more details

State Highway	BSN	Group No	Bridge Name	Summary of Structural Damage Observed After Both Darfield and Christchurch Earthquakes	Items Requiring Further Inspection
1S	3228	1	Kaiapoi Railway River	Severe damage to abutment diaphragms from linkage loads. Cracking in some span diaphragms. Two linkage bolt failures.	Bases of pier columns and tops of outer piles at piers.
1S	3270, 3271		Waimakariri River	None.	Tops of outer piles at piers.
1S	3289		Chaneys Rd Overpass	Loose retrofitted linkage bolts at Abutment A. Disturbance to abutment aprons.	Pile tops at Abutment D and Pier C. Wall columns under seat at Abutment A. Assess liquefaction below approaches.
71	10		Cam Road Underpass	None.	Bases of pier columns.
MIS	3264		Tram Road Underpass	None. (Fractured drainage pipe connections.)	Bases of pier columns.
MIS	3239		Ohoka Road Underpass	None. (Fractured drainage pipe connections.) Spalling damage at base of one column following the 23 December 2011 aftershock.	Bases of pier columns.
Chch City	-		Old Waimakariri River	Cracking in several pier walls. Damage to south abutment walls.	Abutment piles, walls and holding down bolts.
1S	3810	2	Selwyn River	None.	None.
74	21	3	Styx Overpass No 1	None.	None.
74	22		Styx Overpass No 2	Flexural cracking and minor spalling in pier columns.	None.
74	119	4	Anzac Drive	Rotation of abutments and cracking and spalling in portal piers.	None.
Chch City	-		Bridge Street	Rotation of abutments and abutment pile damage. Closing of abutment joints and impact damage. Cracking of one pier column.	Pier column bases and abutment piles.
73	33	5	Heathcote River (Opawa)	Permanent displacements at abutments, minor settlements and walking of bearings at abutments.	Pier column bases and abutment piles.
74	195		Heathcote River	None.	None.
74	210		Railway Overbridge	None.	Pier and abutment pile tops.
74A	7		Rutherford Street	At abutments closing of joints, pounding and displaced bearings.	Bases of all piers.
74	215, 216	6	Port Hills Rd Overpass's	Spalling at the base of two piers and flexural cracking in others.	Bases of all pier columns.
74	217, 218		Horotane Valley O/P	Sliding of pier footings resulting in fine cracking in piers. Sliding, settlement and transverse displacement at abutments resulting in cracking damage and loosening of linkage bolts.	Bases of all pier columns. Abutment linkage bolts.
74	235		Heathcote Valley O/P	None.	None.
75	80	7	Halswell River (Lands.)	Severe flexural cracking in abutment walls.	Tops of abutment piles, unless bridge is to be replaced.
77	769	8	Wairiri Stream	None.	None.
77	790		Selwyn River	Spall, or perhaps pile cracking, in top of one of piles under west abutment.	None.
77	854		Waianiwaniwa River	None.	Outer pile tops at piers.
77	902		Hawkins River	Spalling and cracking of pier pile tops.	Outer pile tops at abutments and piers where not visible.

Table 9. Performance of Retrofitted Bridges in the Darfield and Christchurch Earthquakes

SH	BSN	Bridge Name	Construction Date (Retrofit date)	Replacement Cost, M$	Retrofit Cost $000	Retrofit Details	EQ Mean PGA[1] g	Return Period at Bridge Site of Strongest Shaking Experienced in Main Events, Years	Earthquake Structural Damage	Earthquake Performance of Retrofit
1S	328	Chaneys Road Overpass	1972 (2003)	3.2	?	Linkage bolts at North abutments, which resist the entire longitudinal seismic load.	0.17 0.16 0.09	220	Yielding of retrofitted linkage bolts and disturbance to abutment aprons. Original downstand concrete shear keys anchoring superstructure may have been damaged.	Severe damage prevented by new linkage bolts. Without the retrofit it is likely that the bridge would have been closed to traffic for up to several months. There would have been some risk of collapse.
1S	381	Selwyn River	1920 (2006)	9.5	?	Linkage bolts and anchor brackets added to beam span ends designed to slide (alternate piers).	0.25 0.06 0.03	520	None. Performance better than expected.	Uncertain whether the linkage bolts contributed to the good performance but they may have been a factor.
74	215 216	Port Hills Road Overpass No 1 & 2	1963 (2009/10)	3.7	600	Linkage bolts and anchor brackets at piers and abutments. Steel circular shrouds on two piers in abutment slopes.	0.28 0.83 0.57	> 1600	Undamaged by Darfield EQ. Flexural cracking in lower halves of 5 of the 8 visible piers from Chch EQ (2 others hidden). Severe spalling on two central piers of the No 1 Bridge. Downslope soil movement at abutments.	Damage to the piers was reduced by the new linkages and soil shrouds. If more serious damage had occurred to the piers the bridges would have been closed for up to several months.
74	217 218	Horotane Valley Overpass No 1 & 2	1963 (2009/10)	2.4	160	Shear keys for transverse restraint at abutments, also bolted to beams longitudinally. Linkage bolts on outer beams at each pier to enhance horizontal diaphragm action.	0.28 0.83 0.57	> 1600	Shallow slope failures at both abutments with sliding of abutment and pier footings. Settlement of west abutments. Loosening of abutment linkage bolts. Large transverse displacement of No 2 Bridge at east end and abutment backwall cracking.	The new linkages at the piers successfully transferred load to the abutments preventing significant damage to the piers. The shear keys at the abutments appeared to be effective in limiting the transverse displacements. If more serious damage had occurred the bridges would have been closed to traffic for up to several weeks.
74	235	Heathcote Valley Overpass	1963 (2009)	0.8	100	Shear keys on top of abutment walls to prevent longitudinal sliding of the superstructure.	0.28 0.83 0.57	> 5000	Spalling damage to previous repairs at the ends of abutment nib walls. Loss of mortar between new shear keys and face of abutment walls.	The retrofitted shear keys appeared effective in preventing damage to the abutment nib walls and sliding of the superstructure. Significant damage to the abutment nib walls would have closed the bridge to traffic for a number of days.

Note: 1. Mean PGA is the mean peak ground acceleration of the two horizontal components of nearest strong motion accelerograph. PGA's listed in order for 4-Sep-10, 22-Feb-11 and 13-Jun-11 events.

5. CONCLUSIONS and RECOMMENDATIONS

The conclusions and recommendations are set out in full in the Executive Summary and are not repeated here.

6. ACKNOWLEDGMENT

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APPENDIX A

GROUND MOTIONS AND RESPONSE SPECTRA

APPENDIX CONTENTS

APPENDIX FIGURES

Note: The text in this appendix refers to figures and tables that are located either in the main text, in this appendix or in Appendix B. The figure and table numbers are accordingly prefixed with either no prefix, A or B.

A1. INTRODUCTION

The intensity of ground shaking at the bridge sites was of particular interest for the investigation. Information on intensity was one of the factors used to decide which bridges to inspect within the limited time available but, more importantly, the intensity of shaking estimated for each bridge site was used to interpret the damage observed and to compare the performance with what might be expected on the basis of current design and analysis methods for the estimated intensity of shaking experienced.

A2. STRONG MOTION ACCELEROGRAPHS

The Canterbury region is well covered by strong motion accelerographs (SMA's), both from the national GeoNet network and the Canterbury network. Table 1 lists the locations and peak ground accelerations (PGA's) recorded in both the Darfield and Christchurch earthquakes by the SMA's located within 80 km of the Darfield earthquake epicentre. The list only includes the recorder stations where records from the main Darfield shock had been published at the time of preparation of this report. No records were available from about nine stations within the radius considered. In addition to the epicentral distance, the table includes a distance from each station to the nearest point on the Greendale Fault rupture. The fault rupture distances listed in the table are the values published by Cousins and McVerry (2010), which were based on a smoothed model of the surface fault rupture. The fault rupture distance is of interest as attenuation of strong ground shaking is more directly related to this distance than to the epicentral distance.

In the Darfield earthquake there was a ten-fold variation in the PGA's recorded within 80 km of the epicentre. PGA's varied from 0.07 g at Westerfield, estimated to be 45 km from the fault rupture, to 0.72 g at Greendale which was located about 1.3 km north of the fault rupture towards its western end. Within a radius of up to 18 km (station CACS) from the Christchurch earthquake epicentre the PGA's recorded in the Christchurch earthquake were generally greater than recorded in the Darfield earthquake. Over this radius the PGA's varied from 1.46 g at the HVSC station (1 km epicentral distance) to 0.21 g at the PPHS station (12 km epicentral distance).

A3. INTENSITY OF SHAKING IN EPICENTRAL REGION

Figure 1 shows the location of the SMA's within 50 km of the Darfield earthquake epicentre and their relationships to the state highway bridges within this radius. The SMA locations have been colour coded to indicate the intensity of ground shaking as measured in the Darfield earthquake by the recorded PGA's. Also shown are Modified Mercalli Intensity (MMI) iso-seismals for magnitudes VI and VII. These have been derived from a colour graded MMI plot available from the GeoNet website and reproduced in Figure A14. The original plot was for the whole of the South Island and does not show clear boundaries between the intensity levels. It was enlarged to the area shown in Figure 1 and contours were sketched between zones where the colour grading changes were distinct. The MMI levels of VI and VII shown on the contours in Figure 1 are very approximate but the contours should give a reasonable indication of the areas where the intensity of shaking was considered to be the highest, as established by GeoNet using information from their public reporting procedure. Generally the iso-seismals are consistent with the recorded levels of PGA and were used to extend the shaking intensity information available from the SMA's to a wider area.

From Figure 1 it is evident that the intensity of shaking was greatest progressing in an east-west direction from the ends of the east-west trending fault trace and diminished more rapidly in a north-south direction. The intensity was unexpectedly high in Kaiapoi and the Heathcote Valley, which is located 10 km south-east of the Christchurch city centre. Kaiapoi and the Heathcote Valley are about 32 and 27 km respectively from the Greendale Fault rupture. The high PGA in Kaiapoi was consistent with the high MMI intensity recorded over a significant area in this location. In the Heathcote Valley the high intensity indicated by the recorded PGA was not very apparent from the iso-seismals so it may have occurred over a small area near the SMA station.

Figure 4 shows the location of the SMA's within 20 km of the Christchurch earthquake epicentre and all the SH bridges within this radius except for two small bridges to the south of the epicentre on SH 75 (BSN's 140 and 261). The SMA locations have been colour coded to indicate the intensity of ground shaking as measured by the recorded PGA's in the Christchurch earthquake. Also shown is the approximate MMI iso-seismal for intensity VIII. This was derived from an intensity map available on the GeoNet website which shows MMI's at specific locations estimated from the GeoNet public reporting procedure. The original of this map is shown in Figure A15.

A4. BRIDGE SITES

There are about 50 State Highway (SH) bridges located within 50 km of the Darfield earthquake epicentre and 30 located within 20 km of the Christchurch earthquake epicentre. Of the total of 50 bridges, 25 individual SH bridges were inspected as part of the study of the performance of the SH bridges in the earthquakes. Included in the total inspected were four sets of twin bridges with each pair in three of the sets having similar but not identical details. The fourth set of twin bridges, Styx Overbridges Numbers 1 and 2, were constructed at different times and differed significantly in detail. Two major Christchurch City Council (CCC) bridges that were damaged by the Darfield earthquake were also inspected as part of our study. Twenty-four of the total number of bridges inspected were inspected following the Darfield earthquake and 10 of these were re-inspected following the Christchurch earthquake. In addition three bridges not inspected after the first event were inspected following the second event.

Figures 2 and 3, which are drawn to a larger scale than Figure 1, show the locations of the bridges inspected and the locations of the SMA's. To simplify the figures the bridges not inspected have been omitted. The bridges are labelled in Figures 2 and 3 with their BSN as recorded in the NZTA Descriptive Inventory. The names of the inspected bridges, their state highway number, and their BSN are shown in Table 2. All the SMA locations within the areas covered by Figures 2 and 3 are labelled with the station abbreviation listed in Table 1. Figure 4 shows similar information to Figure 3 but indicates the intensity of shaking in the Christchurch earthquake.

A5. INTENSITY OF SHAKING AND SOIL LIQUEFACTION

Figures 1 and 2 show the MMI iso-seismals for the Darfield earthquake and Figure 3 (to a larger scale than Figure 2) the zones of soil liquefaction as identified by Tonkin & Taylor (2010) in this earthquake. The original more detailed Tonkin & Taylor plot of liquefaction zones covers a wider area than shown in Figure 3 and it is reproduced in Figure A16.

As shown in Figure 2, all the bridges inspected were within the MMI VI iso-seismal and four of these were within the VII iso-seismal zones. Bridge Numbers 3289, 119, 80 and the CCC

Bridge Street Bridge were located within the liquefaction zones shown in Figure 3. Bridges 10 and 3228 were close to a small liquefaction zone but may not have been directly affected by liquefaction. Although Figure 3 shows only the bridges that were inspected within about 20 km of the Christchurch city centre it is understood that liquefaction was not clearly evident at any of the other state highway bridge sites, either within or outside this area.

Figure 4 shows the MMI VIII iso-seismal for the Christchurch earthquake and the main zones of liquefaction that were identified following this event, as shown on a map available on the Earthquake Commission website (reproduced in Figure A17). Eleven of the bridges inspected following the Christchurch earthquake were within the MMI VIII iso-seismal and the other two (Styx Overpasses) were just outside it. Bridge Numbers 7, 33, 119, 195 and the CCC Bridge Street Bridge were located within large areas of liquefaction. Bridges 21 and 22 (Styx Overpasses) were close to a small pocket of liquefaction. In addition, a number of Christchurch City Council bridges were in areas of liquefaction, and were inspected by Brabhaharan. A comparison of the areas that liquefied in central Christchurch in the two main shocks and the 13 June aftershock is shown in Figure A18.

A6. ESTIMATES OF SHAKING INTENSITY AT THE BRIDGE SITES

Two different methods were used to estimate the intensity of shaking at the inspected bridge sites and the bridge response accelerations.

Method 1 Using Recorded Ground Motions

Grouping the Bridges for Ground Motions: The two SMA locations nearest to each bridge were identified. A number of the bridges were in groups quite close together resulting in the groups having the same two nearest SMA locations. Each of the groups of bridges that had the same two nearest SMA's is identified in Table 2. Grouping in this way reduced to eight the number of areas that needed to be investigated for site shaking intensity. One exception to all bridges in the group having the same two nearest SMA's was Group 5, where two of the three bridges had HVSC as one of their nearest two SMA's. In this case it was considered that the SMA record from CCCC (NZS 1170.5 Site Subsoil Class D), which was also quite close to all bridges in the group, was more likely to represent the bridge site ground motions than the SMA record from HVSC (NZS 1170.5 Site Subsoil Class C).

All the bridges inspected were located within 8 km of the nearest SMA to each bridge and all were within 16 km of the second nearest SMA. Because of these quite small separations, it was assumed that the arithmetic mean of the spectral ordinates of the ground motions recorded by the nearest two SMA's would provide a best estimate of the shaking intensity experienced by the bridges in each group. The ground motions recorded in the Darfield earthquake for all eight bridge groups were considered in the bridge performance assessment. The intensity of shaking at the Group 1, 2 and 8 sites in the Christchurch earthquake was clearly less than in the Darfield earthquake and the Christchurch records were not processed for these three groups. Peak ground accelerations in the Darfield and Christchurch earthquakes, and in the 13 June 2011 aftershock, recorded at the closest SMA sites to all bridge groups, are given in Table 2.

Using a weighted rather than an arithmetic mean of the spectral accelerations, based on the mean distance of the bridges in each group to the two nearest recording stations, was considered but because of the variability in the spectral ordinates from the recorded motions this refinement was not considered warranted. In Group 1, four of the bridges were up to 9 km closer to KPOC than SMTC and as the KPOC records had stronger short period components than SMTC these bridges may have been subjected to stronger intensities than given by the mean spectral ordinates. The other three bridges in the Group were approximately mid-way between the two nearest recorders. There was a 9.4 km difference in the distance to the two nearest recorders for the Group 2 bridge but overall the short period spectral accelerations from the Darfield earthquake records at these two stations did not differ greatly (although there are quite large differences between individual peaks in the ordinates). Likewise, for the four Group 8 bridges there were differences of up to 8.5 km to the two nearest recorders but again the overall differences in the low period spectral accelerations were not large. In the other five Groups the difference in distance between the bridges and the nearest two recorders was less than 3 km.

Cousins and McVerry (2010) have listed the local ground conditions at the SMA stations located within 100 km of the Darfield earthquake epicentre in terms of the Site Subsoil Classes used in NZS 1170.5. Most of the SMA stations were on Class D sites (Deep or Soft Soil) but there were a few on Classes E (Very Soft Soil), C (Shallow Soil) and B (Rock) sites.

Table 3 compares the NZS 1170.5 Site Subsoil Classes at the two SMA stations closest to each of the seven Bridge Groups with the Classes at the bridge sites. The table indicates the source of the information used to determine the Subsoil Class for each of the inspected bridge sites:

- Detailed Seismic Assessments (DSA) have been completed for four of the bridges and these indicate the Site Subsoil Class based on bore log information and the Consultant's geotechnical knowledge of the areas near the sites.
- Bore log information was available with the drawing records for ten other bridges and this was used to confirm whether the site was either a Class D or E. Information on the regional geology indicated that they would be in either one or other of the two classes.
- The other nine sites were assessed to be Class D based on information available on the regional geology, the site inspections and the class assigned to the nearest two SMA stations.

Table 3 shows that all the inspected bridges, except those in Group 6, are on Class D subsoil sites. The closest two SMA stations to the groups, excluding Group 6, are all located on Class D and E subsoil sites. The NZS 1170.5 design spectra for Class D and E subsoil sites are identical over the period range 0 to 0.5 seconds. The D spectrum is 15% lower than the E spectrum for a period of 0.7 seconds and 30% lower for a period of 0.9 seconds. Only two of the bridges inspected were expected to have periods of vibration greater than 0.7 seconds and these were expected to have periods of about 0.9 seconds. For the purpose of the investigation it was therefore assumed that difference in the soil conditions at the bridge sites and the nearest two recorders, if either were on Class E instead of D subsoil sites, was unlikely to make a significant difference to the prediction of the bridge response accelerations from the accelerograph records.

In Group 6 two of the bridges are on Site Subsoil Class D sites and one is on a Class B site. One of the nearest SMA recorders to the bridges in this Group was on a Class C site (HVSC) and the other one on a Class B site (LPCC). Again it was assumed that the difference in site classes between the bridges and the nearest recorders was unlikely to affect the predicted intensity of shaking for the bridge sites. It could have been assumed that the records from the SMA on the Class C site would better represent the shaking intensity for the bridges on the Class D sites and likewise for the SMA and bridge on the Class B site. However, because of the wide variability in recorded strong motions it was considered better to derive the assumed intensity of shaking from two rather than one recorder. In addition, the records from one of the two closest SMA's to Group 6 Bridges showed anomalous response spectrum characteristics, as discussed below.

Response Spectra: Acceleration and displacement response spectra for the two horizontal components of ground motion for the records from each of the two SMA's within each group were abstracted from the GeoNet website. These are shown in Figures B1 to B16 at the beginning of the information on each group in Appendix B. Each figure shows the four spectra of elastic 5% critical damped response from the two SMA's relevant to the group. For Bridge Groups 3 to 7 spectra are shown for the two main events and the 13 June 2011 aftershock. Acceleration time histories corresponding to all the response spectra plotted for the two main events are shown in Figures A19 to A44. The Darfield earthquake spectra and time-histories were taken from the originally published GeoNet data. Subsequently the accelerograph data has been reprocessed using an extended high pass filter. There are no significant differences between the original and reprocessed response spectra for periods less than about 1 second, which is the range of interest for the inspected bridges.

On each of the response spectra plots three design spectra are shown for comparison. Two are from NZS 1170.5 and are scaled to 250 and 1000-year return period levels. The NZS 1170.5 zone factor was varied from 0.22 to 0.3 to match the specified zone factor for the area surrounding each group of bridges. The 1000-year return period level corresponds to the lowest design level for fully elastic response specified in the NZTA Bridge Manual for state highway bridges with design working lives of 100 years (excluding importance Level 1 bridges which have low consequences of failure). Structures that have ductile capability are designed to yield at lesser strength values. The 0.18 g uniform spectrum shown in Figures B1 to B16 represents the effective design level for elastic response of older bridges, which made up at least half of the number inspected. These bridges were designed for 0.1 g at working stress level and a horizontal acceleration of at least 0.18 g would be expected to initiate plastic hinging (equivalent to "ultimate strength" moments) in flexural substructure members designed on this basis. For all bridge groups the response accelerations computed from the recorded ground motions clearly exceeded this design level by a large margin. However, it should be remembered that other loading cases, such as eccentric live loading or river debris loading, can control the bridge substructure design in some cases, depending on the form of the structure.

For Group 6 there is a large difference between the short period spectral ordinates of the response spectra from the two SMA locations. This was evident in both main events and the 13 June 2011 aftershock. The reason for this discrepancy is unclear but unusual local site conditions or the extraneous influence of the building in which the SMA was housed are possible explanations for the high PGA and spectral response for periods less than 0.6 seconds at the HVSC location. Cousins and McVerry (2010) stated that the HVSC station is at the head of the Heathcote Valley with the strong short-period response possibly caused by a shallow colluvial wedge of soil.

Averaged Spectral Values from Response Spectra: To simplify the information presented in the acceleration response spectra the PGA and spectral ordinates for periods of 0.2, 0.4, 0.6 and 0.8 seconds were abstracted and averaged over the four horizontal spectra in each group (two components from two SMA stations). Recently completed detailed seismic assessments have shown that the first mode horizontal periods of vibration for many bridges designed prior to 1970 are usually in the range of 0.2 to 0.9 seconds, with the transverse periods tending to be rather shorter than the longitudinal periods. This period range is similar to the range of the periods predicted for the inspected bridges based on simplified static analysis calculations. Generally periods of vibration in detailed seismic assessments are calculated assuming cracked section properties because it is assumed that the structures will be working at their upper strength limits. Since very little cracking was observed in the inspected bridges following the Darfield earthquake, their periods of vibration during the earthquake were probably in the range 0.15 to 0.8 seconds. Therefore it is reasonable to expect that the spectral accelerations derived for the period range of 0.2 to 0.8 seconds and illustrated in Figure 5 will give a good indication of the response accelerations experienced by the bridges in the earthquake. In addition to average spectral ordinates for 0.2, 0.4, 0.6 and 0.8 seconds the Christchurch earthquake records illustrated in Figure 7 were processed to give average spectral values for a 1.0 second period. This was to provide information for assessing the performance of several of the Group 6 bridges in this event where the very strong shaking led to cracking that would have lengthening their periods to greater than 0.8 seconds.

Prior to averaging the spectral ordinates the spectra were smoothed to a small degree to reduce the influence of any large unevenness in the curves. Smoothing was completed by taking a weighted average of adjacent spectral ordinates. The averaged PGA's and spectral ordinates (S_a: T = 0.2, 0.4, 0.6, 0.8 seconds) from the Darfield records are plotted in Figure 5 for the eight bridge groups. All the spectral ordinates for the period T between 0.2 to 0.6 seconds exceed 0.3 g. Similar information derived from the Christchurch records is plotted in Figure 7 for Bridge Groups 3 to 7.

Method 2 Using Attenuation Functions

The second method used for assessing the intensity of shaking at the sites of the eight groups of bridges was based on the McVerry *et al* (2006) attenuation functions for PGA and spectral ordinates that underlie the hazard model used to develop the NZS 1170.5 design spectra. Figures A1 to A4 show comparisons between the PGA's and spectral ordinates for T = 0.2, 0.4 and 0.6 seconds recorded in the Darfield earthquake with predicted mean and plus and minus one standard deviation error bounds computed using the McVerry *et al* attenuation functions for NZS 1170 Site Subsoil Classes D (deep or soft soil) and E (very soft soil). The corresponding comparisons for the Christchurch earthquake are shown in Figures A5 to A8. The McVerry *et al* functions are available for both the largest and the geometric mean of the two horizontal components of ground motion. Because the assessment of the performance of the inspected bridges was the subject of interest, the mean of the PGA and spectral ordinates from the two horizontal components were compared with the attenuation curves for the geometric mean, although attenuation of the largest horizontal component would normally be used for design and was used in the development of NZS 1170.5.

The recorded PGA's and spectral ordinates plotted in Figures A1 to A8 are from the recording stations listed in Table 1 with the exclusion of the four stations with Site Subsoil Classes B and C. In application of the McVerry *et al* attenuation functions for the Darfield earthquake comparison a strike-slip mechanism was assumed. Cousins and McVerry (2010) have indicated that most of the moment release was associated with a strike-slip mechanism although both reverse-mechanism and strike-slip components were thought to have occurred on the fault. The faulting mechanism in the Christchurch earthquake was identified as reverse faulting (GeoNet web site) and the parameters for this mechanism were used to derive the McVerry *et al* attenuation curves for the Christchurch event.

Figure A1 shows that the Greendale (GDLC) PGA at 1.3 km distance is considerably underestimated by the attenuation function as is the Kaiapoi (KPOC) value at 32 km. Most other records are within or close to ± one standard deviation of the median of the predicted values. Figures A2 to A4 show that in general the spectral ordinates were reasonably well predicted by the attenuation functions although there are a few recorded values outside the one standard deviation limits, demonstrating the wide variability expected in spectra computed from records at stations of similar distance from the source. The log-log plots disguise this scatter with the one standard deviation curves representing values that are typically ± 60% greater or less than the median values.

As shown in Figure A5 the Christchurch earthquake PGA's were underestimated by the attenuation curves at distances less than 12 km from the epicentre. Similarly the spectral ordinates for T = 0.4 and 0.6 were underestimated within this distance (Figures A7 and A8). However, the spectral ordinates for T = 0.2 were mainly within ± one standard deviation of the median at epicentral distances less than about 18 km (Figure A6).

Estimates of the PGA's and spectral ordinates (Sa: T = 0.2, 0.4, 0.6 and 0.8 seconds) in the Darfield earthquake were computed for each of the eight bridge groups using the McVerry *et al* attenuation functions for the strike-slip mechanism and the geometric mean of the two horizontal components. The shortest distance between the fault rupture and the bridges in each group was taken as the average of the distance for each individual bridge in the group as the variation of this distance within each group was small. Because of the relatively poor fit of the attenuation functions to the PGA's and spectral ordinates from the Christchurch earthquake at short epicentral distances, no attempt was made to use the attenuation functions to predict the PGA's or spectral ordinates for the bridge groups in this event. Only the short epicentral distances were relevant for the assessment of the bridge performance because beyond distances of about 15 km the shaking in the Darfield earthquake was stronger.

PGA's and spectral ordinates (Sa: T = 0.2, 0.4, 0.6 and 0.8 seconds) from the attenuation function method applied to the Darfield event are plotted in Figure 6 for the eight bridge groups. There is less variation of the spectral ordinates over the range 0.2 to 0.8 seconds than was the case for the method based on the records from the two closest SMA's to each group and plotted in Figure 5. The response accelerations vary from 0.3 to 0.8 g compared with the range of 0.3 to 1.1 g obtained using the two closest SMA's. The attenuation function approach, although not necessarily the most favoured prediction method for this investigation, does smooth out the variation in the spectral response as the attenuation functions are based on regression analysis of a large number of similar magnitude events. It reduces the influence of fault directivity and local site effects, which were evident in the SMA records used in the first method for Groups 6 and 8. Results from both methods can be used to obtain a range of likely acceleration response values for the bridges in each group. Because of the averaging procedures inherent in both methods they give a best estimate and do not predict the minimum or maximum possible response values. Lower and upper bounds to the response accelerations for each bridge group can be estimated by the error bars plotted in Figures 5, 6 and 7.

A7. DAMPING

The plotted spectra and spectral ordinates are for 5% critical damped elastic systems. Some of the bridges may have damping greater than 5% at the response levels experienced in the earthquakes, particularly the shorter bridges where soil-structure interaction at the abutments influences the response. To determine the influence of higher levels of damping on the response accelerations, 10% damped acceleration spectra corresponding to the 5% damped spectra shown in Figures B1 to B16 for the Darfield earthquake were abstracted from the GeoNet data base. The average response over the period range 0.2 to 0.6 seconds of the 10% damped spectra was found to be a factor of 0.8 times the 5% damped spectra. This reduction is identical to that obtained using the empirical expression of Kawashima and Aizawa (1986). For 15% damping, which is unlikely to be exceeded in the response of any of the inspected bridges, the Kawashima and Aizawa expression gives a reduction factor of 0.71.

A8. ANOMALIES IN GROUND MOTIONS

The PGA and short period spectral accelerations from the HVSC station records in both the Darfield and Christchurch earthquakes were higher than expected at this location, which was about 27 km from the Greendale fault rupture surface and 1 km from the Christchurch earthquake epicentre. The reason for these high values has not yet been adequately investigated. Figure B11 shows that the short period (0 to 0.5 seconds) response accelerations were typically about two times higher than for the spectra from the LPCC station located less than 3 km to the south. They were significantly higher than in the spectra from CCCC and PRPC to the north and for CMHS to the west (see Figures 3, B9 and B13). Ground conditions at the HVSC and LPCC stations are different but nevertheless, the difference in the spectra from these locations is larger than expected, making it difficult to estimate the response accelerations experienced by the bridges in Group 6.

The spectra from the Darfield earthquake LINC records had untypically high spectral accelerations for periods less than 0.2 s (see Figure B13 and the acceleration time histories in Figure A40). Again it is unclear whether this feature was caused by extraneous factors or was a particular feature of the ground motions. If it was a feature of the ground motions in this area, it may have influenced the response of the Halswell River Bridge, which is the only bridge in Group 7. This bridge is a very short bridge essentially locked into the ground and would be likely to respond at accelerations represented by the low period end of the spectra.

The HPSC station was clearly affected by liquefaction in both main events and this may have reduced the PGA and short period response accelerations (see Figure B7 and Figures A27 and A29) to levels lower than expected on an unliquefied site. Liquefaction occurred at the sites of both bridges within Group 4 and the HPSC spectral accelerations may therefore be a reasonable indication of the bridge response accelerations.

A9. VERTICAL ACCELERATIONS

Very high peak vertical accelerations were recorded at the SMA sites located within about 15 km of the Christchurch earthquake epicentre. Values recorded in both the main events at the SMA stations closest to Bridge Groups 3 to 7 are summarised in Table A1.

A10. DISPLACEMENT RESPONSE

The displacement response spectra presented in Figures B2 to B16 for each of the Bridge Groups were compared by computing the mean of the spectral ordinates of the four displacement response spectra on each figure associated with each group. These mean values are plotted in Figure A9 and A10 for the Darfield and Christchurch earthquakes respectively. In the Darfield earthquake the mean 5% critical damped displacement response at a 0.4 second period varied from 20 to 35 mm with the Group 8 records giving the largest response. In the Christchurch earthquake the 0.4 second period displacement responses varied from 20 to 70 mm with the Group 6 records showing the largest response. Reductions in the displacement spectral response for damping greater than 5% can be computed using the same reduction factors as for acceleration response – i.e. 0.8 and 0.71 for 10% and 15% critical damping respectively.

Table A1. Vertical Accelerations at SMA Sites Closest to Bridge Groups 3 to 7

Bridge Group No	SMA	Christchurch EQ			Darfield EQ		
		Epicentral Distance	Vertical PGA	Horizontal PGA	Fault Distance	Vertical PGA	Horizontal PGA
3	SMTC	14	0.18	0.18	23	0.17	0.17
3	PPHS	12	0.20	0.21	21	0.28	0.21
4	HPSC	9	0.86	0.25	28	0.13	0.17
4	NNBS	11	0.72	0.77	29	0.14	0.20
5	CCCC	6	0.69	0.48	22	0.16	0.23
5	PRPC	6	1.63	0.67	25	0.31	0.22
6	HVSC	1	1.46	1.46	27	0.28	0.62
6	LPCC	4	0.41	0.88	28	0.16	0.34
7	CMHS	6	0.80	0.40	20	0.26	0.25
7	LINC	19	0.08	0.16	9	0.77	0.43

The number of displacement cycles of the bridges during the earthquake can be estimated by computing the response to recorded accelerograms of an elastic single degree of freedom (SDOF) system. Figure A11 shows the 5% and 10% critical damped displacement response of a SDOF system with a period of 0.4 seconds computed using the Darfield earthquake DSLC N63W horizontal acceleration record as input (shown in Figure A21). This typical displacement response shows that many of the bridges in the areas of strongest shaking would have been subjected to a large number of displacement cycles.

A11. AFTERSHOCKS

Events: Between the 4 September 2010 Darfield and the 22 February 2011 Christchurch **ea**rthquakes there were 13 aftershocks with magnitude greater than 5.0. At the time of preparing this report (February 2012) there had been a further 25 aftershocks following the Christchurch earthquake with magnitudes greater than 5.0 (including the 13 June 2011 and 23 December 2012, M_w 6.0 aftershocks). The locations of the aftershocks prior to the Christchurch earthquake are shown in Figure 1 and those following this event in Figure 4. The largest aftershock prior to the Christchurch earthquake was a magnitude 5.6 event that occurred about 20 minutes after the main event.

Several of the aftershocks that followed the Darfield earthquake were close to SMA stations and two occurred very close to two bridges, leading to the thought that they may have caused damage. The first of these had a magnitude of 5.3 (local magnitude) and occurred on 6 September at 15 hours 24 minutes (Universal Time). The depth was reported as 15.3 km and the epicentre was located about 400 m from the SH1 Selwyn River Bridge. A PGA of 0.032 g was recorded at the DSLC SMA station, which is about 4 km from the bridge. The shaking intensity at the bridge site was probably greater than at the DSLC station but it is unlikely that the PGA at the site was greater than 0.05g. This is a lot lower than the PGA of 0.3 g estimated to have occurred at the site during the Darfield earthquake which did not cause any structural damage.

The second aftershock of interest was the Mw 4.8 (5.1 local magnitude) event on 18 October at 22 hours 32 minutes (Universal Time). The depth was reported as 5.0 km and the epicentre was located about 1.5 km south of the SH 75 Halswell River Bridge (Landsdown) almost directly under SH 75. This event caused renewed liquefaction on SH 75 and it was thought that it may have increased the cracking in the abutment walls of the Halswell River Bridge, which were damaged in the main event. A PGA of 0.039 g was recorded at the LINC SMA station, which at a distance of 7.6 km from the bridge is the nearest recorder to the bridge. The PGA at the bridge site would probably have been greater than at the LINC station (perhaps up to 50% higher based on attenuation distances) and the shaking intensity may have been sufficient to increase the lateral spreading. The Darfield earthquake, which caused significant lateral spreading at the bridge site, was estimated to have produced a PGA at the site of about 0.3 g.

Following the Christchurch earthquake the nearest aftershock to the SH bridges was the 13 June 2011 M_w 6.0 event at a depth of 9 km and with its epicentre located about 2.0 km from the Port Hills Road Overpasses, Horotane Valley Overpasses, Railway Overbridge, Garlands Road (SH 74A), Rutherford Street and Heathcote River bridges. The epicentre of the 23 December 2011 M_L 6.0 aftershock with a depth of 7 km was located 1.6 km to the east of the CCC Bridge Street Bridge and within 5.0 km of the ANZAC Drive and Heathcote River bridges. Both these aftershocks would have produced strong shaking at the nearby bridge sites but it is unlikely that they produced an intensity of shaking at any of the sites that exceeded the level reached in the 22 February 2011 M_w 6.2 Christchurch earthquake. The Christchurch earthquake main event was at a depth of 6 km and its epicentre was located within 3.5 km of all of the bridges mentioned above except the Bridge Street and ANZAC Drive bridges.

Comparison of the Effect of Aftershocks Compared with the Main Event: Records were available at LPCC for a magnitude 5.0 aftershock with an epicentre located about 800 m from the recorder. This aftershock was recorded on 7 September at 19 hours 49 minutes (Universal Time) and had a focal depth of 6.7 km. Comparisons of the acceleration response spectra and an acceleration time history from one of the horizontal components recorded at LPCC for the aftershock and the main event are shown in Figures A12 and A13 respectively. The PGA's and short period spectral ordinates from the aftershock were about 50% of the corresponding values in the main event and the duration of the strong shaking in the aftershock was very much shorter than in the main event. Although the LPCC aftershock had a magnitude less than the magnitude 5.3 event near the Selwyn River Bridge, the records from it suggest that the intensity of shaking from the aftershocks that were very close to the Selwyn and Halswell River Bridges was unlikely to have exceeded the intensity of shaking at the bridge sites in the Darfield earthquake and the duration was considerably shorter. With the exception of the 13 June 2011 aftershock, it also seems unlikely that any of aftershocks that followed the Christchurch earthquake would have produced shaking intensity at the state highway bridge sites that exceeded the intensity recorded in the two main events.

A12. RETURN PERIODS FOR SHAKING INTENSITY

Estimation of Return Periods: The return period for the ground shaking intensity associated with each bridge group was estimated by averaging the spectral ordinates of the GeoNet processed spectra over spectral ordinates for the period ranges 0.1 to 0.5 seconds and 0.5 to 0.9 seconds and comparing these with the NZS 1170.5 spectral shape factor for the site subsoil class at the recording station, adjusted using the NZS 1170.5 return period factor. For the comparison the NZS 1170.5 zone factor appropriate for each of the Bridge Group locations was applied. The spectral ordinates were averaged over the four components of ground motion associated with each Bridge Group. The 0.1 to 0.5 second range was chosen for the comparison as this corresponds to the plateau of the NZS 1170.5 shape factor for Site Subsoil Classes D and E. Thirty six points in the processed spectra from each record were averaged over this range. The range 0.5 to 0.9 seconds was selected as most of the bridges inspected had estimated periods in the 0.1 to 0.9 second range. Ten points at equal 0.05 second intervals in the processed spectra were averaged over the 0.5 to 0.9 second range.

Results of the Estimation: The return periods calculated for the shaking intensity corresponding to each bridge group by the above method are listed in Table 4 for both the Darfield and Christchurch earthquakes. The very strong spectral ordinates in the 0.1 to 0.5 second range of the record from the HVSC recording station, which was the closest recording station to the Group 6 bridges, indicated a return period for this part of the spectrum of about 4,000 years in the Darfield earthquake and greater than 10,000 years in the Christchurch earthquake. This very strong short period intensity was not found in the records from the other recording stations. Because of this anomaly, separate return periods are shown in Table 4 for the records from the two recorders closest to the Group 6 bridges. The return periods of shaking intensity varied considerably between the groups, as is also indicated by the spectral ordinates plotted in Figures 5, 6 and 7. The return periods are also sensitive to the period range of interest.

Comparison of Return Periods with Design Levels: Excluding the HVSC record (Group 6), the overall averages of between 140 to 580 years in the Darfield earthquake were well below the 2500-year return period used for current design of bridges on important routes. New bridges on the routes included in Groups 1 to 6 would be designed for the 2500-year return period level but with significant reductions of design forces by up to a factor of 6 if appropriate ductility were built into the structure. For bridges on the routes included in Groups 7 and 8 (SH's 75 and 77) a 1000-year return period level would probably be used. Design earthquake force levels are proportional to the Return Period Factor, which varies from 0.59 to 1.05 for the range of return periods of 140 to 580 years. The return period factors for 1,000 and 2,500 years are 1.3 and 1.8 respectively. Thus, for a fully elastic structure designed for a 2,500-year return period, the strength would be 1.8/1.05 = 1.7 times that designed for a 580-year return period event, or 3 times that designed for a 140-year event. However, the strength of the 2500-year structure would most likely be reduced by a factor of 2 to 6 if adequate ductility were designed into it.

The shaking intensity at the Group 3 to 6 sites, as measured by the equivalent return period of the spectral ordinates averaged over the 0.1 to 0.9 second range, was greater in the Christchurch earthquake than in the Darfield earthquake. For Groups 3 and 4 the average return period level was 280 and 390 years respectively, again well below the 2500-year design level for important routes. For Groups 5 and 6 the average return period level was 1400 and 5000 years (excluding the HVSC records) respectively. These return periods correspond to Return Period Factors of 1.4 and 2.4 and represent 0.8 and 1.3 times the 2500-year elastic force levels respectively.

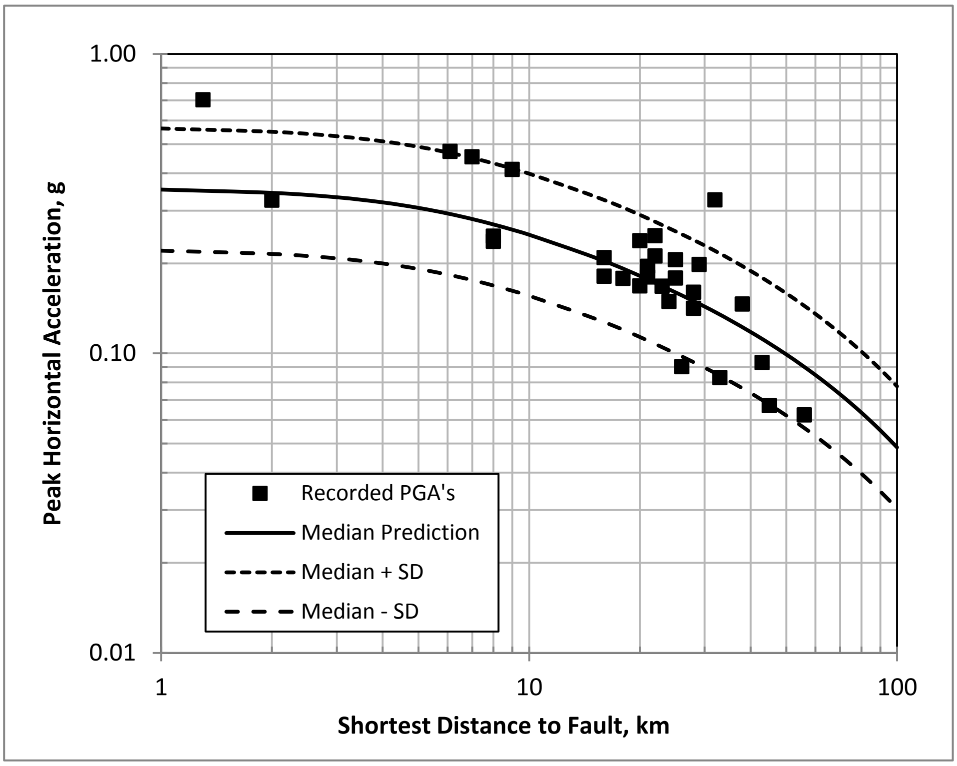


Figure A1. Darfield Earthquake. Comparison of recorded PGA's with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

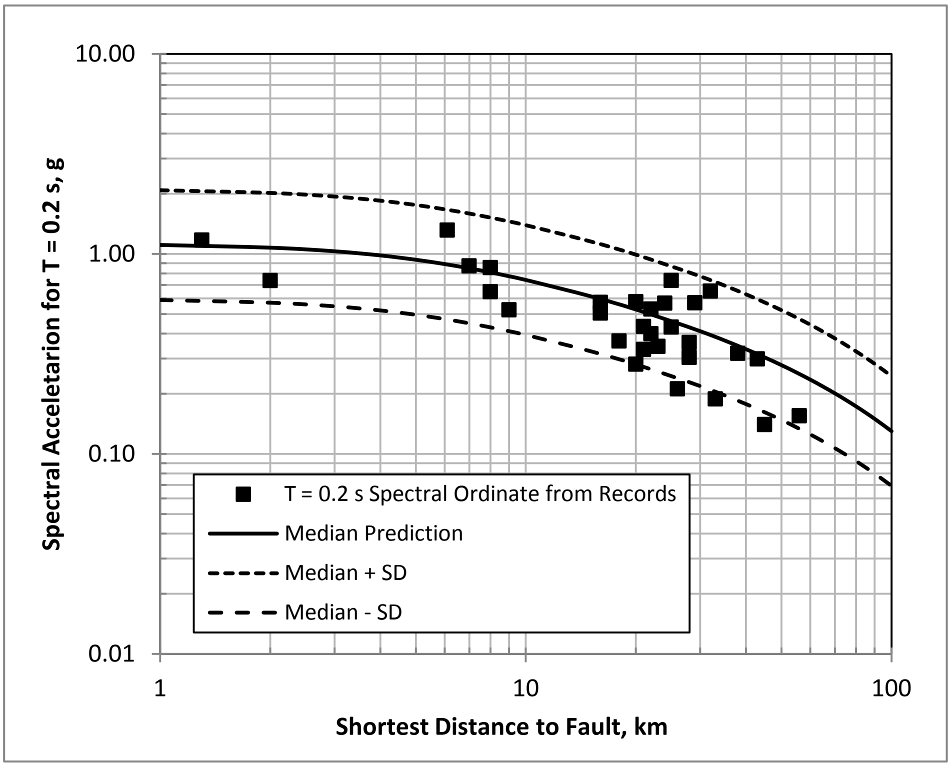


Figure A2. Darfield Earthquake. Comparison of spectral acceleration, T = 0.2 seconds computed from records, with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

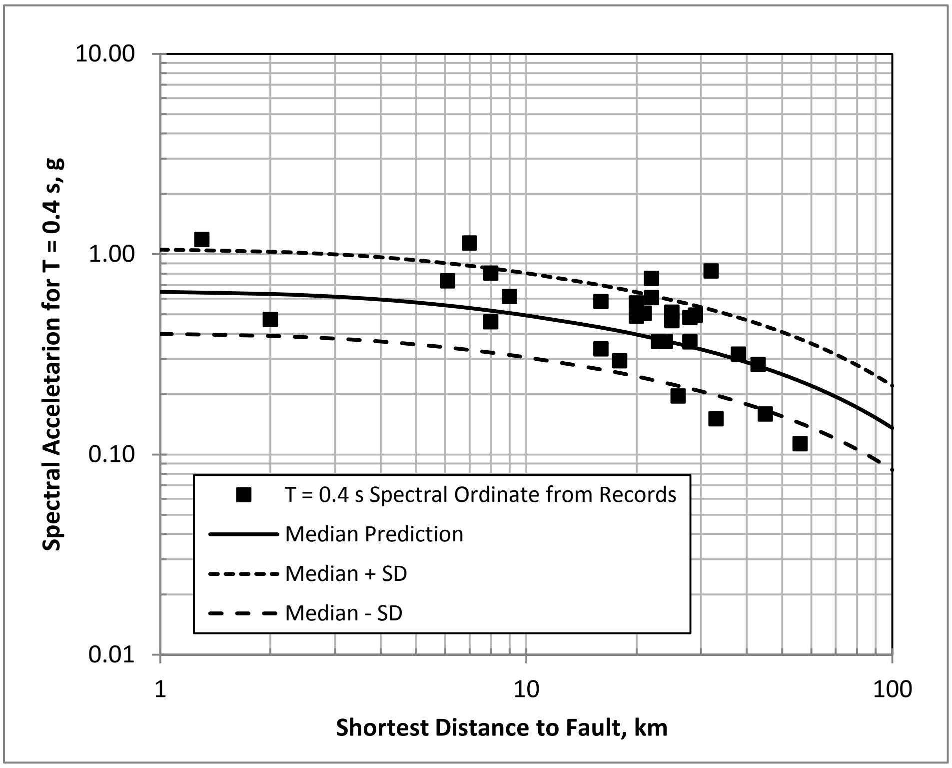


Figure A3. Darfield Earthquake. Comparison of spectral acceleration, T = 0.4 seconds computed from records with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

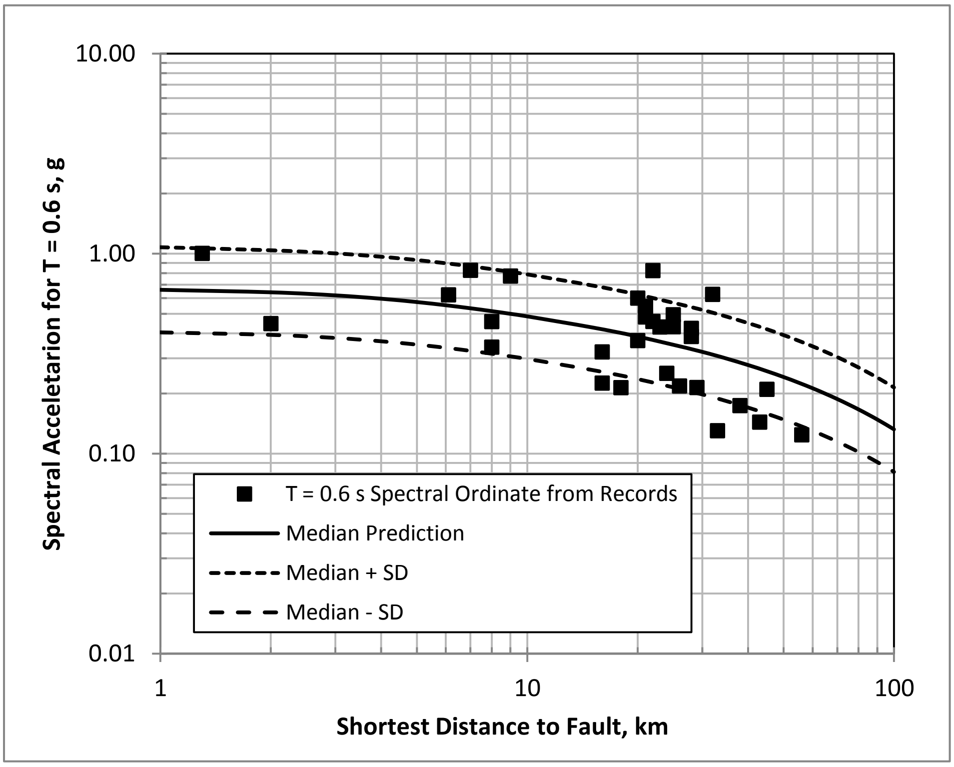


Figure A4. Darfield Earthquake. Comparison of spectral acceleration, T = 0.6 seconds computed from records, with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

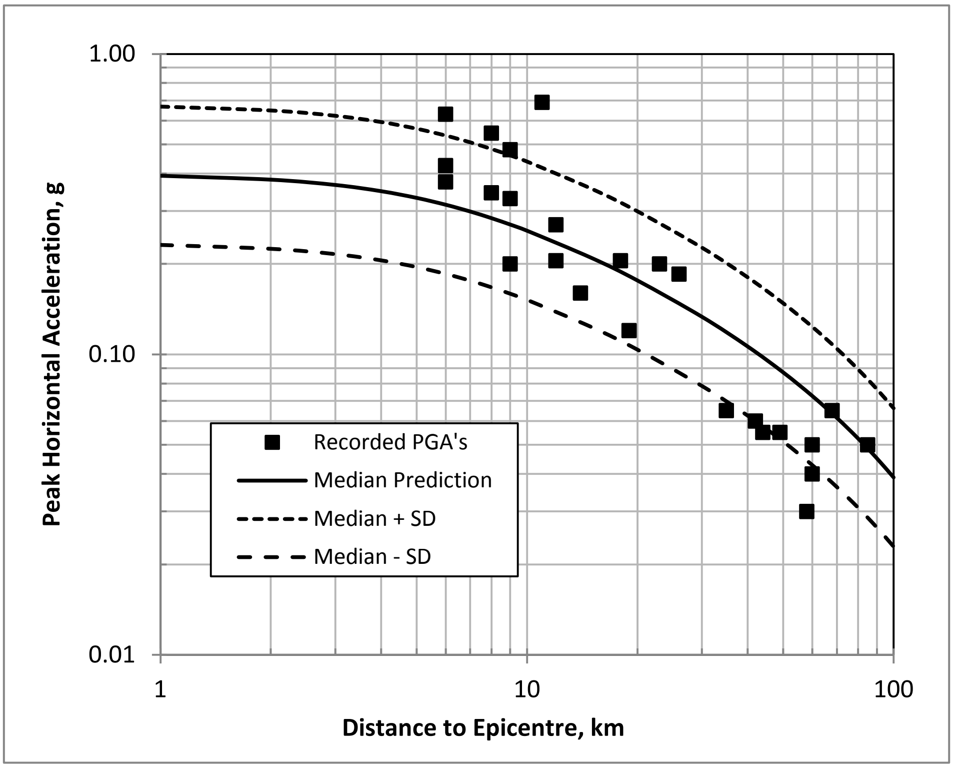


Figure A5. Christchurch Earthquake. Comparison of recorded PGA's with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

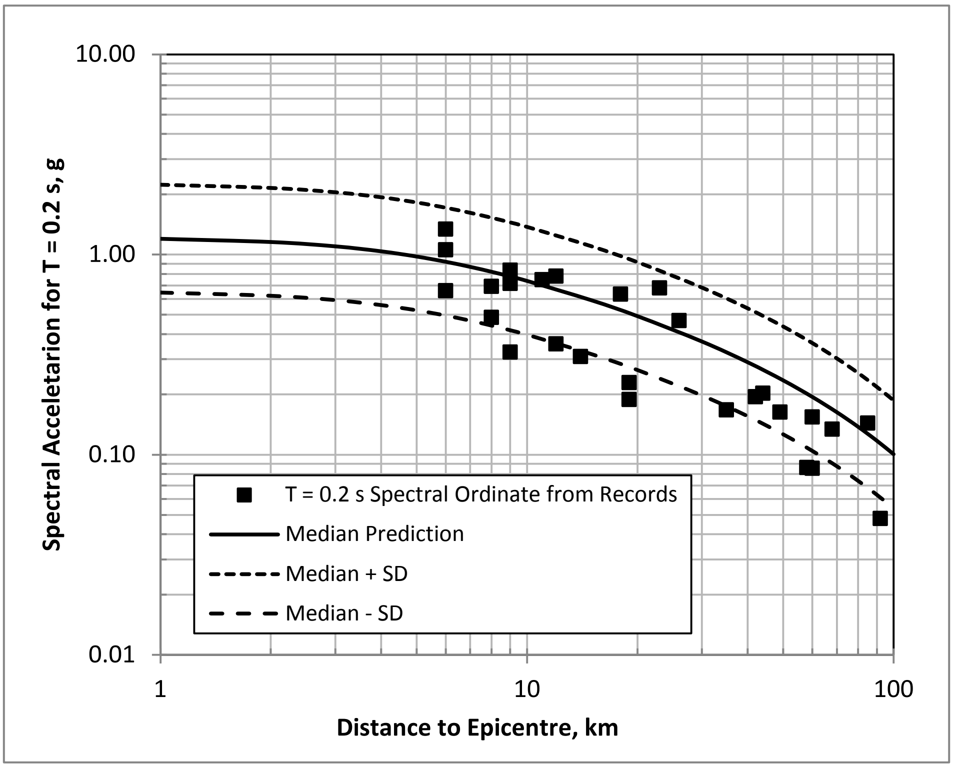


Figure A6. Christchurch Earthquake. Comparison of recorded PGA's with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

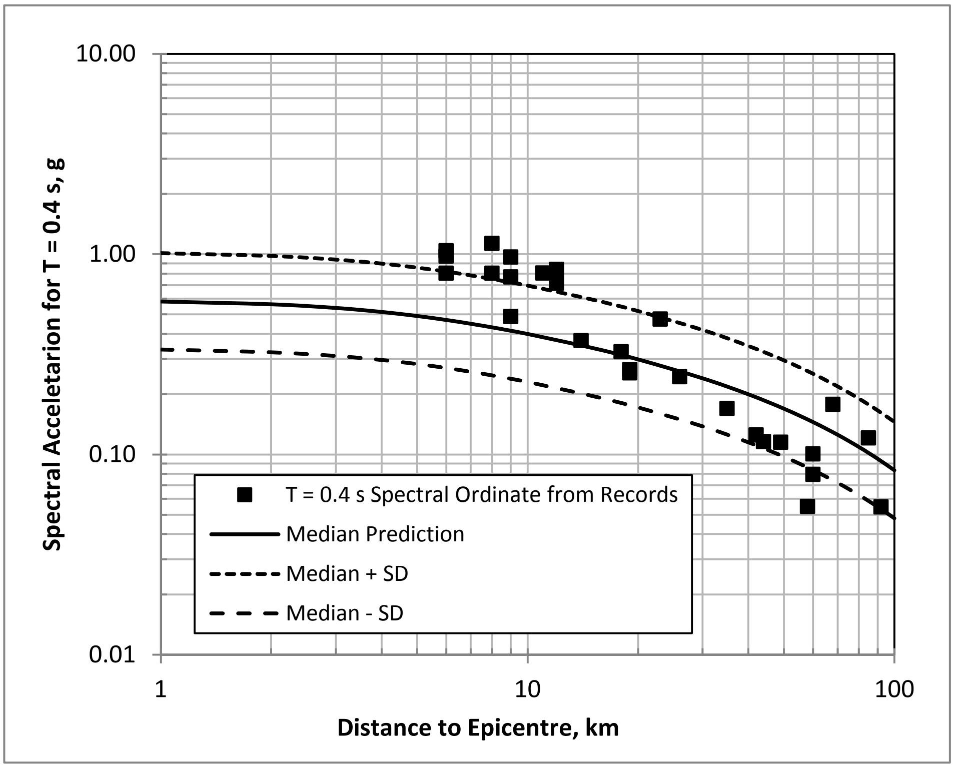


Figure A7. Christchurch Earthquake. Comparison of recorded PGA's with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

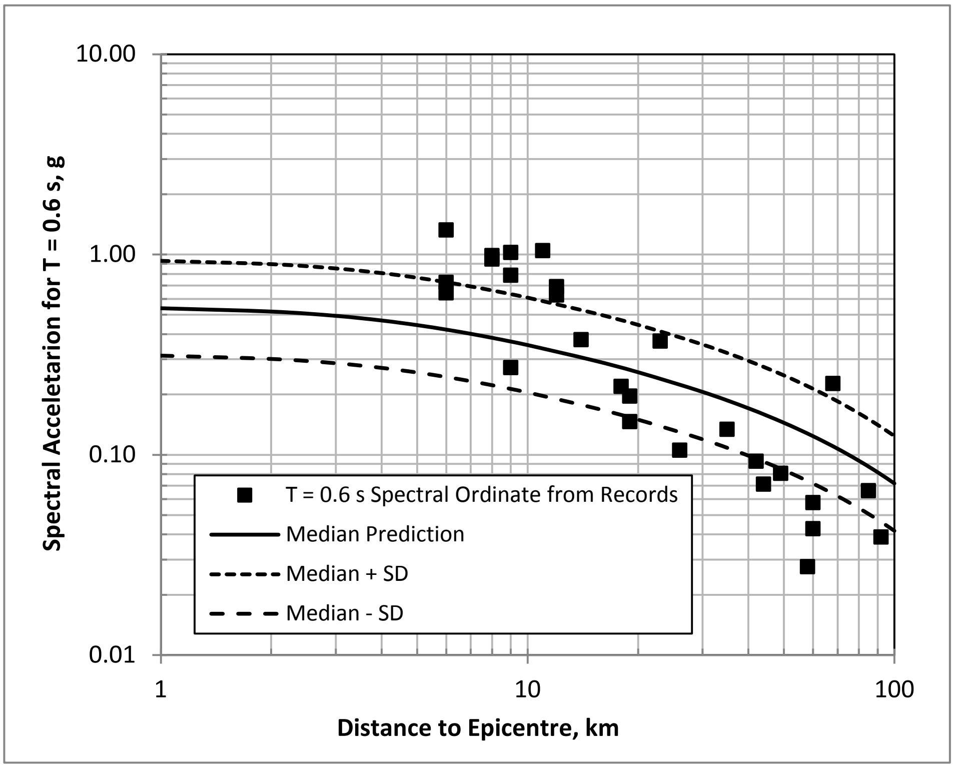


Figure A8. Christchurch Earthquake. Comparison of recorded PGA's with McVerry *et al* (2006) attenuation function. Mean of two horizontal components. Soil Site Categories D and E.

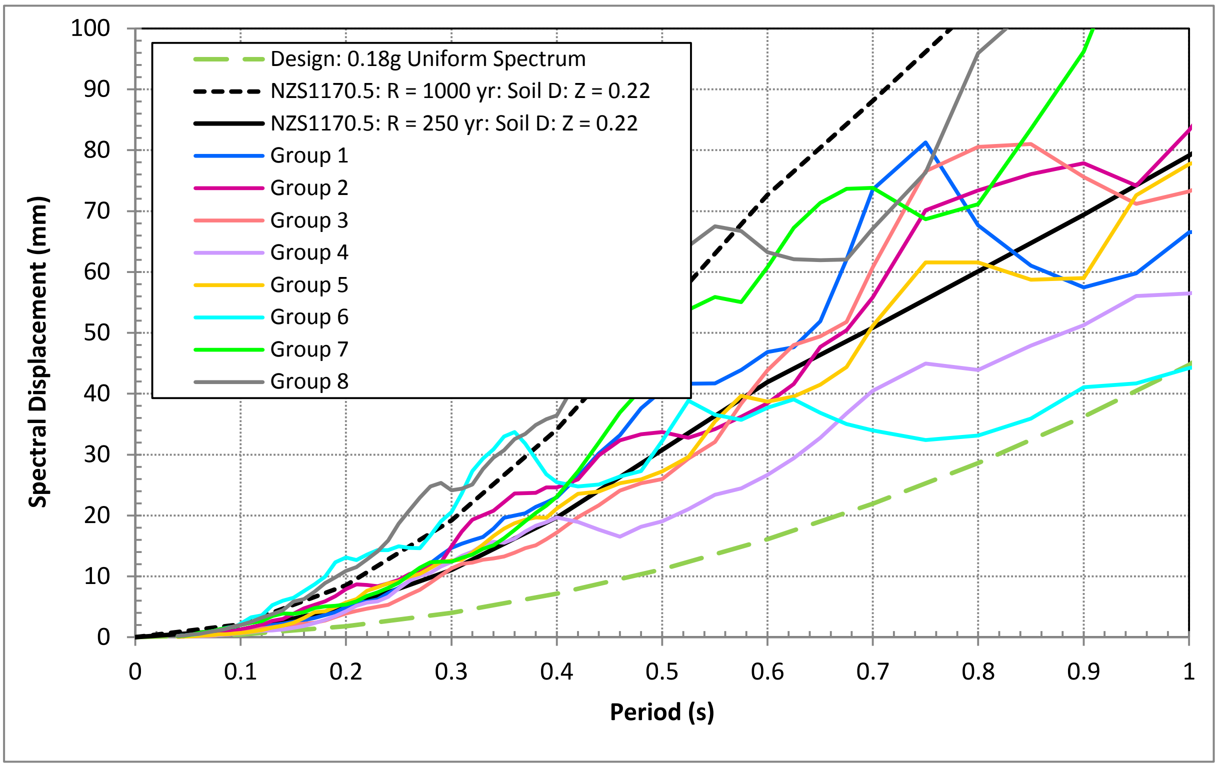


Figure A9. Darfield Earthquake. Mean spectral displacements from the two horizontal components of the two closest SMA's in each bridge group.

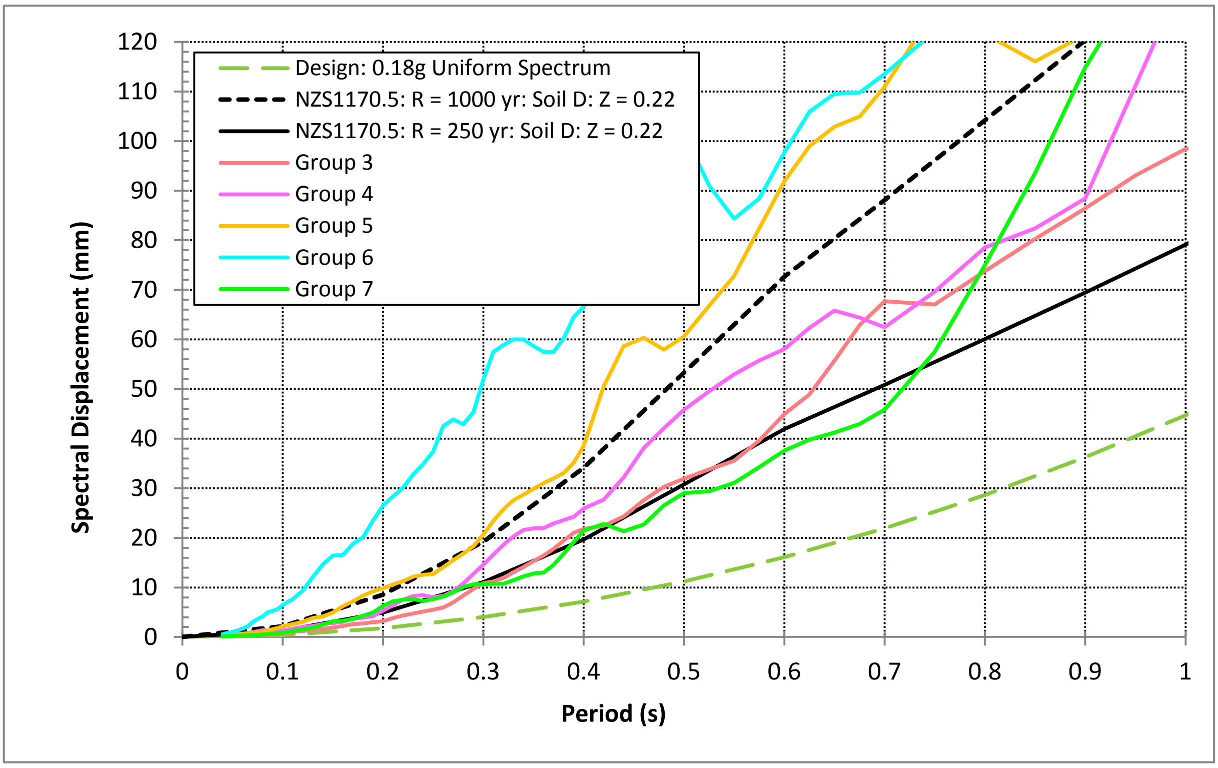


Figure A10. Christchurch Earthquake. Mean spectral displacements from the two horizontal components of the two closest SMA's in each bridge group.

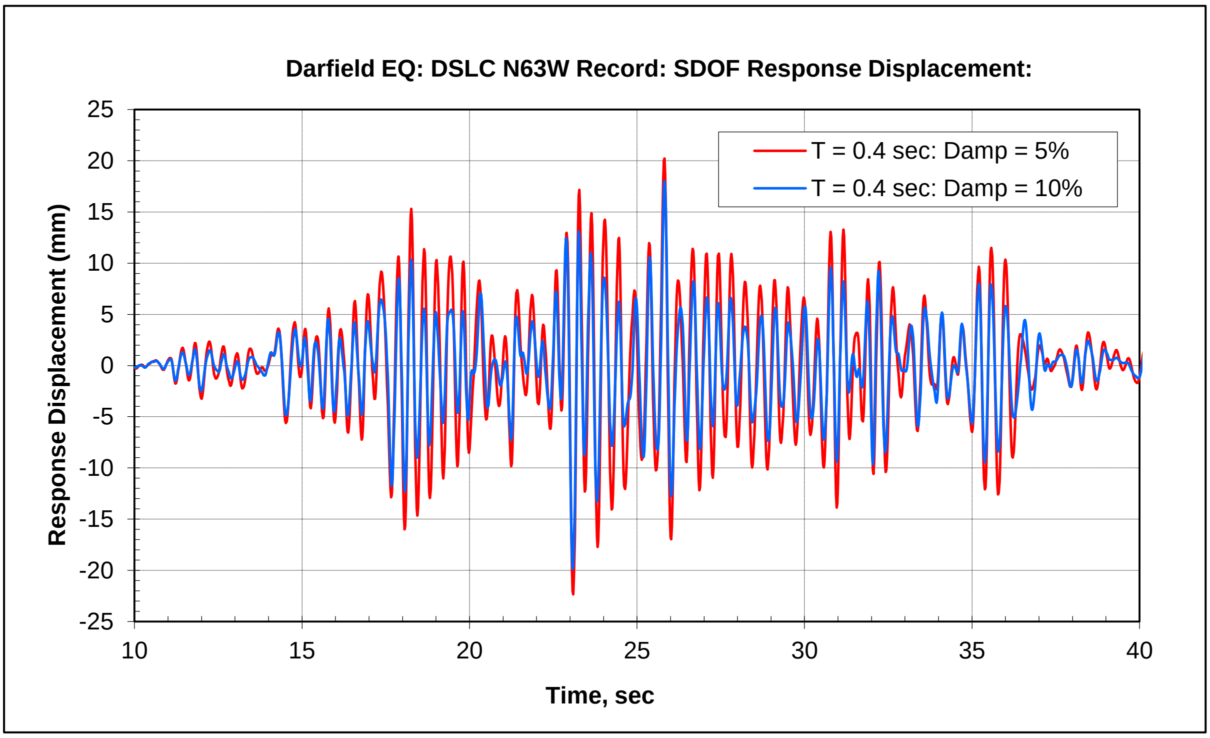


Figure A11. Displacement response of elastic SDOF system to DSLC N63W accelerogram.

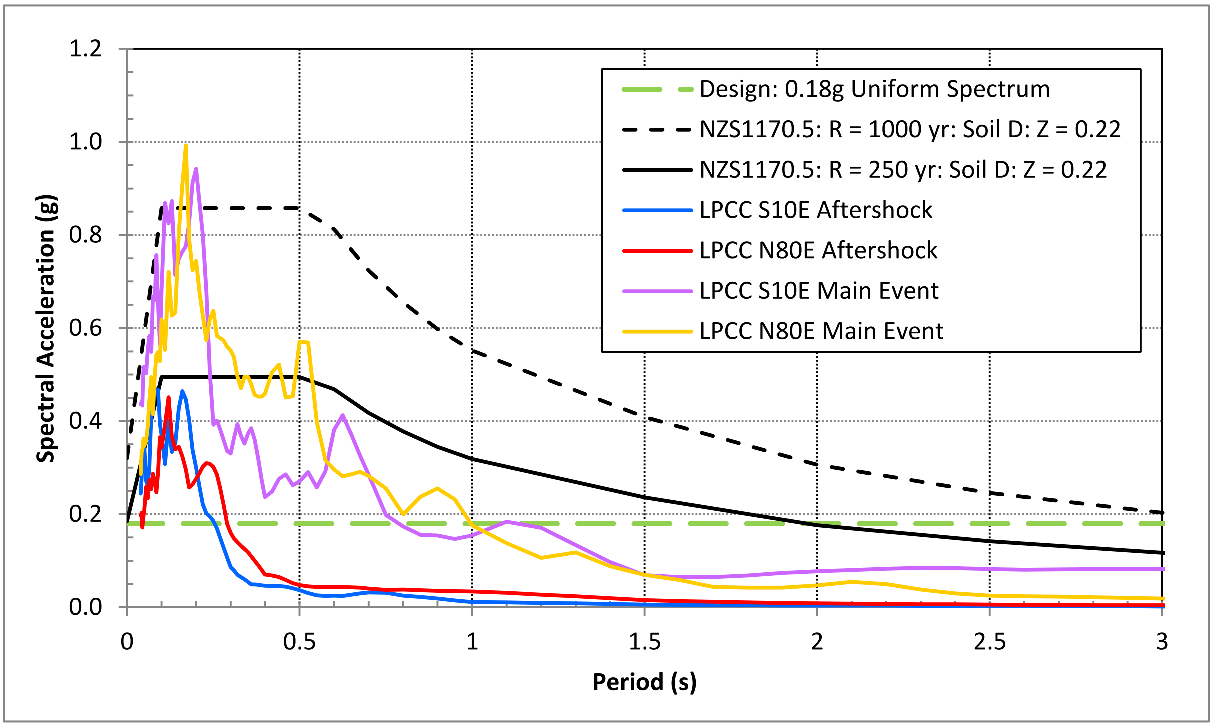


Figure A12. Comparison of acceleration spectra at LPCC from main Darfield event & aftershock No 20100907_194957 (7-Sep-10, 19 hr, 49.57 min: M = 5.02, Depth 7 km).

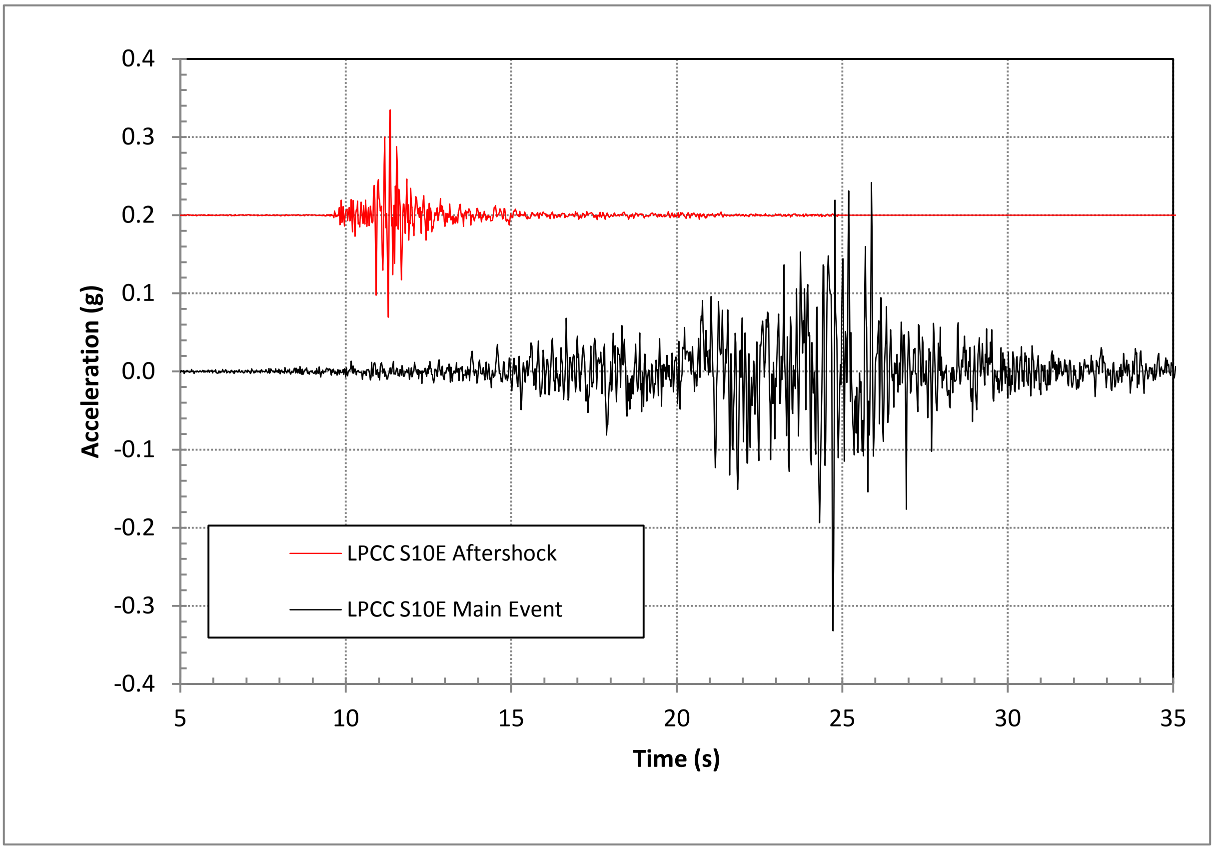


Figure A13. Comparison of acceleration time histories at LPCC from main Darfield event & aftershock No 20100907_194957 (7-Sep-10, 19 hr, 49.57 min: M = 5.02, Depth 7 km).

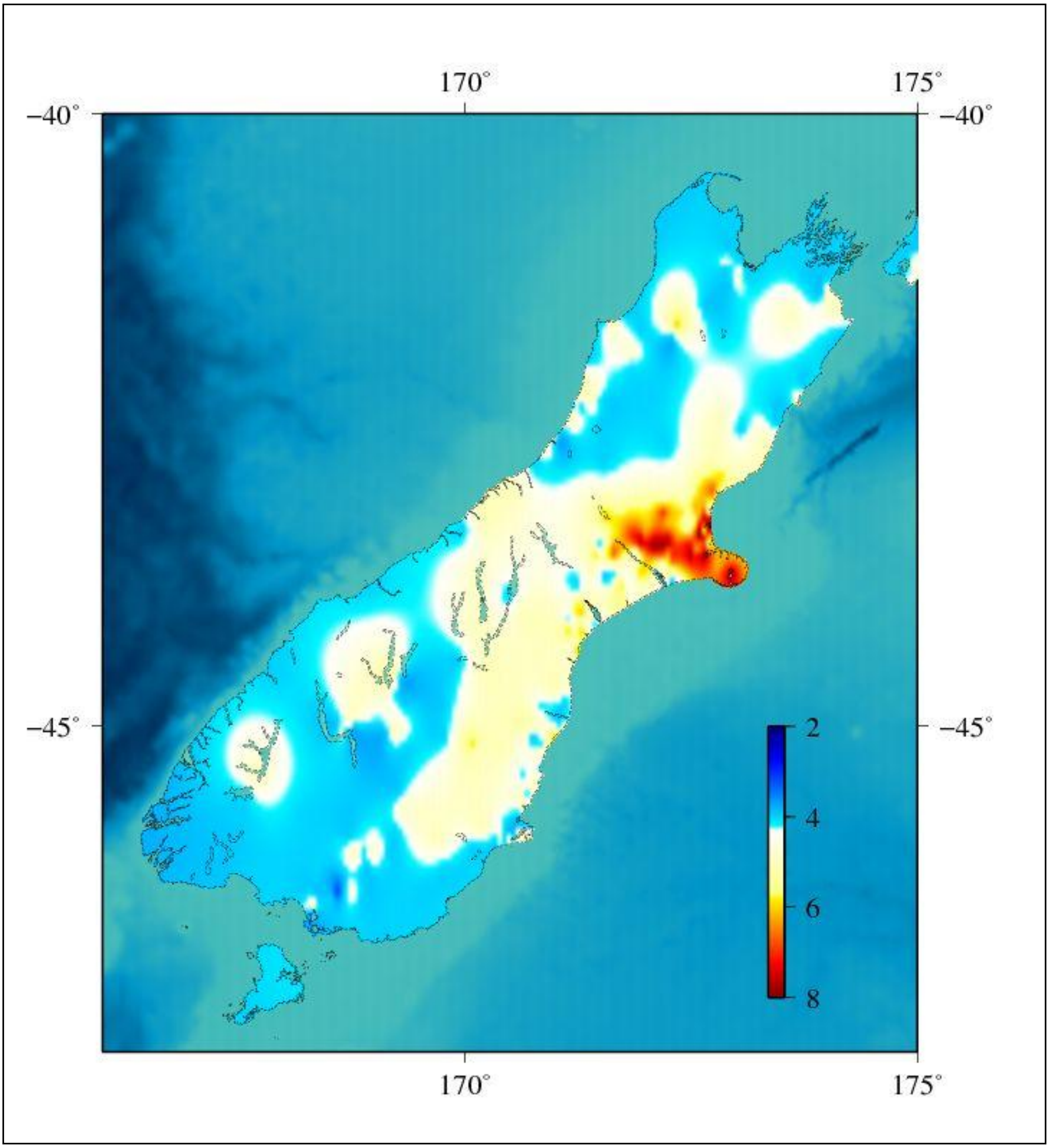


Figure A14. South Island MMI's for M = 7.1, Darfield Earthquake. From GeoNet.

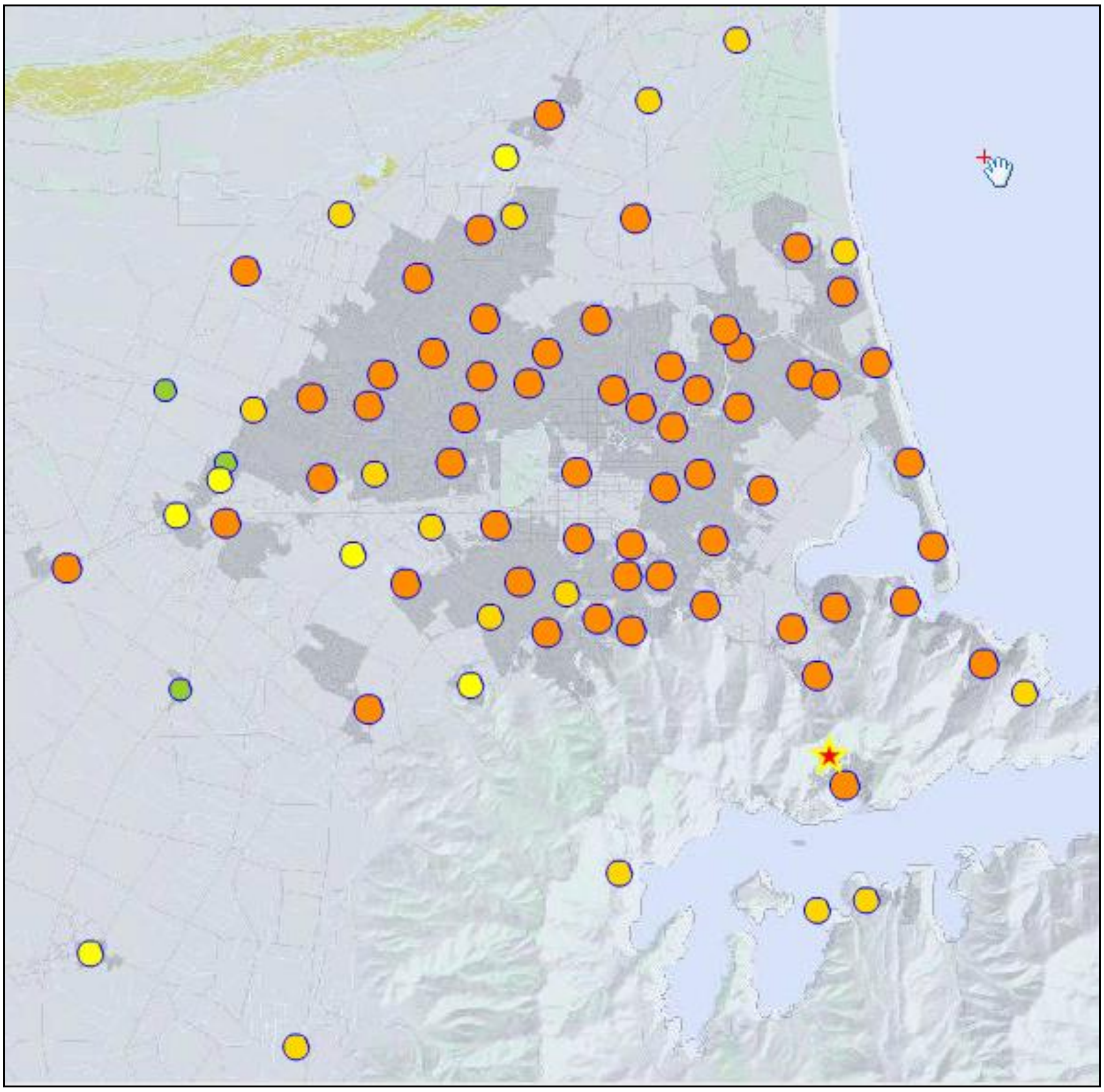


Figure A15. MMI's for M = 6.3, Christchurch Earthquake. Orange, dark yellow, yellow and green indicate MMI's of VIII, VII, VI and V respectively. From GeoNet.

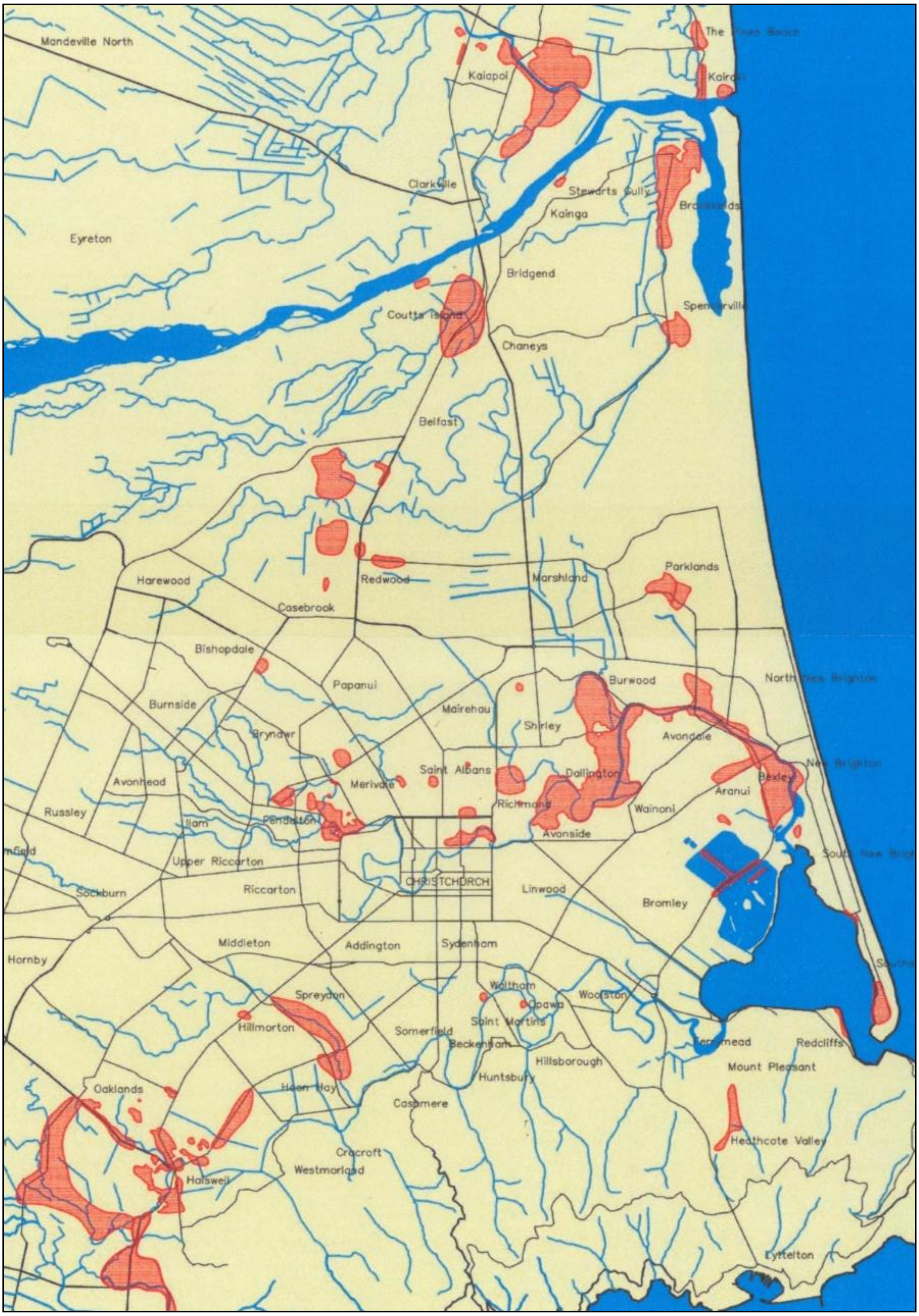


Figure A16. Darfield Earthquake. Areas of liquefaction. From Tonkin & Taylor (2010).

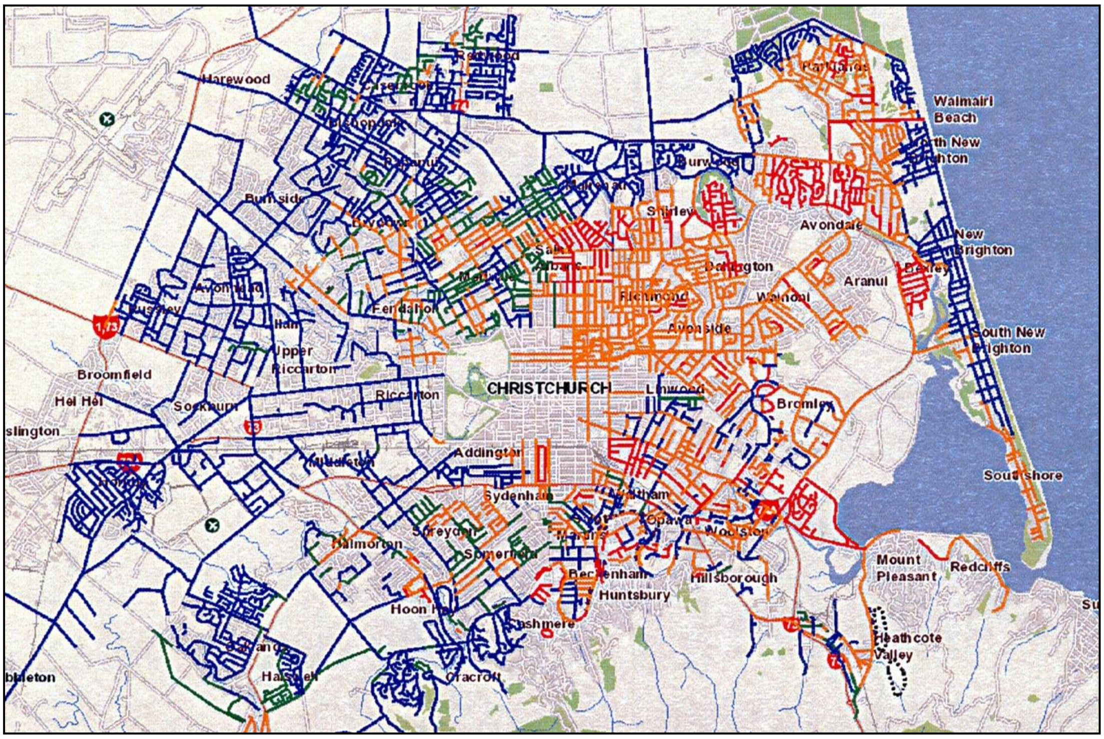


Figure A17. Christchurch Earthquake. Areas of liquefaction. Red, orange, green and blue streets indicate: severe, moderate, some, and no visible liquefaction respectively. From EQC Website.

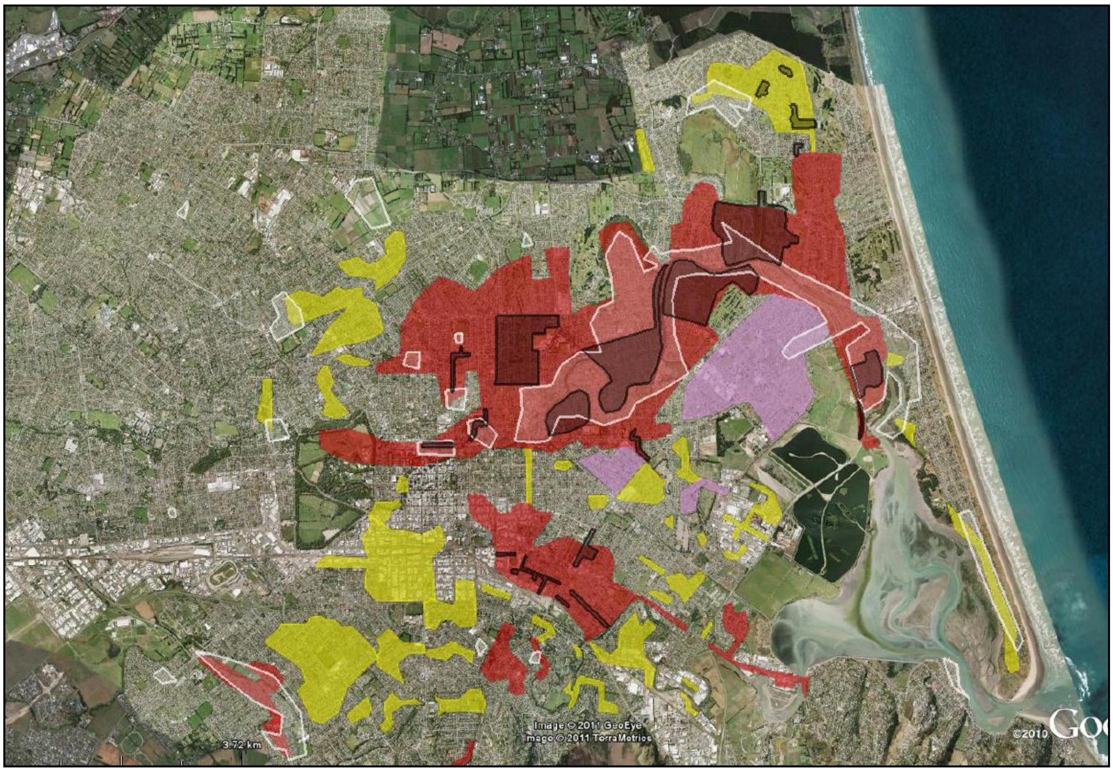


Figure A18. Preliminary liquefaction maps documenting areas of observed liquefaction in the 4 September 2010 (white contours), 22 February 2011 (red, yellow, magenta areas), and 13 June 2011 (black contours) earthquakes. Red = moderate to severe, yellow = low to moderate, magenta = visible on streets. (Cubrinovski and McCahon, 2011)

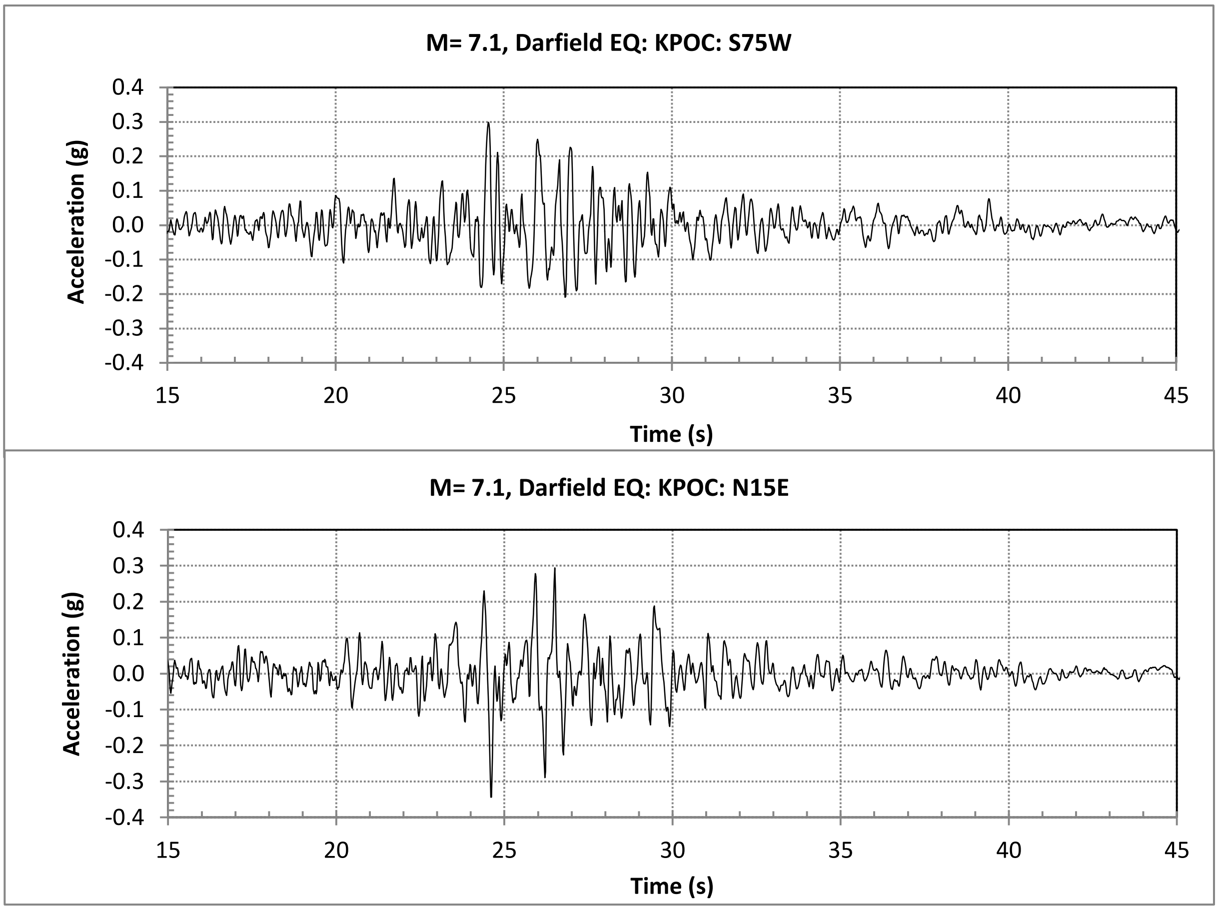


Figure A19. Acceleration time histories from nearest SMA to Group 1 bridges.

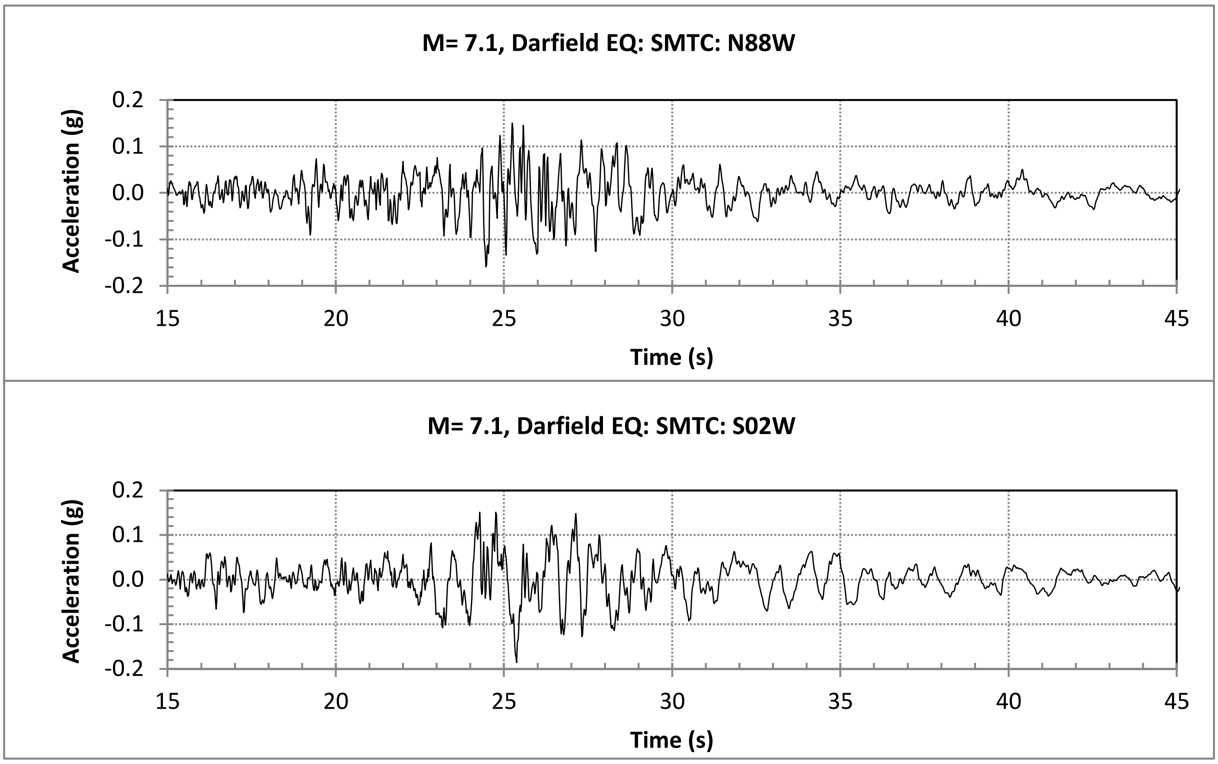


Figure A20. Acceleration time histories from second nearest SMA to Group 1 bridges

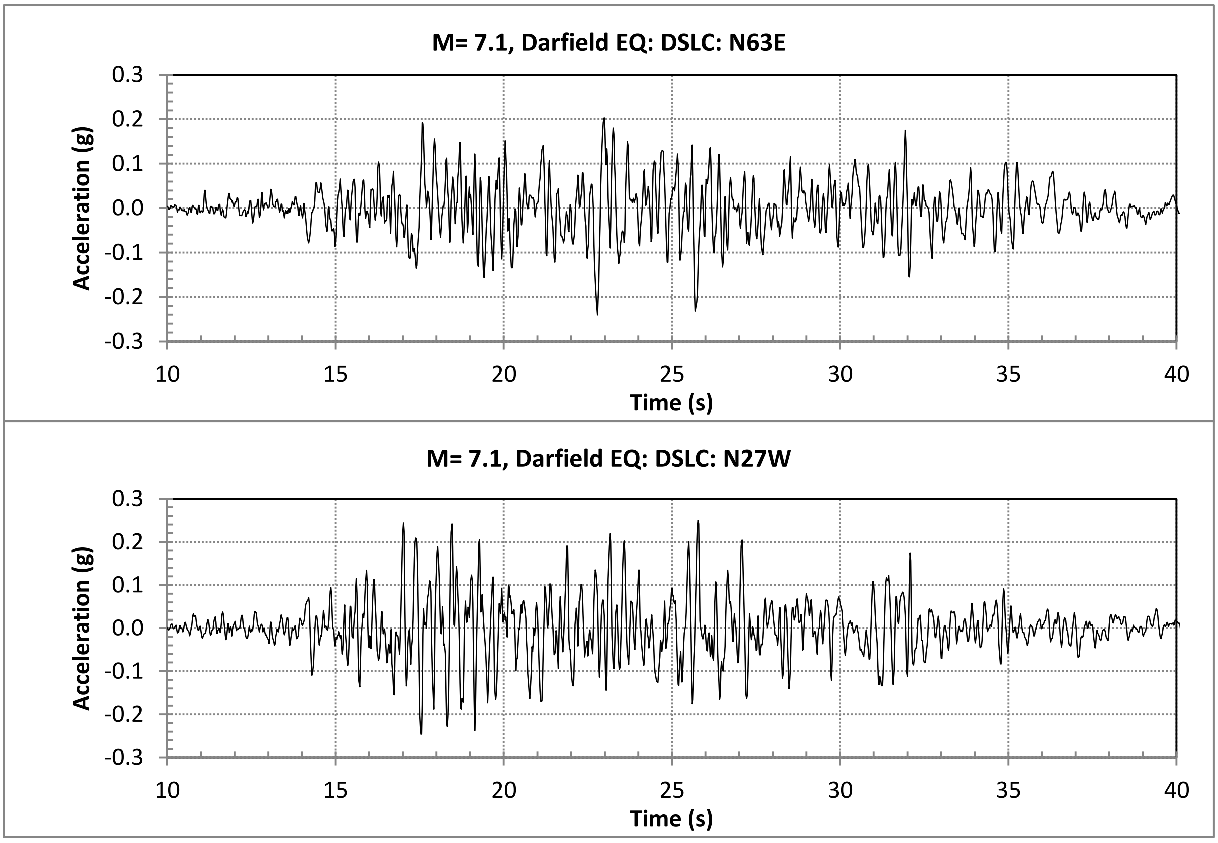


Figure A21. Acceleration time histories from nearest SMA to Group 2 bridge.

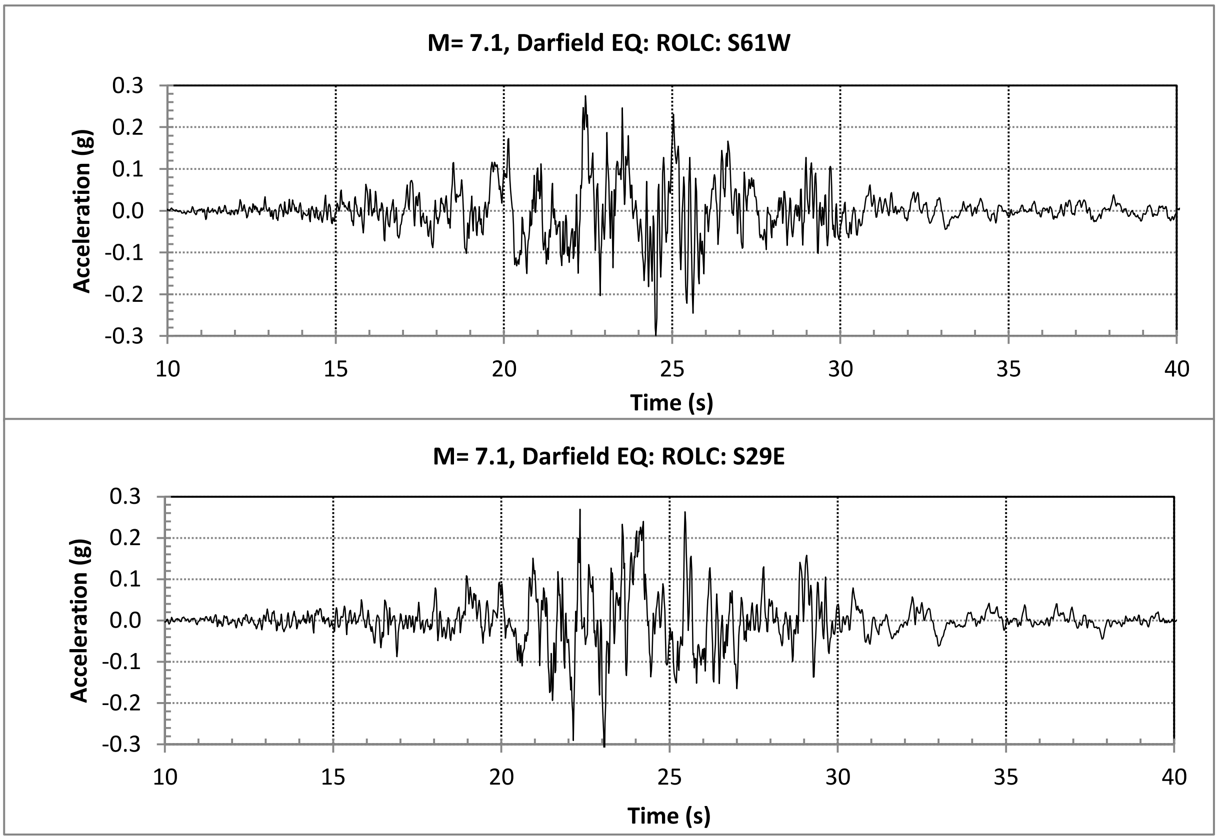


Figure A22. Acceleration time histories from second nearest SMA to Group 2 bridge.

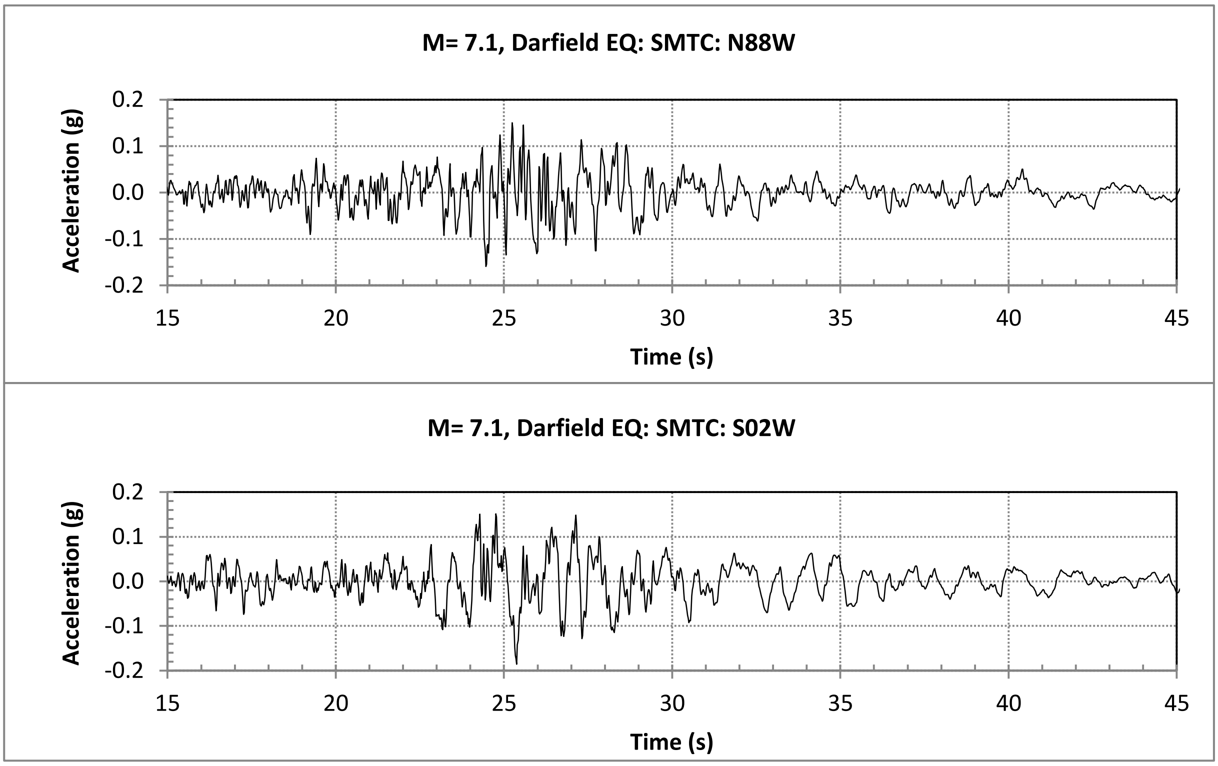


Figure A23. Acceleration time histories from nearest SMA to Group 3 bridges.

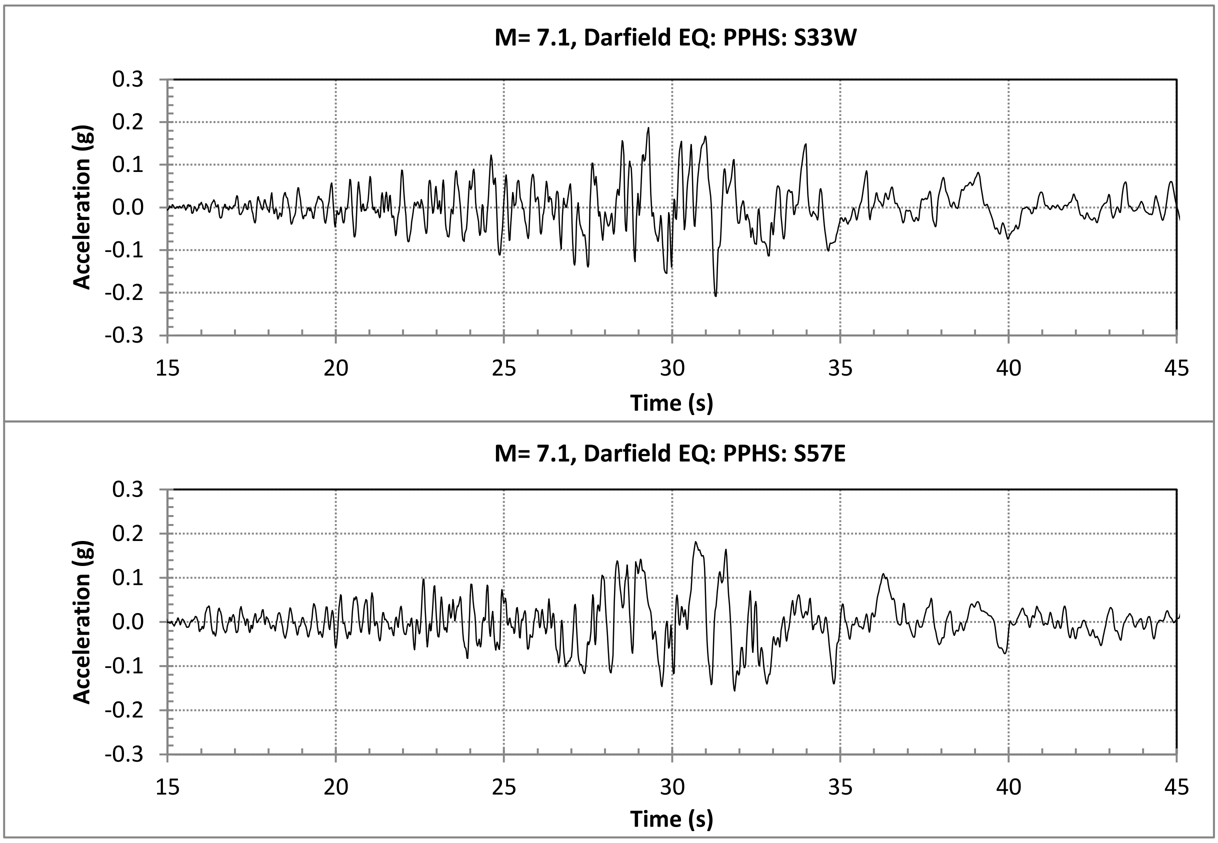


Figure A24. Acceleration time histories from second nearest SMA to Group 3 bridges.

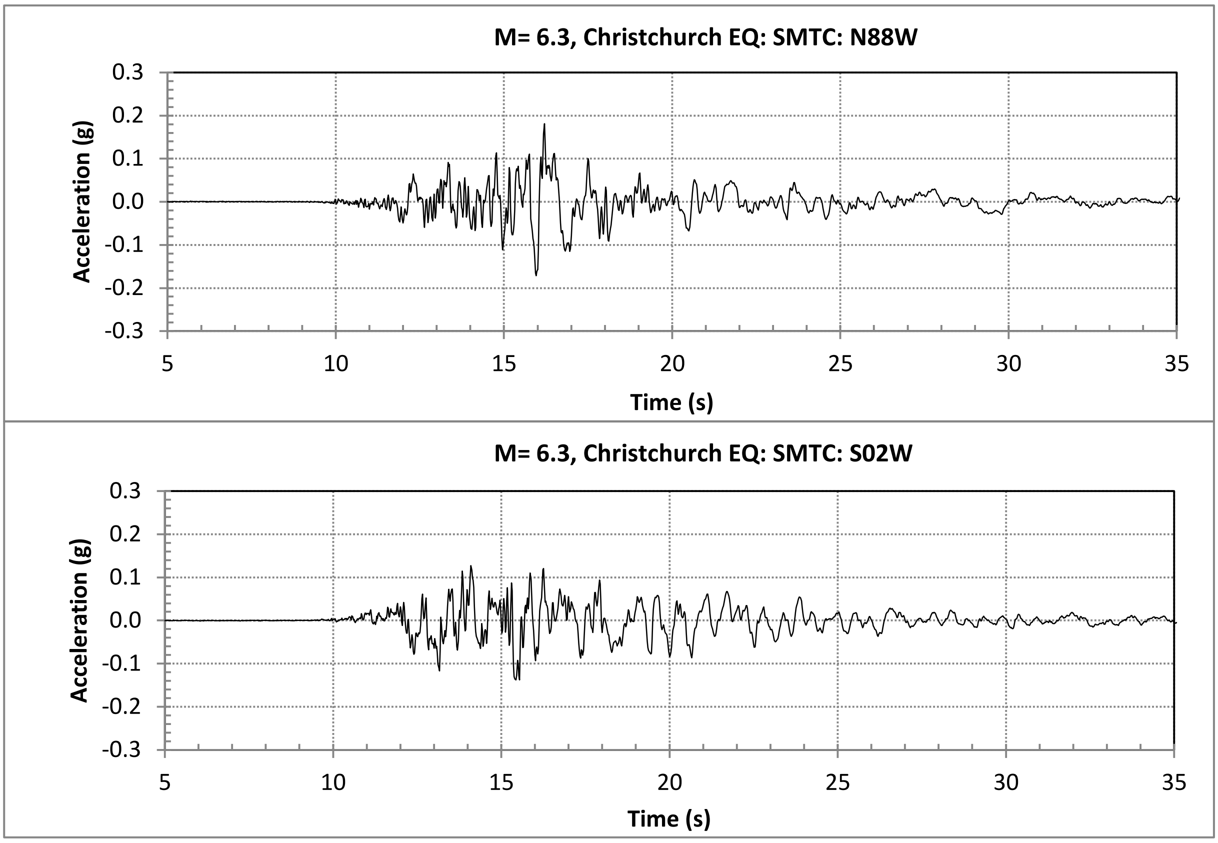


Figure A25. Acceleration time histories from nearest SMA to Group 3 bridges.

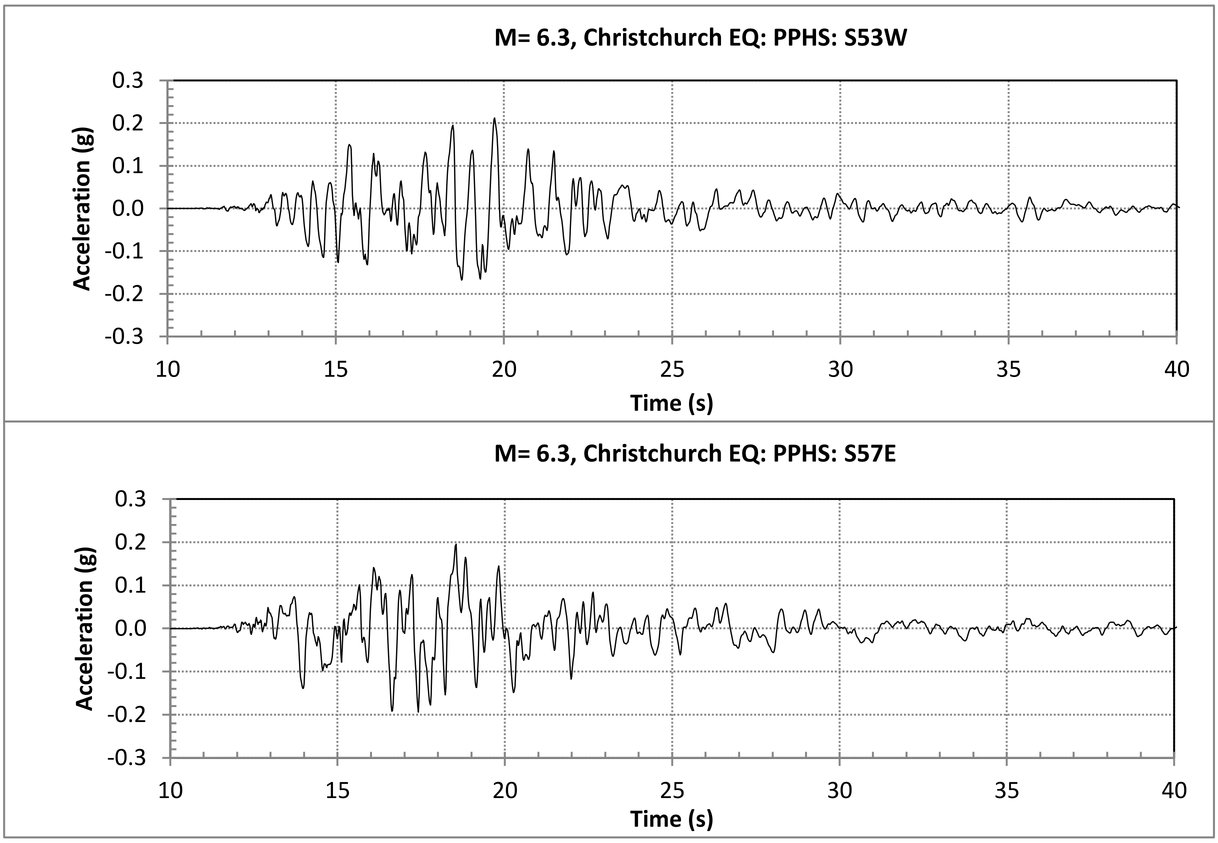


Figure A26. Acceleration time histories from second nearest SMA to Group 3 bridges.

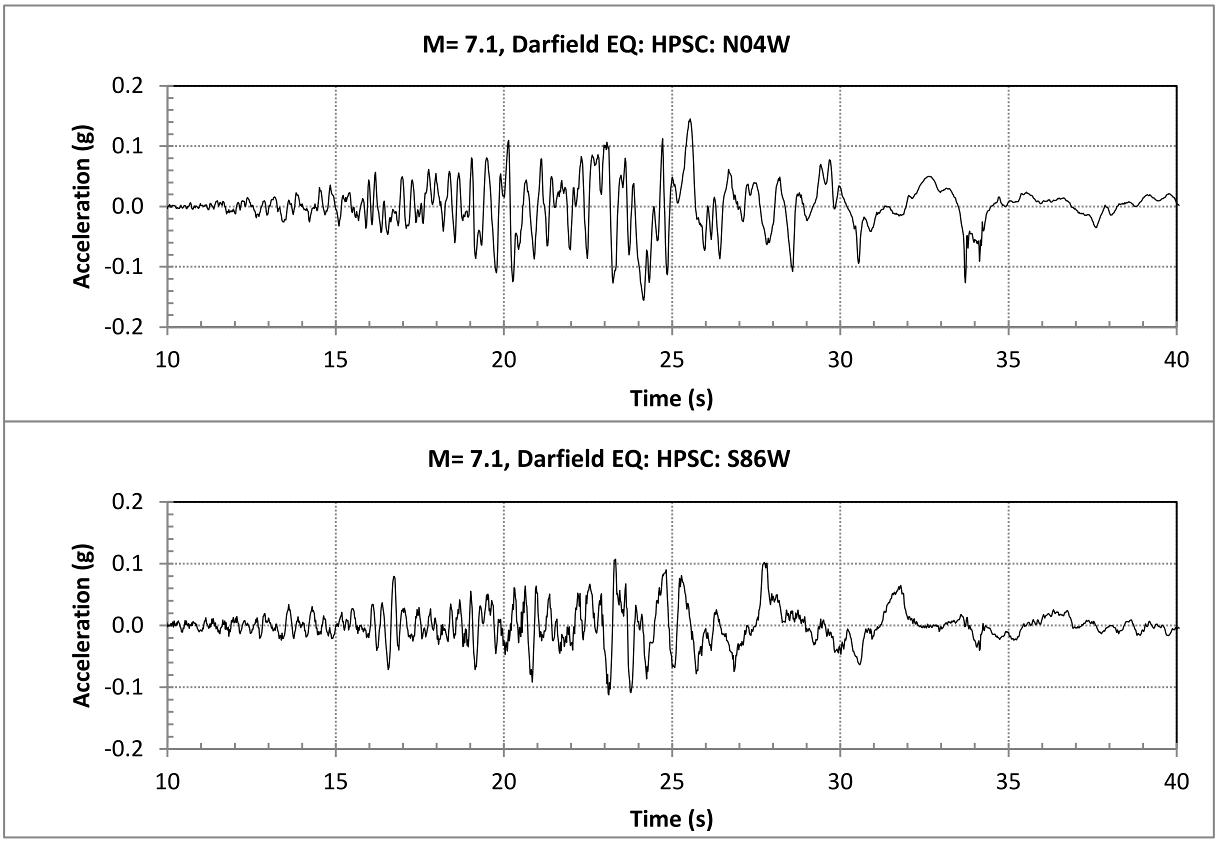


Figure A27. Acceleration time histories from nearest SMA to Group 4 bridges.

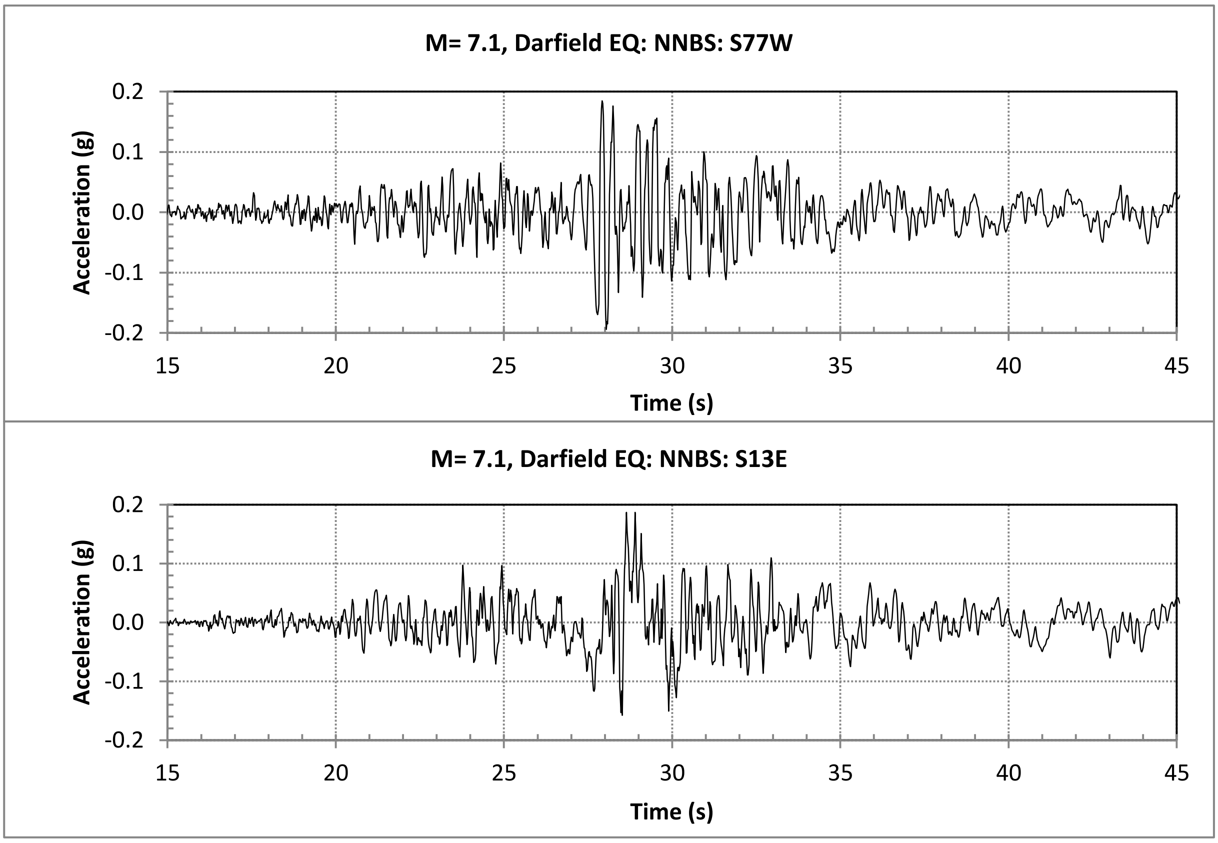


Figure A28. Acceleration time histories from nearest SMA to Group 4 bridges.

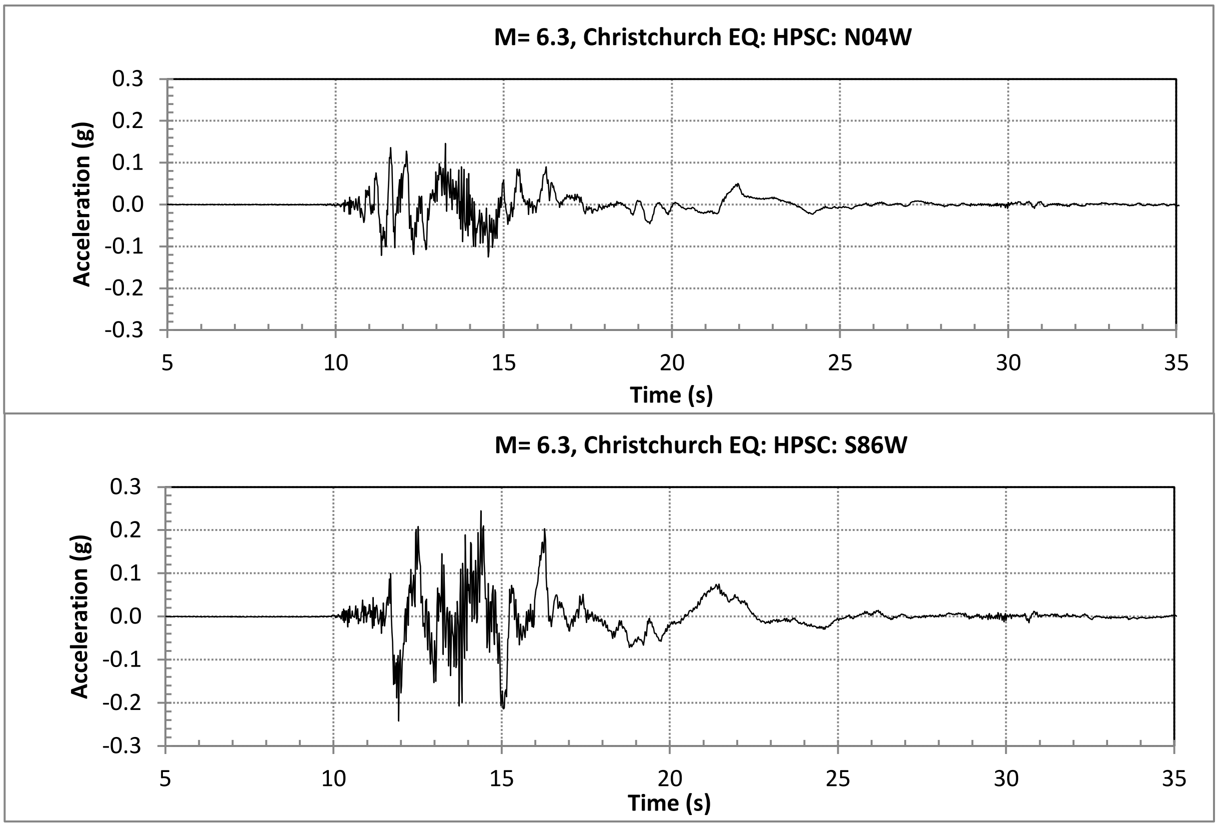


Figure A29. Acceleration time histories from nearest SMA to Group 4 bridges.

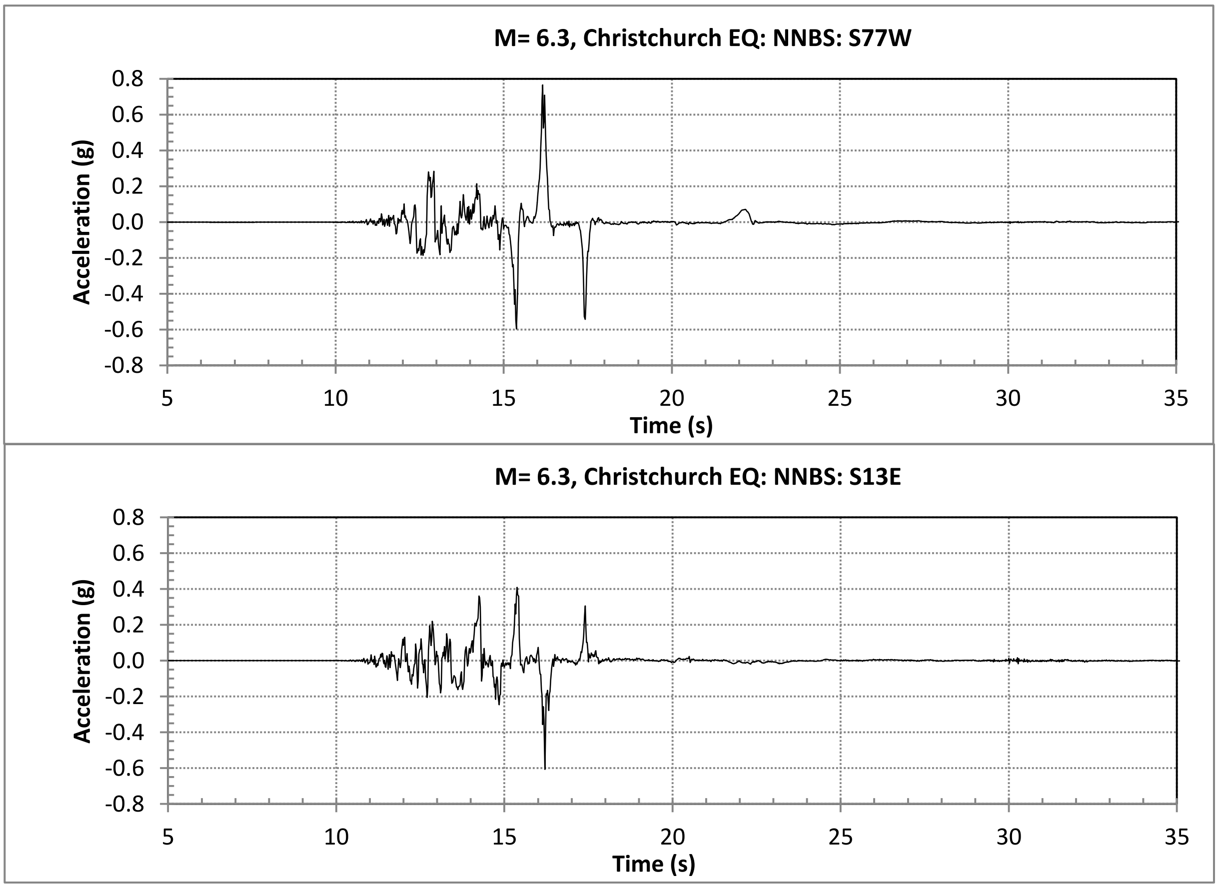


Figure A30. Acceleration time histories from second nearest SMA to Group 4 bridges.

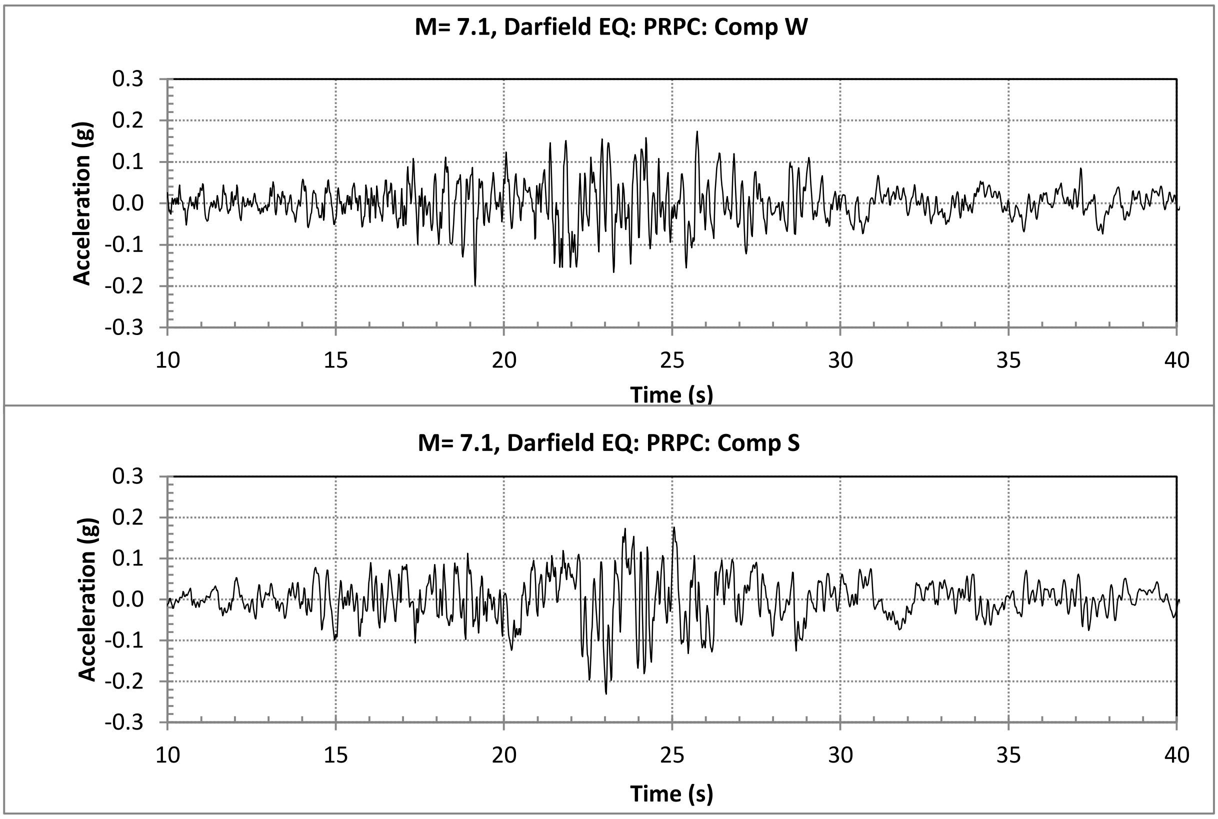


Figure A31. Acceleration time histories from nearest SMA to Group 5 bridges.

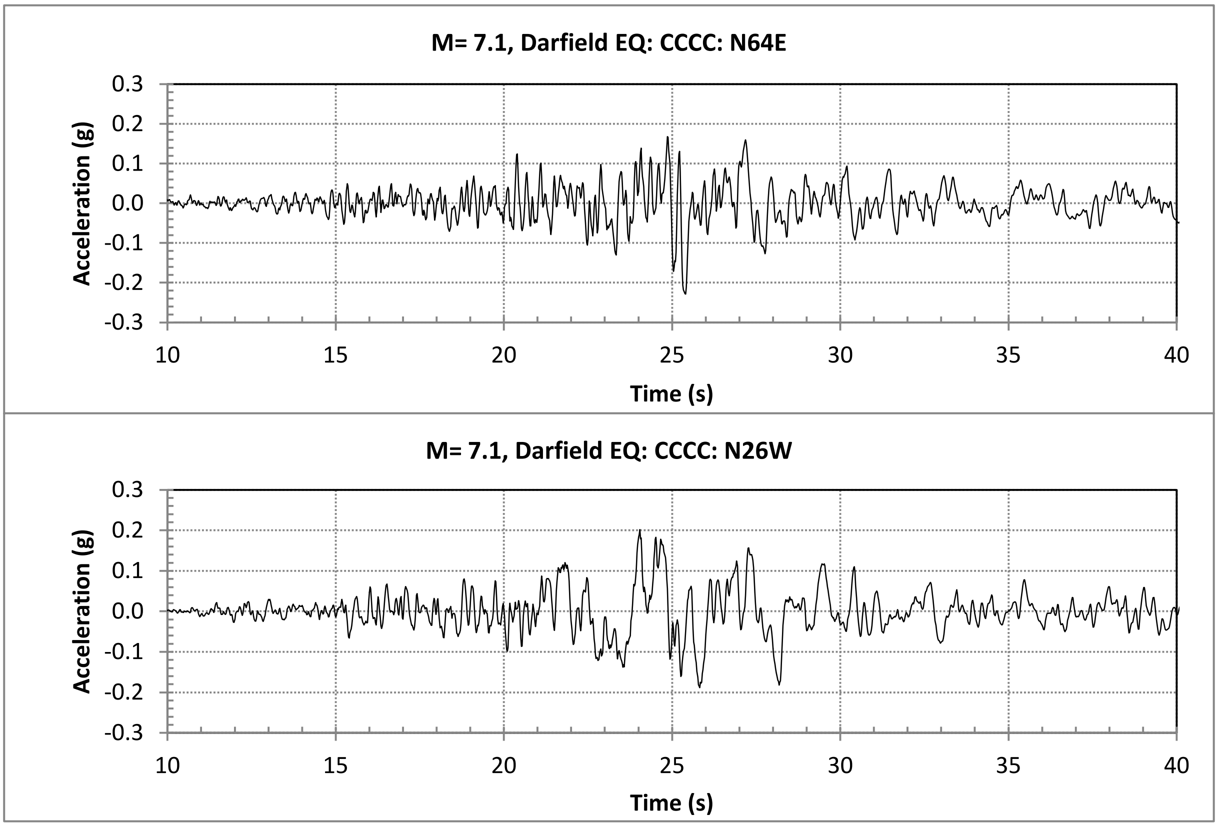


Figure A32. Acceleration time histories from second nearest SMA to Group 5 bridges.

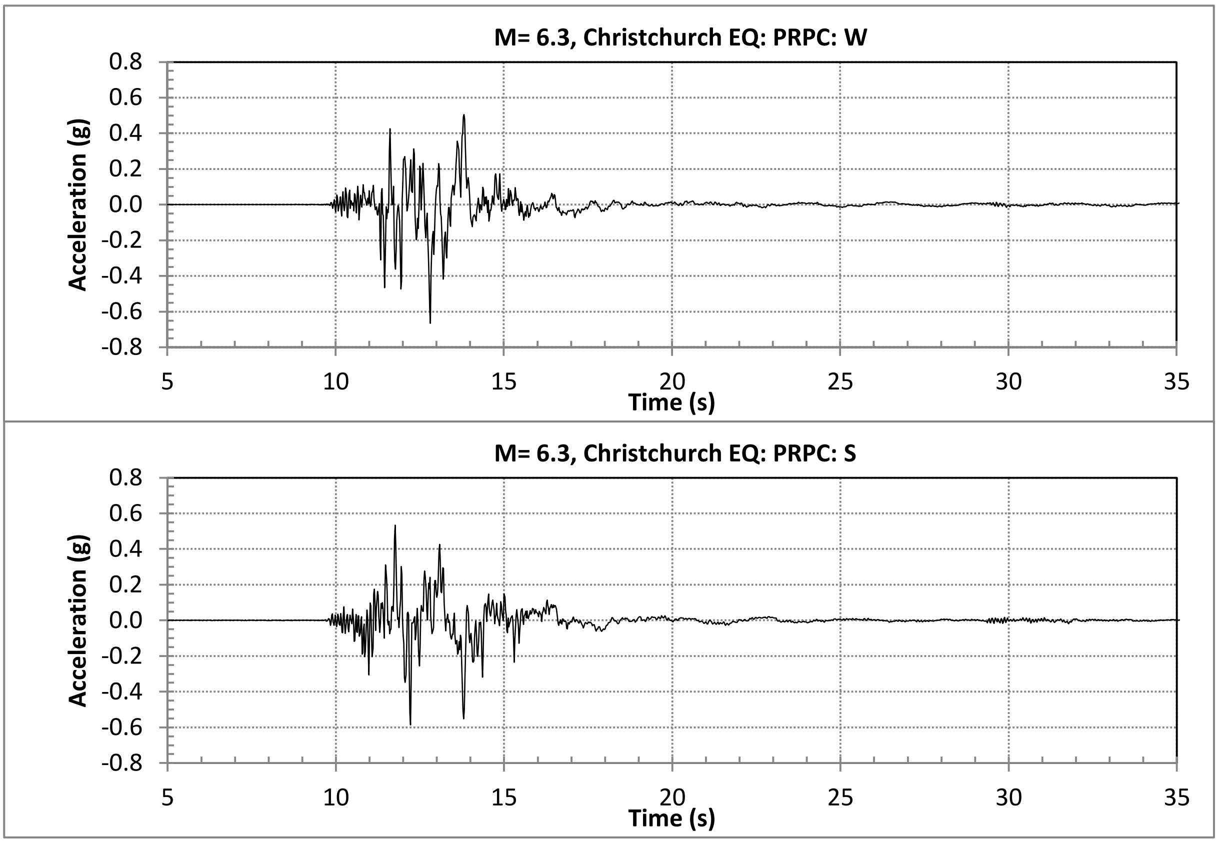


Figure A33. Acceleration time histories from SMA nearest to Group 5 bridges.

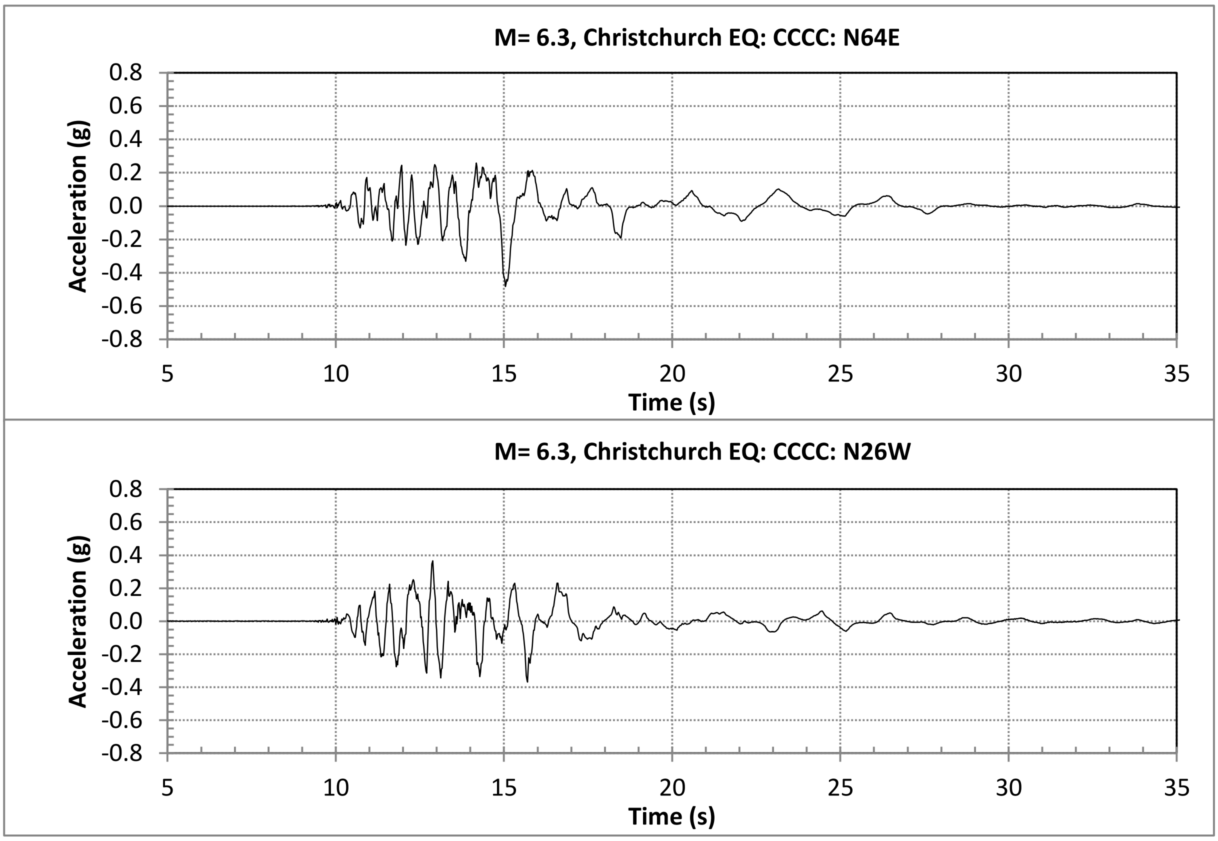


Figure A34. Acceleration time histories from SMA second nearest to Group 5 bridges.

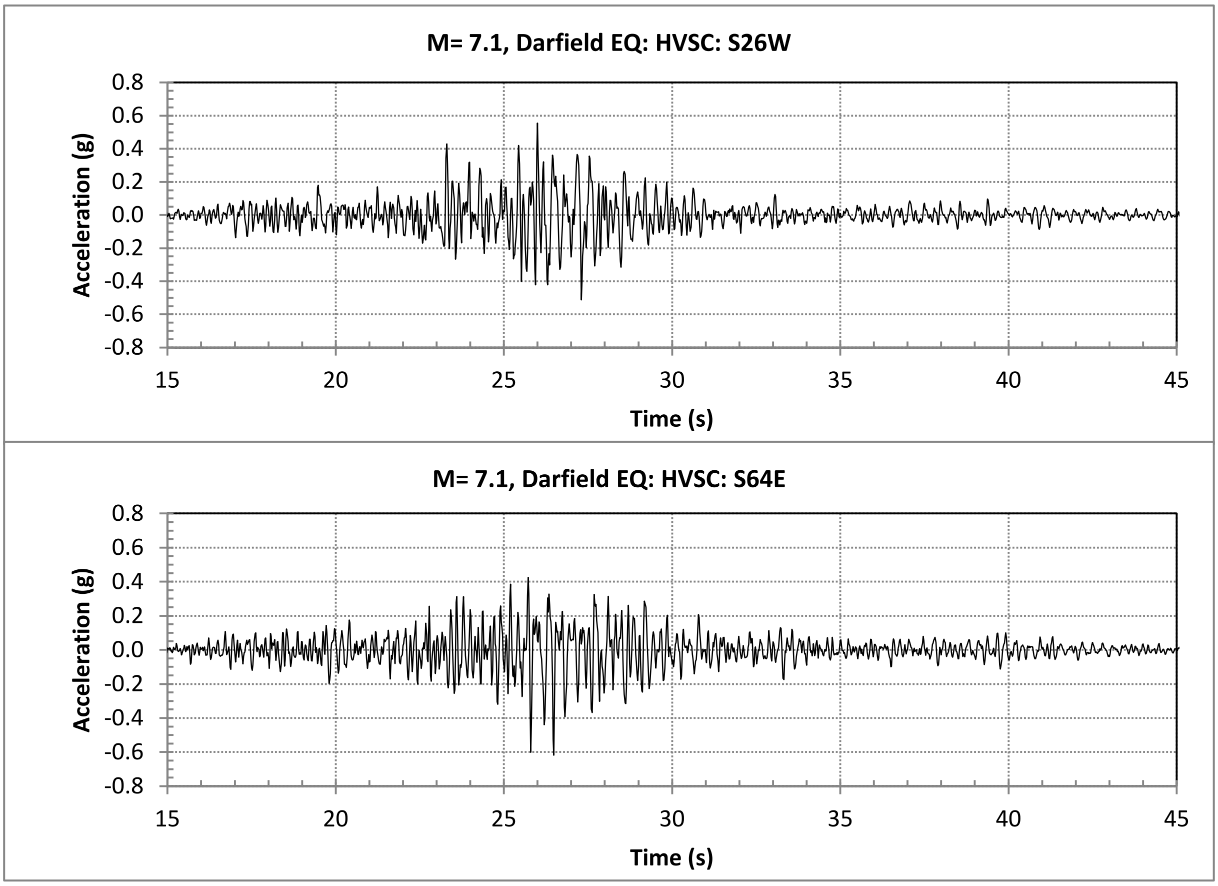


Figure A35. Acceleration time histories from SMA nearest to Group 6 bridges.

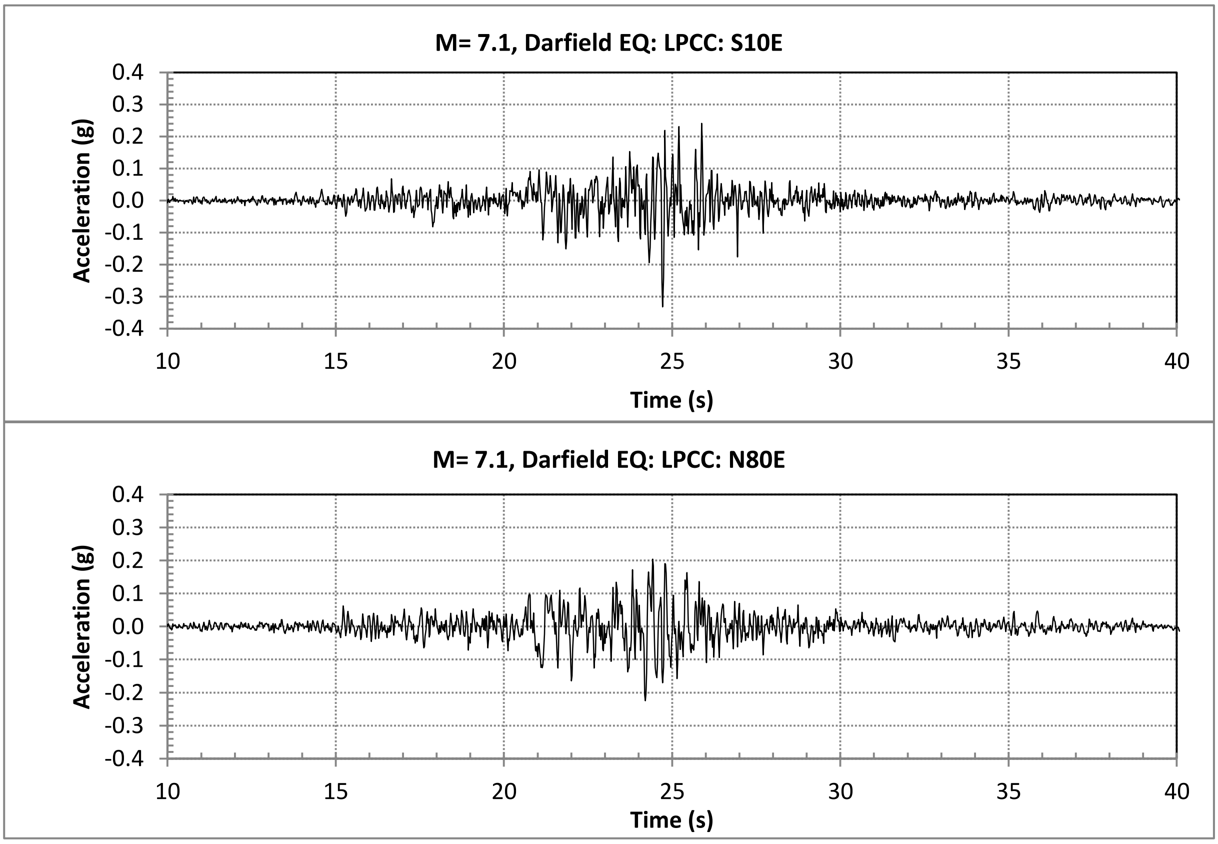


Figure A36. Acceleration time histories from SMA second nearest to Group 6 bridges.

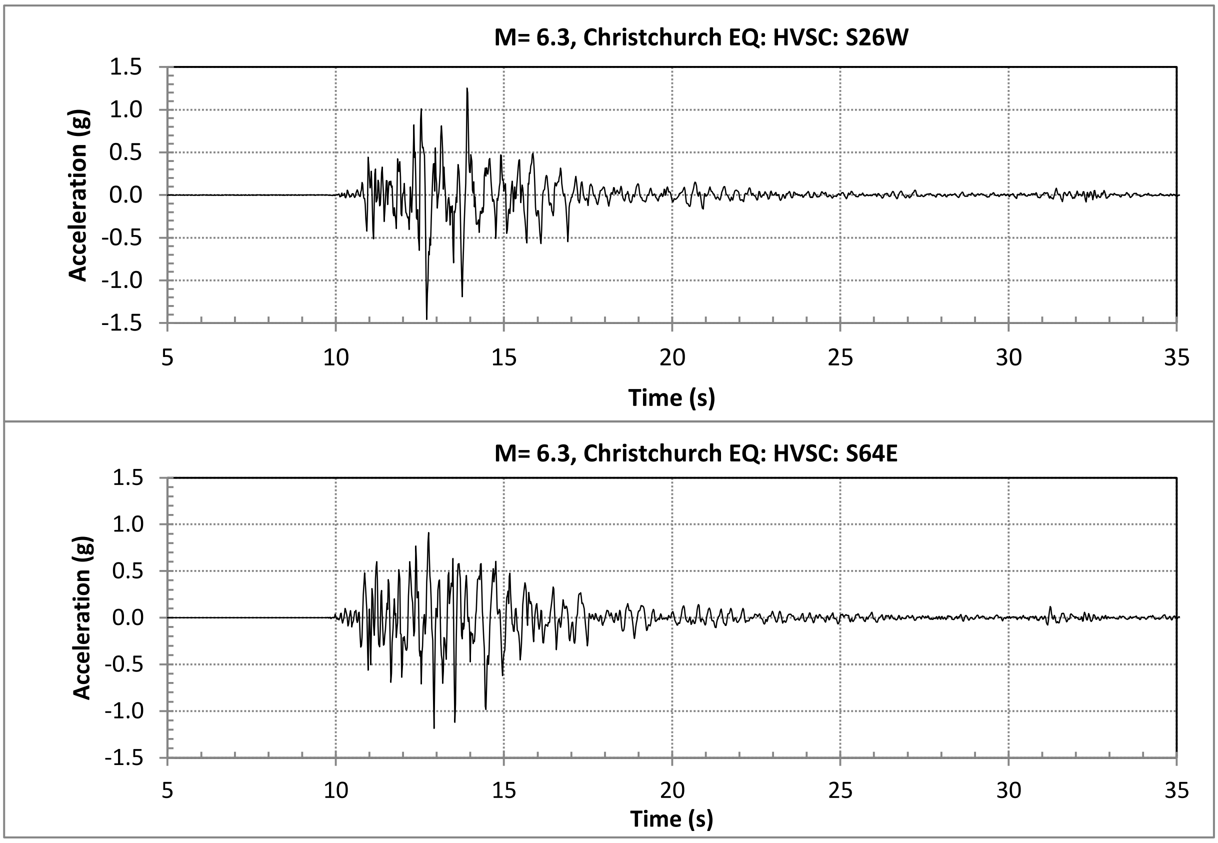


Figure A37. Acceleration time histories from nearest SMA to Group 6 bridges.

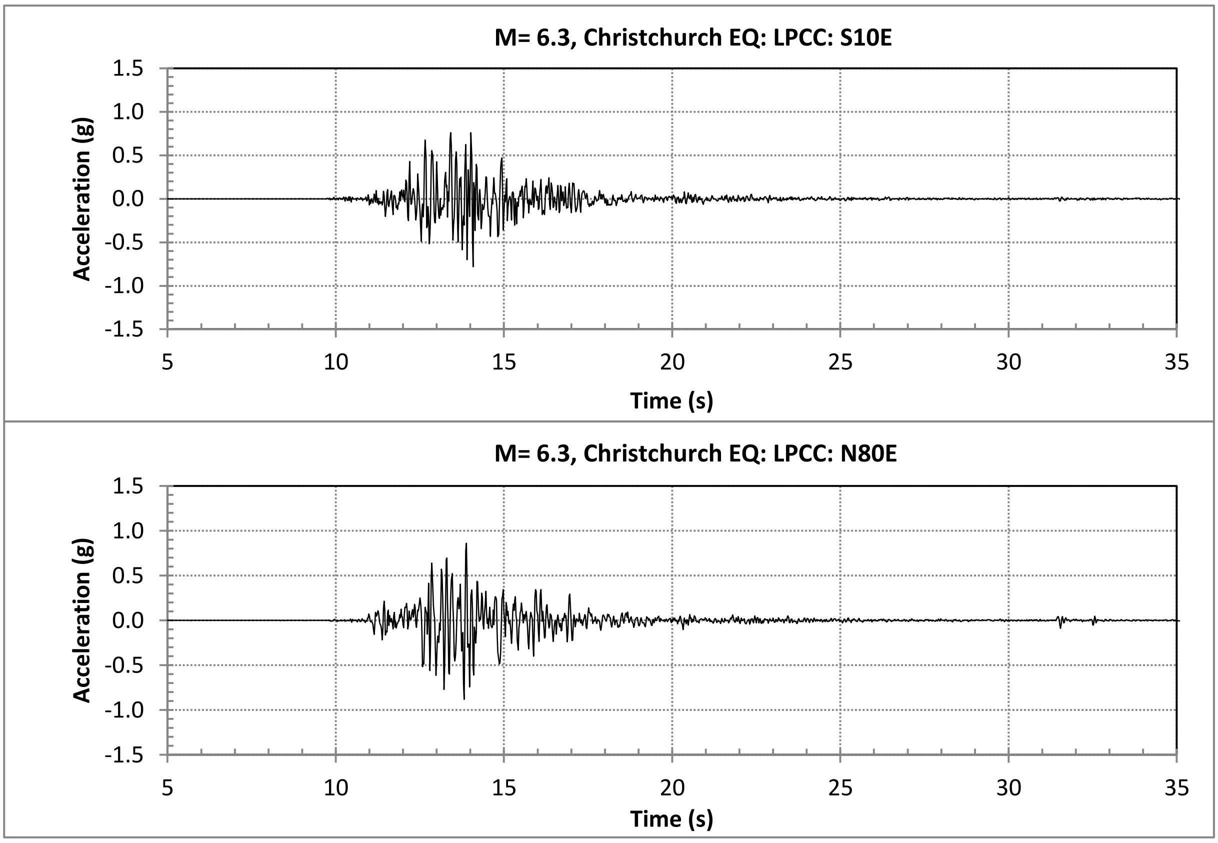


Figure A38. Acceleration time histories from second nearest SMA to Group 6 bridges.

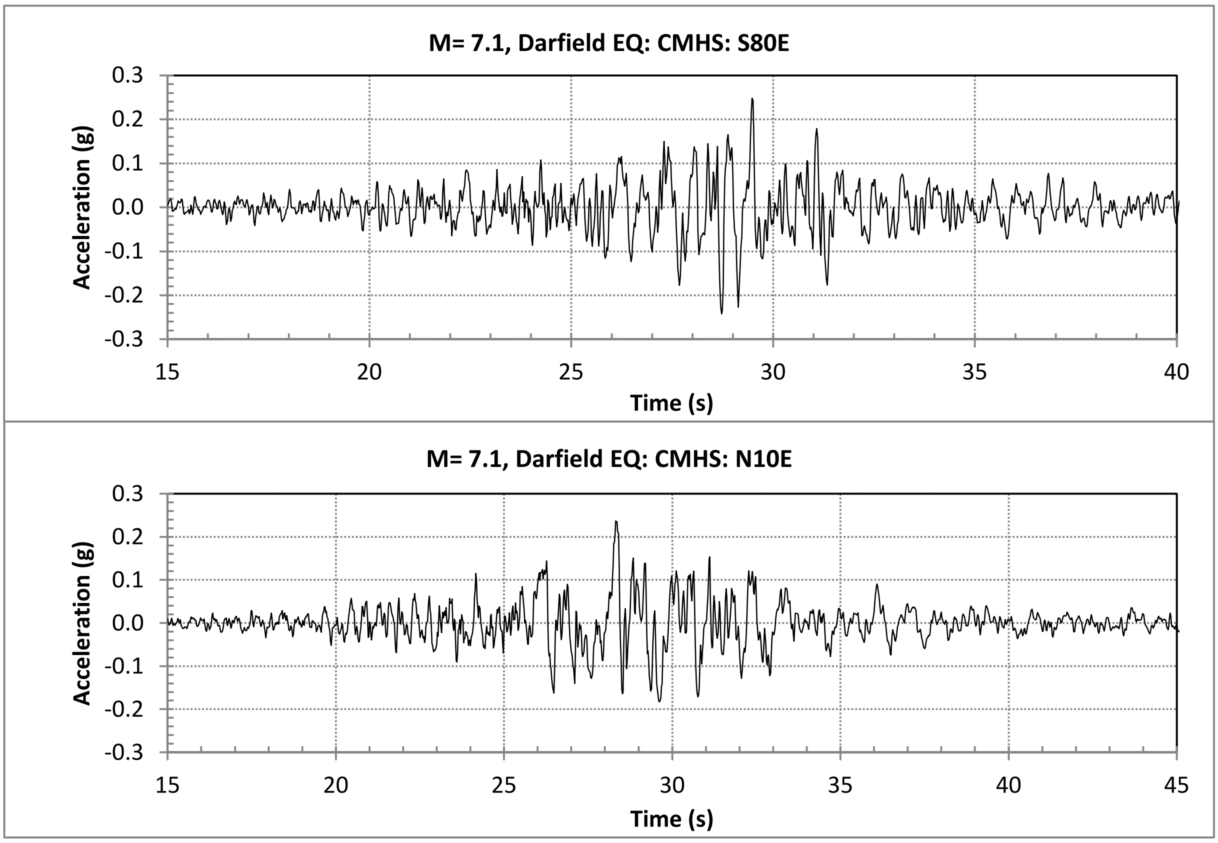


Figure A39. Acceleration time histories from nearest SMA to Group 7 bridge.

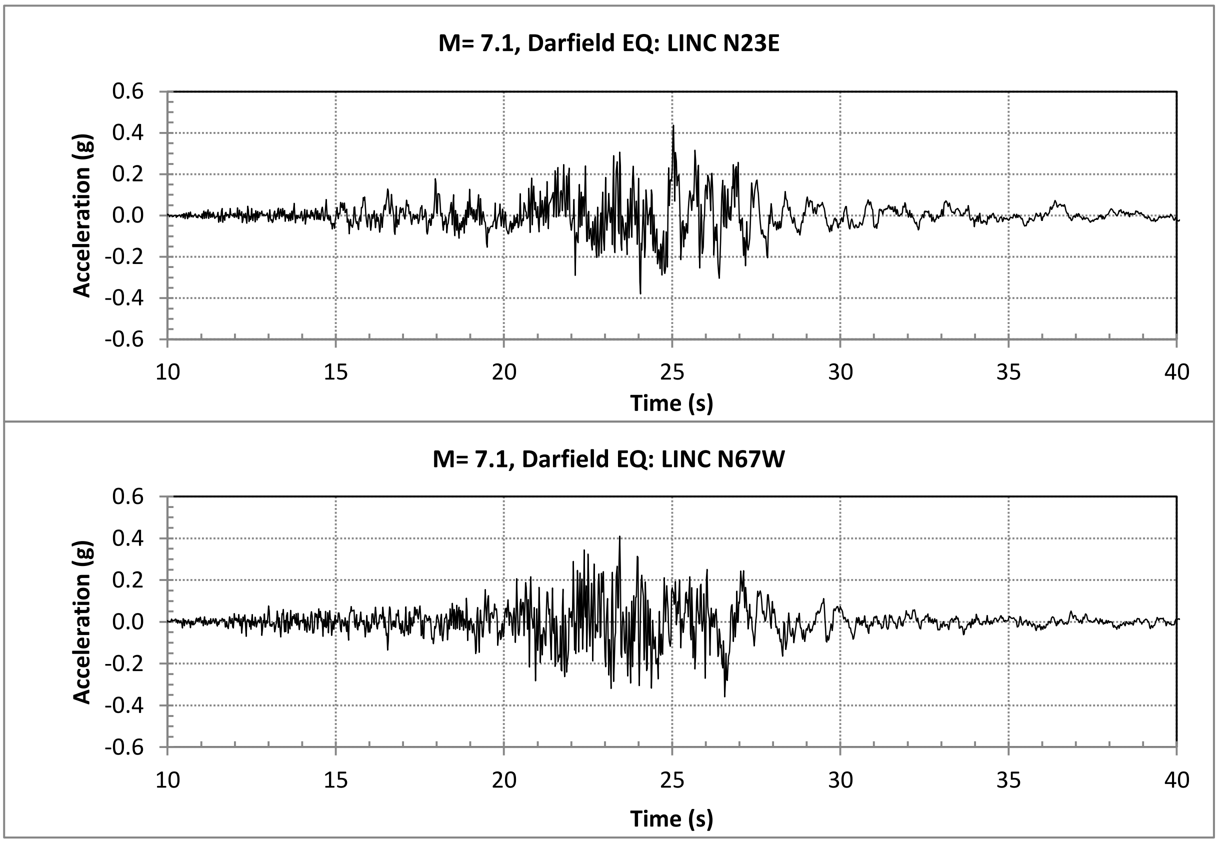


Figure A40. Acceleration time histories from second nearest SMA to Group 7 bridge.

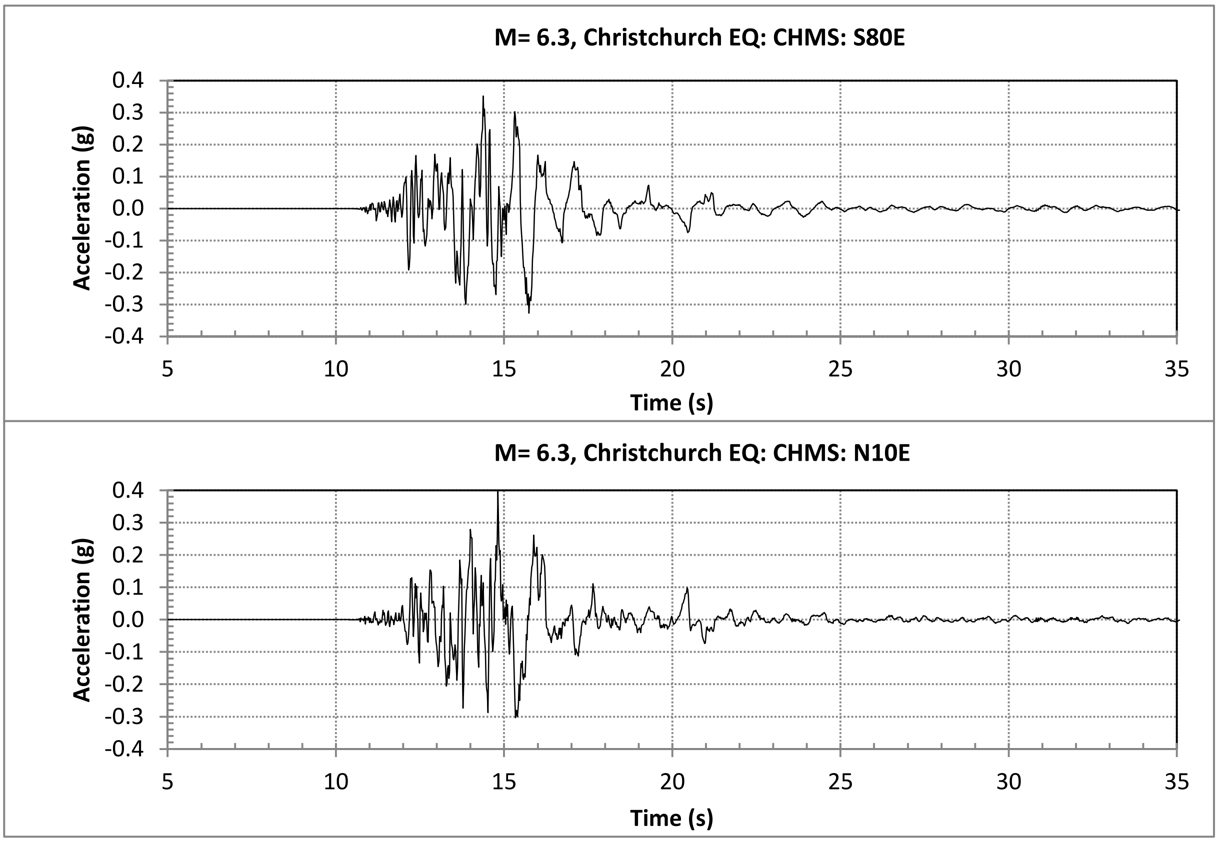


Figure A41. Acceleration time histories from nearest SMA to Group 7 bridge.

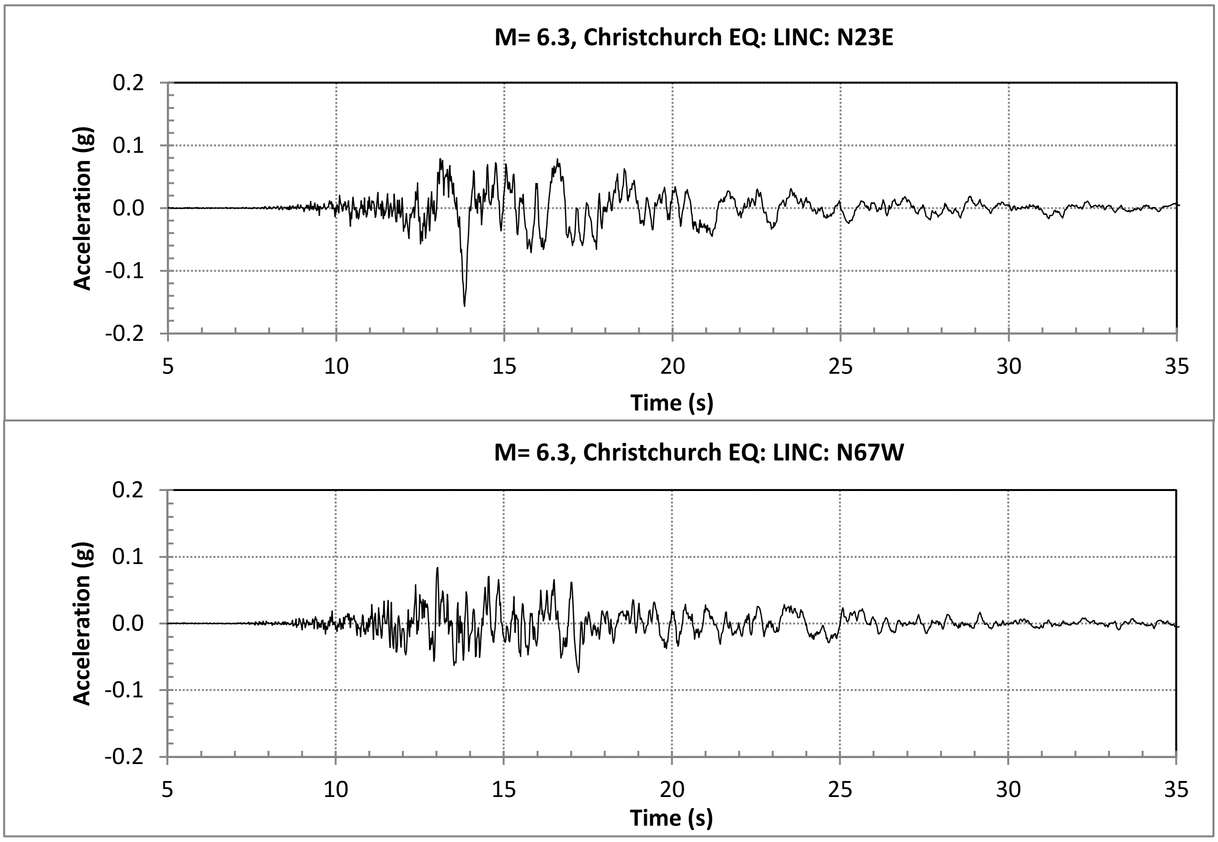


Figure A42. Acceleration time histories from second nearest SMA to Group 7 bridge.

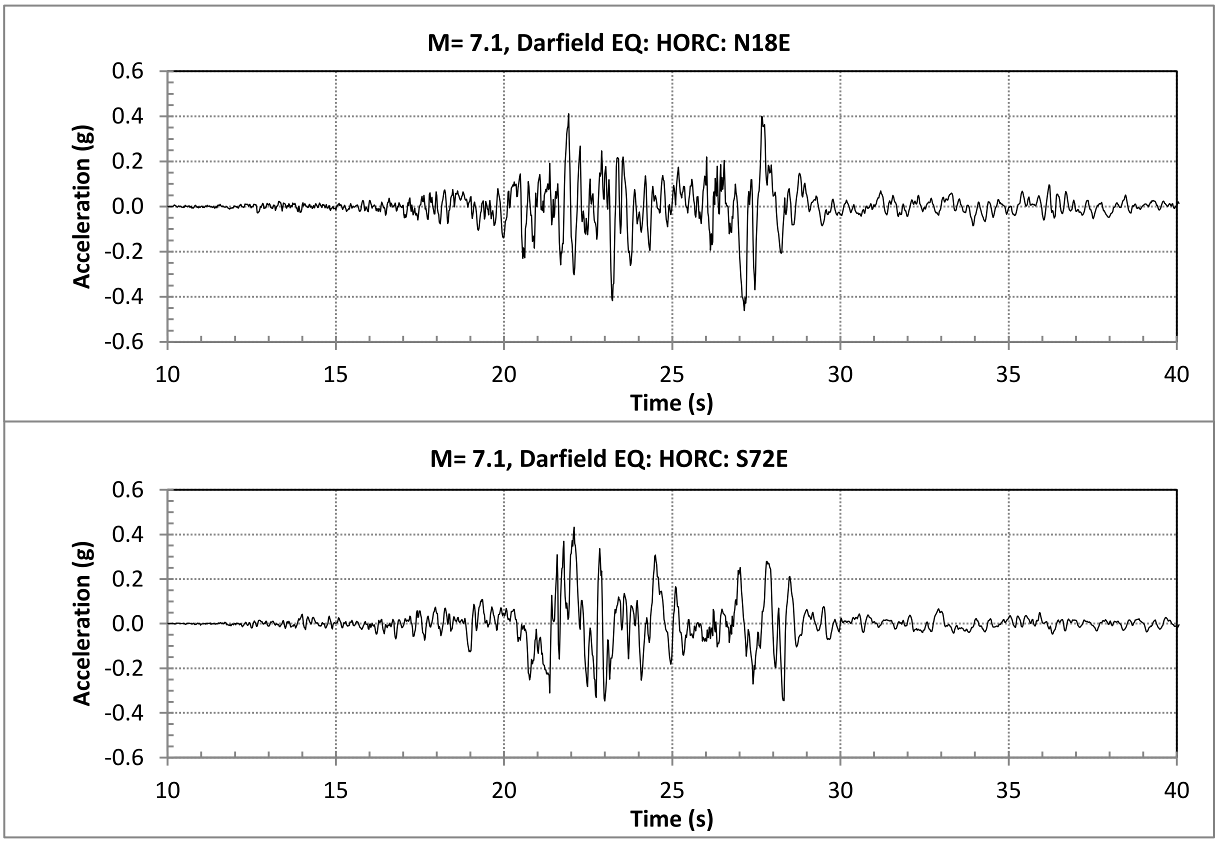


Figure A43. Acceleration time histories from nearest SMA to Group 8 bridges.

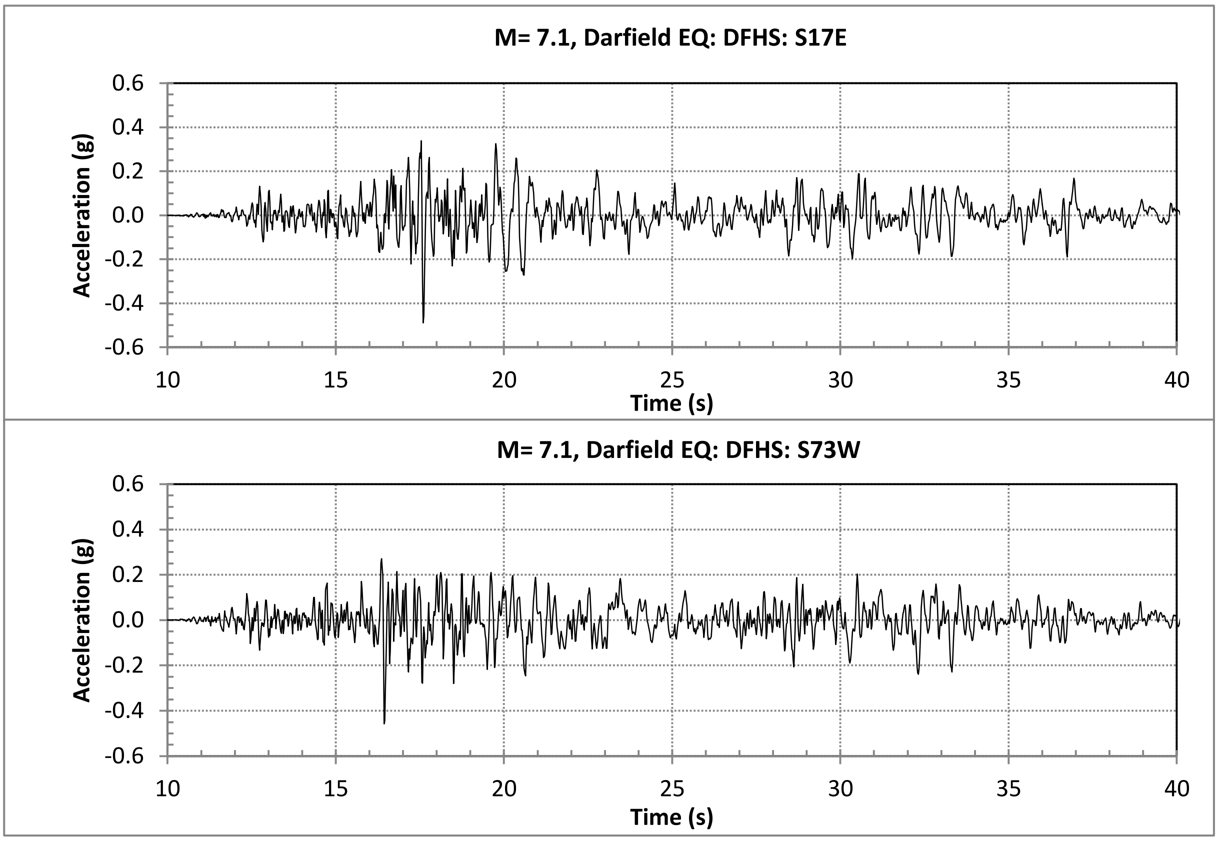


Figure A44. Acceleration time histories from second nearest SMA to Group 8 bridges.

APPENDIX B

DETAILS OF BRIDGES AND FIELD OBSERVATIONS MADE
(Refer to Table 3 of the Report for Bridge Groupings)

Note: The text in this appendix refers to figures and tables that are located either in the main text, in Appendix A or in this appendix. The figure and table numbers referred to are accordingly prefixed with either no prefix, A or B. There are also close-up photographs of damage in some bridges and these figures' numbers are prefixed with an identifier initial related to the name of the bridge.

GROUP 1 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA FOR 4 SEPTEMBER 2010: 5% DAMPING

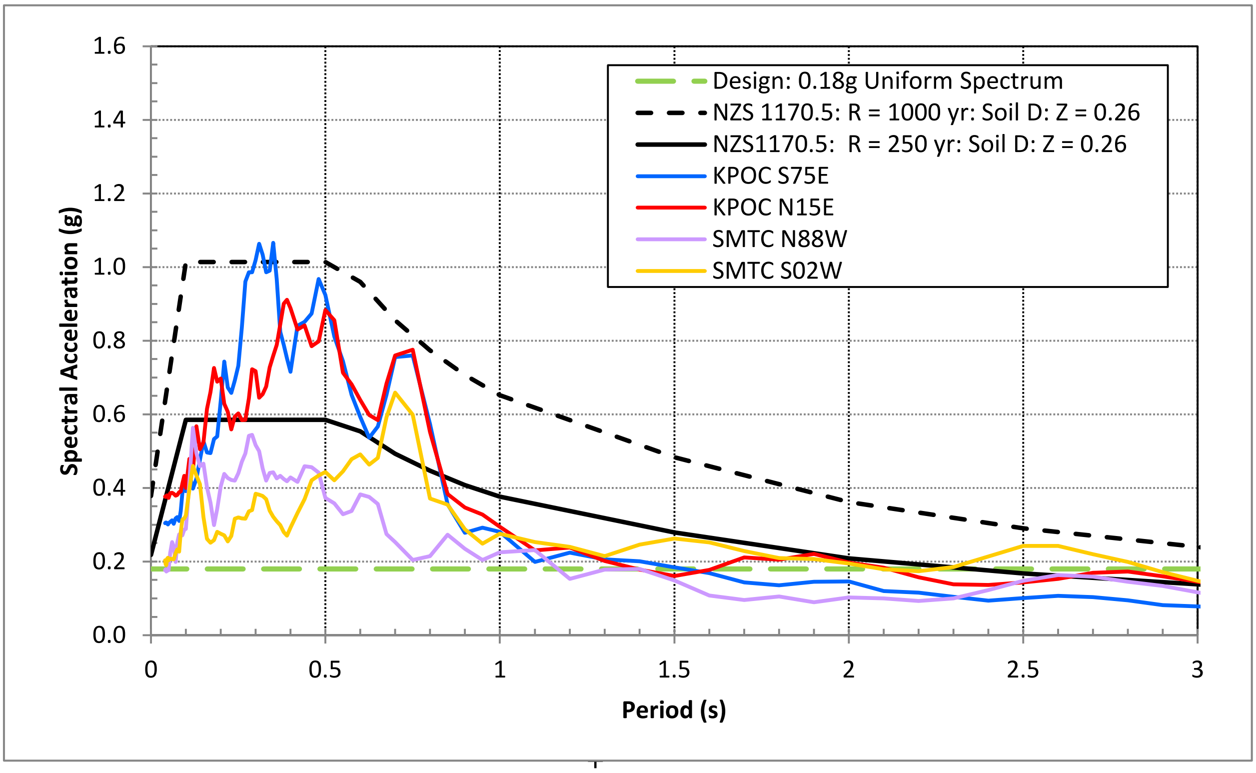


Figure B1: Acceleration response spectra from two nearest SMA's to Group 1 bridges - see Table 2.

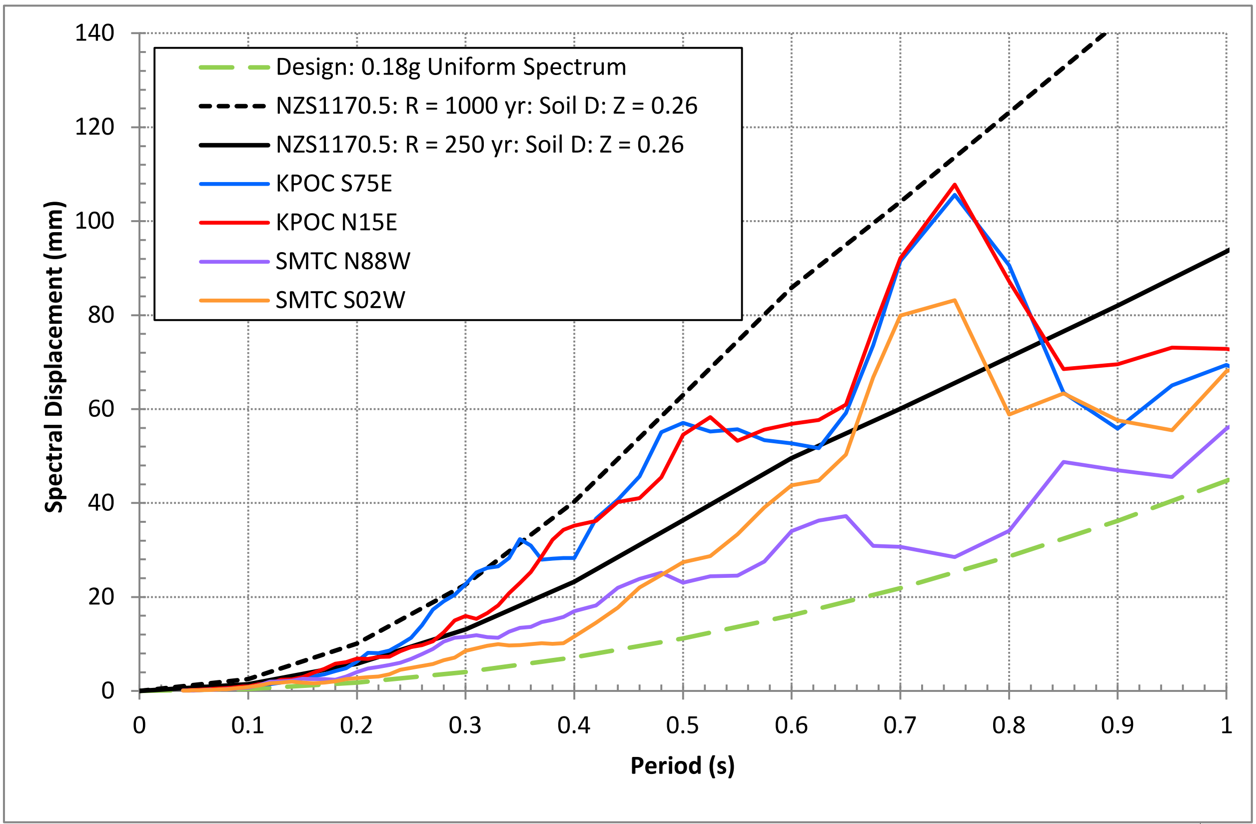


Figure B2: Displacement response spectra from two nearest SMA's to Group 1 bridges - see Table 2.

SH1 KAIAPOI RAILWAY AND RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
22 September 2010

Details of SH1 Kaiapoi Railway and River Bridge (Details per bridge - Two Similar Bridges)

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH1, Region 6; RP 317/5.83; BSN 3228</td></tr>
<tr><td>Location</td><td colspan="5">18 km north of Christchurch city centre</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">43 km Darfield EQ 4 Sep 10; 22 km Chch EQ 22 Feb ’11; 21 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">31 km Darfield EQ 4 Sep 2010</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.26 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1968. Year built 1970.</td></tr>
<tr><td>Geometry</td><td>Length: 149m</td><td>No Spans: 6</td><td>Max Span: 24.8m</td><td>Max. Ht: 7m.</td><td>Width over deck slab: 12.12m each carriageway</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight; 1.6m rise from north to south; 15º skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 lanes plus 2 shoulders on each carriageway.</td></tr>
<tr><td>Superstructure</td><td colspan="5">178 mm composite reinforced concrete deck on simply supported PSC I beams.</td></tr>
<tr><td>Piers</td><td colspan="5">Six circular RC columns total for the two bridges, carrying a RC crosshead forming a portal frame, and supported on a RC pile cap.</td></tr>
<tr><td>Abutments</td><td colspan="5">RC pile cap and backwall on 508mm square PSC piles. Friction slab attached to each abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">Single concrete pile cap for the two carriageways, on 508mm square PSC piles.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">Twelve boreholes, six to 12m depth and six to 21m, all showed sands and gravels to their full depth. Borelogs are held by NZTA with the drawings.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">> 21 metres</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: 2 holding-down dowels into each of 3 diaphragms per span per pier.</td><td colspan="2">Abutments: 2 holding-down dowels into each of 3 diaphragms per abutment.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: 3 linkage bolts in each of 3 diaphragms per bridge at each pier.</td><td colspan="2">Abutments: 3 linkage bolts in each of 3 diaphragms per bridge at each abutment.</td></tr>
<tr><td>Bearings flexible in Shear?</td><td colspan="3">Piers: Limited – due to HD bolts and thin (19mm) 178 mm x 457 mm single layer rubber bearings.</td><td colspan="2">Abutments: : Limited – due to HD bolts and thin (19 mm) 178 mm x 457 mm single layer rubber bearings.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">Piers and abutments are designed for an extra lane to be added on the median side of each bridge.</td></tr>
</table>

SH1 Kaiapoi Railway and River Bridge, looking north, (span three of six).

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

The bridge was one of the seven bridges in the Group 1 ground shaking intensity category. The mean PGA from the nearest two recorder stations to this group was 0.25 g. However, the bridge was only 1.5 km from the KOPC recorder location where a PGA of 0.34 g was recorded.

The bridge sustained significant secondary structural damage:

- Failure of the diaphragms between the span beams at the abutments (Figure K1). There was severe spalling at the bottom corners of all diaphragm sections and cracking across the top corners in some sections, suggesting primarily a shear failure of the connection to the beams near the bottom of the diaphragms. They are loaded by three linkage bolts positioned about 380 mm from the bottom edge of the diaphragm sections. It is understood that the damage was less pronounced at the south abutment, which was not inspected.
- Two of the 12 linkage bolts on the eastern side of the northern-most pier failed with fractures occurring through threads on the head end of the 38 mm diameter bolts (Figure K2). There was fine cracking across the bottom corners of some of the diaphragm sections on this pier.

Figure K1
Damaged diaphragm at abutment.

Figure K2
Fractured linkage bolt in pier diaphragm.

- There was spalling at ground level on one of the columns on the second nearest pier to the south end of the bridge. This was in an area of very low concrete cover and was primarily caused by corrosion evident on the circular stirrups although high strains during the earthquake may have increased the extent of the damage.
- Cracking in the surface soil and gapping at several columns in the second to nearest pier to the south end suggested some ground displacement towards the river bank. The gapping on the river side indicated this lateral displacement, and there was no gapping on the opposite side towards the abutment, indicating that the piers had not moved significantly during the earthquake.
- The pile tops were covered by either soil or water at the abutments and all the piers and therefore could not be inspected.
- On the southern approach embankment of the northbound lanes there was a shallow slide that undercut the guardrail and kerb foundation and extended for a length of about 45 m.
- No evidence of damage to the approaches due to relative movement of the abutments was seen, presumably because of the presence of the 11 metre long friction slabs attached to the backs of the abutments at a depth of 1.8 metres.

- There was a large failure on the steep western slope of the southern approach embankment, which had recently been protected with polythene sheeting and planting. There were no signs of liquefaction, and the failure may be related purely to strong ground shaking.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The friction slabs at the abutments make these components very stiff for any direction of loading. However, the beams are supported on 19 mm thick rubber pads at the abutments and piers and there are 38 mm thick rubber washers at one end of the linkage bolts. Gaps from shrinkage and creep at the joints (indicated as about 12 mm on joint repair drawings) allow some movement in the longitudinal direction at low response levels. The longitudinal period of vibration for small levels of displacement was estimated to be about 0.6 seconds. This would shorten as the gaps closed under strong shaking but damping would increase if passive resistance and sliding become significant at the abutments. In the transverse direction the first mode period of vibration was estimated to be about 0.7 seconds. Although the portal piers are relatively stiff there is some flexibility in the rubber bearing pads. Assuming simple SDOF response with 5% critical damping and no energy dissipation in the pile/ground connection (i.e. $S_p = 1$), the response accelerations would have been about 0.5 g in both the transverse and longitudinal directions (Figure 5) with a corresponding displacement response of about 50 mm (Figure A9).

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans. The analysis was based on the highest pier (Pier D) and it was assumed that the pile tops were at river bed level, although they may be about 500 mm above bed level. For an assumed steel yield stress of 300 MPa the analysis indicated that the pile sections would reach their ultimate flexural strengths at a response acceleration of about 0.55 g. The columns would reach their ultimate flexural strengths at about the same acceleration level and the outer span of the pier cap at a level between 0.5 to 0.55 g. A simple two-dimensional modal analysis showed that the transverse load carried by the highest two piers in the first transverse mode would be about 20% higher than given by the tributary mass static analysis assumption. For this bridge there is no significant deck diaphragm action because the linkage bolts with thick rubber washers are relatively flexible in tension and the beam bearing pads result in the abutments being not greatly stiffer in the transverse direction than the piers. The two separate (northbound and southbound) superstructures, although nominally identical, will not vibrate in phase and this could significantly reduce the peak dynamic response. Overall it is thought that the simple static analysis probably gave a reasonable approximation to the maximum loads carried by the two highest piers. The bridge performed somewhat better than expected as cracking damage would have been expected in the high piers at about the predicted response level of 0.5 g.

Structure Strength Longitudinally: The response of the bridge in the longitudinal direction is sensitive to the size of the gaps in the deck joints. If there were no gaps all the longitudinal force would be transferred to the "compression" abutment which is very stiff with good strength resistance from the combined effect of passive resistance, friction slab and piles. With wide gaps almost the entire load would be transferred to the "tension" abutment where the abutment resistance is provided by the friction slab and piles which are very stiff in relation to the piers. The abutment linkage bolts and bearings transfer the load to the abutment and neither is very stiff, so gaps need to provide for their axial and shear deformations before the complete load is transferred to the friction slab. The linkage bolt stiffness is significantly reduced by the rubber washers. The washers are strained into their non-linear range and become stiffer as the load increases to the bolt yield level. The drawings show that the bridge was originally constructed without joint gaps, using Malthoid as a separation membrane, but drawings prepared for joint repairs show nominal gaps of 12 mm. Creep and shrinkage following construction would open gaps to at least 5 mm. Assuming 10 mm gaps at the piers and 5 mm gaps at the abutments would result in approximately half the total longitudinal inertia force being carried by the "compression" and "tension" abutments under strong longitudinal response. Based on this distribution and an assumed yield stress of 265 MPa the linkage bolts at the "tension" abutment would yield at a response acceleration of about 0.55 g. The linkage bolts at the adjacent pier would require a higher response acceleration to cause the observed failures. However, they are also loaded by rotation of the span ends under transverse response and the proportion of the bridge inertia carried at the "tension" abutment may have been greater than assumed. The response acceleration in the longitudinal direction could have been higher than 0.5 g as the bridge was aligned in the same direction as the strongest of the two horizontal components recorded at the near-by KPOC recorder station.

The strength of the abutment diaphragm sections was estimated using a simple two-dimensional plate model loaded by the linkage bolts. This indicated that a shear friction type of failure would occur at the lower edge when the bolts were loaded to about 70% of their yield strength. As it seems likely that the bolts reached at least this level of loading failure of the diaphragms would be expected.

Conclusions

- Although the bridge appeared to perform a little better than expected in the transverse direction the damage observed to the diaphragms and linkage bolts was consistent with predictions for the longitudinal response.
- The bases of the pier columns and the tops of the outer piles at selected piers should be inspected.

SH1 WAIMAKARIRI RIVER BRIDGE (SOUTHBOUND AND NORTHBOUND)

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
21 September 2010

Details of SH1 Waimakariri River Bridge (Each Bridge)

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH; Region 6; RP 327/0.0; BSN 3270 &3271.</td></tr>
<tr><td>Location</td><td colspan="5">13 km north of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">41 km Darfield EQ 4 Sep 10; 19 km Chch EQ 22 Feb ’11; 18 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">28 km Darfield EQ 4 Sep 2010.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.26 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1965. Year built 1967.</td></tr>
<tr><td>Geometry</td><td>Length: 422m</td><td>No Spans: 17</td><td>Max Span: 24.8m</td><td>Max. Ht: 9m.</td><td>Width over deck slab: 9.9m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and level. 15º skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 lanes plus shoulders.</td></tr>
<tr><td>Superstructure</td><td colspan="5">Composite reinforced concrete deck on simply supported PSC I beams.</td></tr>
<tr><td>Piers</td><td colspan="5">Tapered RC walls 900 mm thick x average. 5.5 m, supported on RC pile cap and carrying a tapered RC pier cap 13 m long x 1.2 m wide.</td></tr>
<tr><td>Abutments</td><td colspan="5">RC pile cap and backwall supported on 6 x 508 mm square vertical PSC piles. Friction slab attached to each abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">8 x 508 mm square raked PSC piles per pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">Numerous borelogs, some to 30 m depth, show sands and sandy gravels over their full depth, generally compact at 15 to 20 m depths. Borelogs are held by NZTA with the drawings.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">> 30 metres.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">Not stated.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: 2 x 25 mm dowels into each of the 3 diaphragms at each end of each span.</td><td colspan="2">Abutments: 2 x 25 mm dowels into each of the 3 diaphragms at the end of each end span.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: 2 x 32 mm diameter linkage bolts linking each of 3 pairs of diaphragms at each pier.</td><td colspan="2">Abutments: 2 x 32 mm diameter linkage bolts linking diaphragms to backwall at each abutment.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: Limited – due to HD bolts and thin (19mm) 178 mm x 457 mm single layer rubber bearings.</td><td colspan="2">Abutments: : Limited – due to HD bolts and thin (19 mm) 178 mm x 457 mm single layer rubber bearings.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">Piers and abutments are designed for an extra lane to be added on the median side of each bridge.</td></tr>
</table>

SH1 Waimakariri River Bridge (southbound and northbound).
Central six spans of seventeen, looking north-west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

The southern six spans of both bridges were inspected from the south river bed. Because of time constraints, and because the network consulting staff confirmed that no damage had been seen on these bridges during their post-earthquake inspections, the northern ends of these bridges were not visited.

- No significant damage was observed during the inspection. However, the pile tops were covered by soil at the abutments and either soil or water at the all the piers visited and therefore could not be inspected.
- No signs of movement of the abutments or piers in the ground were noted.
- No structural cracking was seen.
- The linkage bolts showed no sign of disturbance.
- It appeared that the diaphragm end beam between the main beams at the southern abutment had moved several millimetres relative to the abutment seating with the direction of movement tending to close any deck joint.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The friction slabs at the abutments make these components very stiff for any direction of loading. However, the beams are supported on 19 mm thick rubber pads at the abutments and piers and there are 38 mm thick rubber washers at one end of the linkage bolts. In addition raked piles at the piers make the piers relatively stiff in the longitudinal direction. Because of the pier stiffness, and flexibility of the beam rubber bearings and the rubber washers in the linkage system only about 15% of the total longitudinal inertia force is resisted by the abutments. Gaps at the deck joints caused by shrinkage and creep of the spans allow some relative movement between the spans in the longitudinal direction at low response levels. The longitudinal period of vibration was estimated to be between 0.9 seconds to 1.0 s. In the transverse direction the first mode period of vibration was estimated to be between 0.5 to 0.6 s. Assuming simple SDOF response with 5% critical damping and no energy dissipation in the pile/ground connection (i.e. $S_p = 1$), the response accelerations would have been about 0.25 g and 0.5 g in the longitudinal and transverse directions respectively with corresponding displacement responses of about 60 mm and 40 mm. Because of the complex stiffness distribution in the longitudinal direction the maximum longitudinal SDOF displacement would only occur near the centre of the bridge.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and 50% of the inertia force from the pier. It was assumed that the pile tops were at river bed level (there is variation along the length of the bridge). For an assumed steel yield stress of 300 MPa the analysis indicated that the top sections of the outer piles would reach their ultimate flexural strengths at a response acceleration of about 0.36 g. Because of the relatively heavy piers and the variation in embedment depth of the piles along the length of the bridge the simple static analysis does not provide a good representation of the dynamic behaviour.

Structure Strength Longitudinally: A simplified static analysis was carried out for the longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the stiffness of the abutment structures for tension and compression direction loads, the stiffness of the piers with raked piles (assumed uniform along the length of the bridge), the shear stiffness of the beam bearing pads and the compression stiffness of linkage bolt rubber washers. This indicated that the longitudinal inertia load on the piers near the centre of the bridge was approximately 95% of the tributary mass from the adjacent spans. Loads on the three closest piers to either abutment were significantly less with part of the load in these spans being transferred to the abutments. Based on an assumed yield stress of 300 MPa for the pier reinforcement the analysis indicated that the piers would reach their ultimate flexural capacities at a response acceleration of about 0.22 g. Spalling damage would be visible at about this level. The pier main reinforcement is lapped at the base of the main pier section and there is no special confinement reinforcement so the piers would only have moderate levels of ductility. The abutments (backwalls and friction slabs), linkage bolts and the pier piles would not be stressed beyond their yield levels at the 0.22 g response acceleration level expected to damage the piers.

Although the Waimakariri River Bridge has similar end diaphragm beams to the ones that failed in the Kaiapoi Railway River Bridge they are only loaded by two linkage bolts instead of three and the bolt forces do not reach yield level at the predicted response acceleration.

Conclusions

- Even making some allowance for the approximations in the analysis the bridge appeared to perform better in the transverse direction than expected. The bridge also performed better in the longitudinal direction than predicted by a simple static analysis, assuming 5% damping.
- The response in the longitudinal direction would be influenced by travelling wave effects that result in a phase lag between the input motions at the piers along the length. Considering that the bridge is about 422 m long and that the shear wave velocity in the upper soil layers is probably less than 200 m/s the input motions could be strongly out of phase over the length of the bridge resulting in a significant reduction in the longitudinal inertia forces. Travelling wave effects might also reduce the response in the transverse direction.
- A more detailed assessment of the performance of this particular bridge in the earthquake would be warranted.
- An inspection of the pile tops at selected piers is required to determine whether there was any cracking or spalling damage to the outer piles.

SH1 CHANEYS ROAD OVERPASS

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
20 September 2010

Details of SH1 Chaneys Road Overpass

SH, Region, RP & BSN	SH1, Region 6, RP 327/1.92, BSN 3289				
Location	11 km north of Christchurch city centre.				
Distance to EQ Epicentre	40 km Darfield EQ 4 Sep 10; 17 km Chch EQ 22 Feb '11; 17 km Chch EQ 13 Jun '11.				
Distance to Fault Rupture	27 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.26 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1971. Year built 1972.				
Geometry	Length: 76m	No Spans: 3	Max Span: 30.5m	Max. Ht: 8m.	Width over deck slab: 12.2m
Alignment, Skew, Grade	650 m radius curve and 5% superelevation, Piers and abutments radial to curvature.				
No of Lanes	2 lanes plus shoulders.				
Superstructure	Prestressed concrete deck and 4 solid tapered ribs 560 m wide (ave.) by 1120 mm deep.				
Piers	Slightly tapered RC wall piers 6.4 m (ave.) wide by 530 mm (ave.) thick. Piers are hinged to superstructure and pile cap for longitudinal rotation at top and bottom.				
Abutments (A – North, D – South)	North: RC abutment block and backwall supported on 4 RC walls onto RC pile cap, which is carried on 23 x 430 mm square raked PSC piles. This abutment acts as the longitudinal anchor to the bridge. South: RC abutment and backwall supported on 9 x 430 mm square raked PSC piles. Abutment drawings show no settlement or friction slabs.				
Pier Foundation	RC slab pile cap 8.7 m x 2.4 m wide by 1.2 m thick, supported on 16 x 430 mm square raked PSC piles.				
Soils, Borehole info.	Generally loose sands or sandy gravels to 12 m depth, with more compact gravels or sandy gravels below to 30 m. 3 borelogs to 30 m depth are held by NZTA with the dwgs.				
Depth of Sediment	>30 metres.				
Liquefaction Risk	Extensive liquefaction occurred in a wide area around the bridge.				
Hold-down System	Piers: 20 (top) and 24 (bottom) x 32 mm long dowels through the concrete hinges at top and bottom of the piers.	Abutments: North: 6 x 32mm HD dowels. South: 6 x 32mm HD dowels in slotted holes through superstructure.			
Linkage System	Continuous superstructure between abutments.	Abutments: North: Superstructure anchored to abutment with downstand shear keys. 10 x 36 mm dia linkage bars retrofitted South: No linkages fitted at sliding end as overlap presumably considered adequate.			
Bearings flexible in shear?	Piers: Concrete hinges with dowels – no transverse flexibility but free longitudinal movement.	Abutments: North: Superstructure rests 4 x 19 mm thick rubber pads 406 mm x 381mm. South: Steel bearings provide for rotation and longitudinal movement but restraint transversely.			
General Condition	Good with no deterioration observed.				

SH1 Chaneys Road Overpass, looking south-west towards Christchurch.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

The bridge structure sustained only minor damage but the approaches and abutment slopes were significantly affected.

- The retrofitted linkage bolts - 10 bolts fitted with toroidal rubber rings at the north abutment (Abutment A) at which the structure is anchored longitudinally - were loose with a maximum gap of about 5 mm between the nuts and steel washers that bear on the rubber rings (Figure C1).
- Permanent transverse movement of about 5 mm of the bridge eastwards relative to the south abutment (Abutment D, at which the superstructure is free to slide longitudinally but is restrained transversely) indicated by the closing of the clearance gap between the fingers in the deck finger joint plates (Figures C2 and C3).
- Slumping and settlement of the abutment slopes which had displaced the precast concrete paving slabs of the abutment aprons. Soil slumping was worst on the northern slope where the upper paving slabs were dislodged by cracking in the soil (Figure C4).
- Ground settlement of the approaches at both abutments was of the order of 100 mm relative to the abutment structures (Figure C5).

- The piers had tilted towards the northern abutment by about 10 mm over their height but this displacement could have been caused prior to the earthquake by creep and shrinkage, which would have shortened the bridge by a total of at least 30 mm at Abutment D.
- Relative movements between the northern approach embankment and the bridge had twisted several guardrail posts on the approach.
- The settlement and embankment movements had separated and misaligned the superstructure drainage pipes from their connection points on the abutment slopes (Figure C6).
- Lateral spreading of the northern approach fill caused longitudinal cracking of the pavement and cracks near the top of the eastern slope near the guardrail posts. We were told that the need for pavement repair to eliminate the differential step~~s~~ between the backfill and abutment wall at the northern abutment~~s~~ and pavement damage on the approaches restricted the bridge to a single lane with a 30 km/h speed restriction for four or five days following the earthquake.

Figure C1
Linkage bars
at north abutment.

Figure C2
Finger joint
at south
abutment.

Figure C3
Bearings at south
abutment.

Figure C4
Damaged
paving at
north
abutment.

Figure C5
Repaired
pavement at north
approach.

Figure C6
Settlement at
abutment.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Liquefaction and Ground Damage: The Geology of the Christchurch Area, 1: 250,000 Geological Map 16 (Geological and Nuclear Sciences Ltd, 2008) shows the geology of the site to be Holocene age “grey river alluvium”. The overpass is located on an embankment approximately 6.8 m high. It is located about 1,350 m south of the Waimakariri River.

Site investigations have previously been carried out as part of the Christchurch Northern Arterial / QE II Drive Scheme Stage Investigation & Reporting. The site investigations near Chaneys Road Overpass included cone penetration tests. Additional cone penetration tests were carried out at Chaneys Road Overpass following the September 2010 earthquake (see Figure C7). No boreholes are available in close proximity to the bridge site. The cone penetration tests indicate interbedded loose to medium dense sand, silt-sand and silt and some clay layers in places.

There was extensive liquefaction in the 4 September 2010 Darfield earthquake, apparent as sand boils and extensive lateral spreading of the on-ramp, see Figure C7. The liquefaction did not cause extensive damage to the highway embankment either side of the bridge. The embankment north of the bridge had longitudinal cracks in the lanes indicating that the embankment slopes had displaced outwards. There were also signs of deformation on the slopes of the approach embankment on the north-east side of the bridge.

The overburden pressure from the embankment appears to have reduced the extent of liquefaction under the approach embankments. Liquefaction analyses (assuming a peak ground acceleration of 0.17g from Styx Mill Transfer Station SMTC) using the results of CPTs carried out from ground level (away from the embankment) indicate that the ground between 2 m depth (below assumed groundwater level) and about 9 m depth is likely to have liquefied, with the exception of some dense layers of sand and some silt layers. If this liquefaction had extended below the embankment, then there would have been extensive lateral spreading failure of the embankments. However, when the full additional overburden pressure effect of the embankment was taken into consideration, the analysis indicates that only very thin layers are likely to have liquefied in the Darfield earthquake. Stability analyses indicate that this very limited liquefaction would have caused lateral spreading of tens of millimetres. The observed displacement of the order of 100 mm and longitudinal cracks in the embankment indicated that it was likely that some liquefaction did take place, probably under the flanks of the embankment where the overburden pressure is smaller. The relatively short duration of the Darfield earthquake would also have been a factor limiting the displacement of the approach embankments. The subsidence associated with the liquefaction would have led to settlement of the embankments relative to the piled bridge structure, which was observed as a step between the bridge structure and its approaches.

The observations and assessment highlight the importance of appropriately taking into consideration the overburden pressure effects of the highway embankments in the seismic assessment and design of highway structures. Based on evidence from the CCC Bridge Street Bridge approach embankments, embankment overburden pressure is not always sufficient to prevent liquefaction.

Structure Response: The bridge is very stiff in both longitudinal and transverse directions. In the longitudinal direction the bridge is anchored to the raked piles of the northern abutment and in the transverse direction there is significant diaphragm action in the continuous superstructure, which distributes part of the inertia load from the central pier-supported section of the bridge to the stiff piled abutments. First mode periods of vibration were estimated to be in the range of 0.3 to 0.35 seconds for longitudinal response and 0.2 to 0.3 seconds for transverse response. Assuming simple SDOF response with 5% critical damping the response accelerations in the earthquake would have been about 0.55 g and 0.5 g in the longitudinal and transverse directions respectively with corresponding displacement responses of about 15 mm and 10 mm.

Structure Strength Transversely: Approximate static analyses were carried out for transverse loading on separate models of the piers and abutments to estimate the stiffness of their piled foundations. The distribution of the transverse earthquake loads on the piers and abutments was estimated by static analysis using a two-dimensional model consisting of a beam representing the deck and linear springs representing the transverse stiffness of the piers and abutments. Static analysis of this simplified model was used to estimate the distribution of the transverse inertia loads on the piers and abutments. For an assumed steel yield stress of 300 MPa in the reinforcement the analysis indicated that the top sections of the outer piles at the piers and all the piles in the southern abutment (Abutment D) would reach their ultimate flexural strengths at a response acceleration of about 0.5 g. The analysis also indicated that the 19 mm diameter bolts fixing the runner bars to the top plates of the steel bearings at Abutment D, which restrain the bridge transversely but allow sliding longitudinally, would fail in shear at a response acceleration of about 0.6 g.

There were no obvious signs of transverse movements of the piers at ground level. However, the expected displacements of the pier piles at their ultimate strength capacities (reduced by tension in the outer piles) would only be about 5 mm and a displacement of this order might not be distinct at the interface of the pier and the surface soil. Because of the soil disturbance from settlement and slumping at the abutments, it was not possible to determine whether there had been significant lateral movements.

Structure Strength Longitudinally: A simplified static analysis was carried out for the longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and carried by the raking pile system at the anchoring north abutment (Abutment A). It is not possible to reliably predict the pull-out resistance of the raking abutment piles as detailed soil properties are not available and liquefaction may have affected the frictional resistance. Our best estimate is that pull-out would be likely at a response acceleration of about 0.3 g. Damping associated with pull-out would be high and significant movements from rotation of the abutment would be unlikely at response accelerations up to about 0.5 g. The combined capacity of the shear keys, holding down bolts, bearing friction and linkage bolts is sufficient to resist the outward inertia force from the superstructure (away from the soil) from a response acceleration of about 0.55 g. However, there is a large variation in the relative stiffness of the various linkage components and the concrete shear keys constructed with small contact gaps would initially carry most of the load. They would probably commence to fail at a response acceleration of about 0.15 g, transferring the load to the linkage and holding down bolts. A possible explanation for the loose linkage bolts is that they yielded after the shear keys were damaged and that the looseness is a result of the bridge superstructure coming to rest after it had moved back against the abutment backwall. The downstands of the shear keys (monolithic with the superstructure) are not visible and it was therefore not possible to confirm their condition. The abutment backwalls have sufficient shear strength and passive resistance to resist the superstructure inertia force from a response acceleration of about 0.45 g (not including the resistance from the pile foundations) so loading in the direction towards the backfill would be unlikely to cause damage.

The analysis indicated that the four vertical tapered walls supporting the seating block and backwall at Abutment A would reach their flexural capacities (at their base) when loaded by the superstructure inertia force from a longitudinal response acceleration of about 0.45 g and their shear capacities (at their top sections) at a response acceleration of about 0.5 g. They are covered by soil and so it was not possible to confirm their condition during our inspection. Since pullout of the piles was likely to occur at lower response accelerations than predicted to damage the columns this may have influenced their damage threshold.

Conclusions

- An inspection of the pile tops at the south abutment (Abutment D) and the outer pile tops at the adjacent pier (Pier C) should be carried out to determine whether they are damaged. The wall columns supporting the north anchoring abutment (Abutment A) should be inspected at their junction with the pile cap.
- The liquefaction at and below the approach embankments should be further investigated and assessed, as this may have a beneficial effect in the assessment of embankments on liquefaction prone ground.

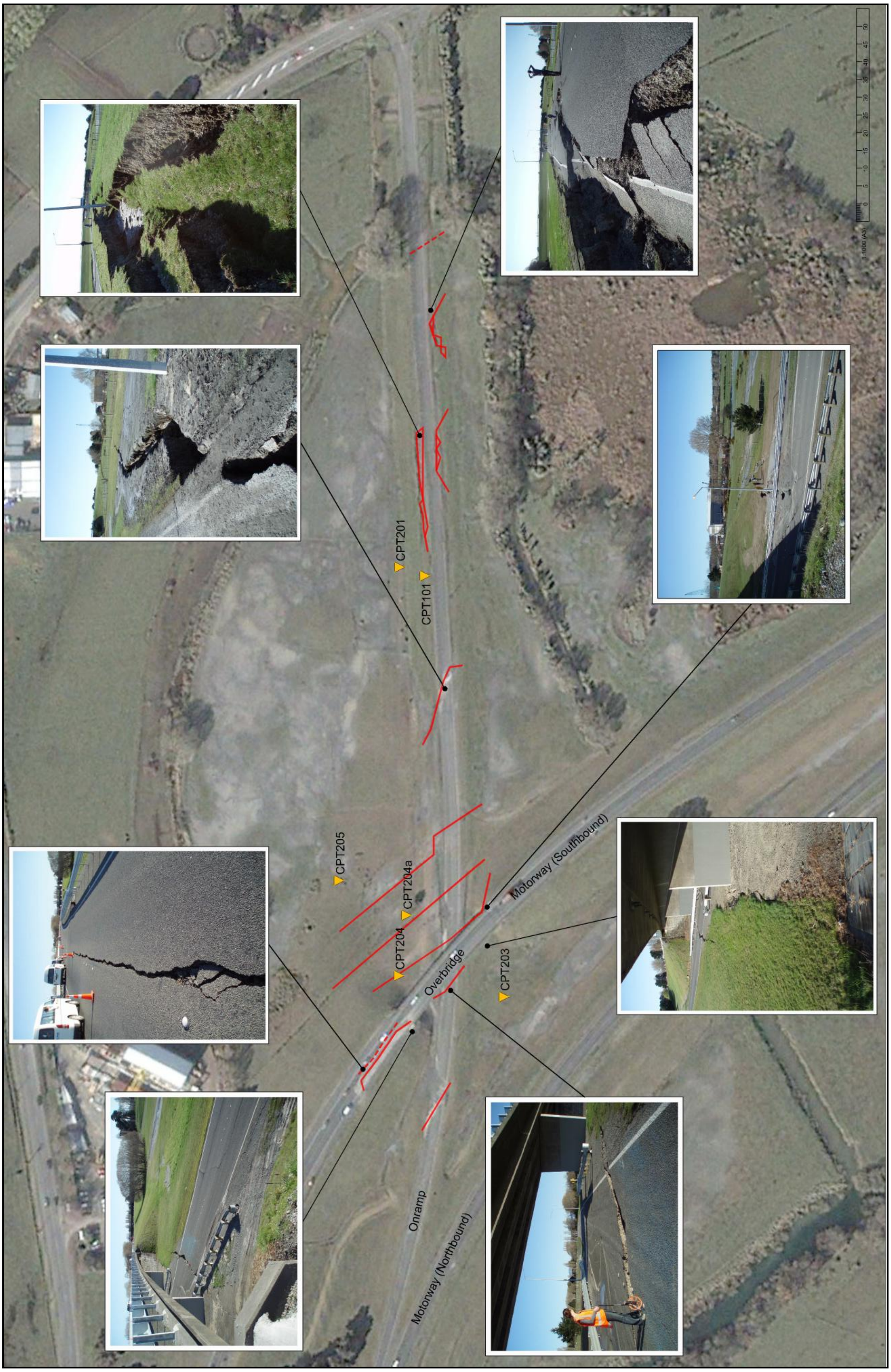


Figure C7. Ground damage observed near Chaneys Road Overpass.

SH71 CAM ROAD UNDERPASS

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
20 September 2010

Details of SH71 Cam Road Underpass

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="6">SH71; Region 6; RP 0/0.95; BSN 10</td></tr>
<tr><td>Location</td><td colspan="6">18 km north of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="6">43 km Darfield EQ 4 Sep 10; 23 km Chch EQ 22 Feb ’11; 23 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="6">31 km Darfield EQ 4 Sep 2010.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="6">0.26 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="6">Designed 1967. Year built 1969.</td></tr>
<tr><td>Geometry</td><td>Length: 61m</td><td>No Spans: 4</td><td>Max Span: 18.3m</td><td>Max. Ht: 7.5m.</td><td colspan="2">Width over deck slab: 19.8m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="6">Straight and practically level (200 mm rise at mid length). 11º skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="6">3 lanes plus median, shoulder and 2 footpaths.</td></tr>
<tr><td>Superstructure</td><td colspan="6">RC 8-cell box girder with cantilevered footpaths.</td></tr>
<tr><td>Piers</td><td colspan="6">3 x 914 mm diameter RC columns per pier monolithic with box girder diaphragm.</td></tr>
<tr><td>Abutments</td><td colspan="6">RC abutment block and backwall supported on 8 x 430 mm square vertical PSC piles. Friction slab attached to each abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="6">3.05 m x 2.4 m x 1.2 m thick pile cap under each pier column, each supported on 6 x 430 mm square PSC piles and linked by tie beam to others at the pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="6">Generally sands or sandy gravels to 7 m depth, with more compact gravels or sandy gravels below, to 21 m. 2 borelogs to 21 m depth are held by NZTA with the drawings.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="6">>21 m</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="6">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: None required as columns are monolithic with box girde.r</td><td colspan="3">Abutments: 8 x 19 mm holding down dowels.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Continuous superstructure between abutments.</td><td colspan="3">Abutments: 9 x 32 mm linkages cast into each end diaphragm of box girder and passed through the abutment backwalls.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: Monolithic construction.</td><td colspan="3">Abutments: 9 Freyssinet 4-layer rubber bearings 457 mm x 254 mm x 56 mm thick at each abutment.</td></tr>
<tr><td>General Condition</td><td colspan="6">Good with no deterioration observed except some fine cracking (shear?) in the girder outer webs near the abutments.</td></tr>
<tr><td>Other Features</td><td colspan="6">None.</td></tr>
</table>

SH71 Cam Road Underpass, looking east towards Kaiapoi.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following minor points were noted:

- There was a fine crack about 300 mm below the top of the northern column of the eastern pier. It was distinct only on the south side of the column suggesting that it was probably a flexural crack. The bending moments from horizontal earthquake loading are greatest near the tops of the columns, which are essentially fixed against rotation where they frame into the box girder diaphragm beams. However, the main steel area at the base of the columns where they frame into the pile caps is only about 25% of the area at the tops of the columns resulting in the base of the columns being more critical in flexure than the column tops. The bottoms of the columns are covered by a depth of about 700 mm of soil and were not inspected.
- Detailed investigation work was carried out on the pier columns following the inspection and involved excavation down to the bases of the columns. This revealed fine circumferential cracking above ground level in the three columns at each of the end piers and the centre column of the central pier. The cracks were less than 0.1 mm in width and were located at distances of up to 1.0 m above ground level. No cracking was found below ground level. The cracks indicate that the longitudinal 32 mm diameter bars had

reached yield level during the earthquake. The unexpected location of the cracks above ground level was probably related to laps in all the column bars at the pile cap level.

- The fill against the abutment backwalls had settled by an estimated depth of about 50 mm. This settlement was indicated by the new sections of pavement that had been placed on the approaches following the earthquake, and the misalignment of the kerbs on the approach fill where they were in contact with drainage sumps constructed monolithic with the abutments. Settlement and kerb disturbance appeared to be greatest at the south-east corner of the bridge. The abutments are fitted with friction slabs and these probably reduced the depth of surface settlement.
- There was evidence of settlement of up to 100 mm at the face of the abutment seating where it is in contact with the paved aprons under the bridge. Only minor disturbance to the aprons was visible indicating little down-slope movement.
- A photograph taken shortly after the earthquake showed cracking along the centre of the eastern approach ramp, indicating a small amount of lateral spreading of this approach. Other photographs showed sand ejection and water pools near the site, indicating that there had been local liquefaction, which may have caused some of the lateral spreading and settlement.
- The horizontal 150 mm diameter drainage connection pipes that are cast into the box girder end walls and pass through cored holes in the abutment backwalls appeared to be undamaged. Similar pipes on the Tram and Ohoka Road Underpasses were cracked and this might indicate that the transverse movements of the Cam Road Underpass were less than on the other two bridges. A 75 mm gap between the ends of the box girder and the abutment backwall allows the pipe connection to be readily inspected. There was no indication that this gap had been significantly reduced by movements of the approaches.
- Near-vertical fine cracking was very evident on the outer web faces of the box girder near the abutments. Apparently this cracking has been present for a long period of time and was not related to the earthquake.

  (Subsequent investigation by the Regional Bridge Consultant concluded that the cracking was not related to gravity shear overload. It is also unlikely to be seismic related.)

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: A Detailed Seismic Assessment (DSA) of the bridge was completed by Opus International Consultants (Opus) in October 2010 after the earthquake. We also completed a simplified longitudinal static analysis to verify some of the DSA results. Periods of vibration of 0.64 and 0.65 seconds for the longitudinal and transverse directions respectively were stated in the DSA. Our analyses indicated that the periods would be about 0.6 seconds. Assuming simple SDOF response with 5% critical damping the response accelerations in the earthquake would have been about 0.5 g in both the longitudinal and transverse directions with corresponding displacement responses of about 45 mm.

Structure Strength Transversely and Longitudinally: Non-linear pushover analyses carried out as part of the DSA indicated that plastic hinging would form in the base of the columns at a response acceleration of 0.36 g for both the transverse and longitudinal directions. The pier columns were found to be the most critical elements with the pile cap tie beams found to be the next most critical. Hinging was predicted to occur in the tie-beams at a response acceleration of about 0.41 g under transverse loading.

The DSA was based on a probable yield strength of the reinforcing of 250 MPa which seems unduly conservative. The drawings state that the allowable working stress for the reinforcement was 138 MPa (20,000 lb/in^2). At the time of the design the steel working stress for flexure was not taken to be greater than 0.5 times the yield stress, so the specified yield stress of the reinforcement used in the bridge was likely to be 275 MPa. The probable yield strength of the reinforcing is therefore expected to be 1.1 x 275 = 304 MPa. Thus the response acceleration expected to cause plastic hinging in the base of the columns is about 0.45 g which is significantly greater than the 0.36 g indicated in the DSA.

Conclusions

- Based on the DSA analyses it appears that the bridge may have performed better than expected. However, the reinforcement steel strength was probably higher than assumed in the DSA and the cracking in the columns near ground confirmed that yield level had been reached in the column reinforcement during the earthquake indicating that the performance was not much better than expected.
- It is possible that the earthquake response accelerations were less than the 0.5 g estimated. The continuous box girder bridge is a relatively simple structure and the DSA analyses should have given a reasonably accurate estimate of the bending moments in the columns for elastic response. The simple analyses indicated that about 20% of the superstructure inertia loads were transferred to the abutments. If the bearings were stiffer than assumed a greater proportion of the load could have been distributed to the abutments, reducing the loads on the columns.
- The bridge was only 1.4 km from the KPOC recorder where the record indicated response accelerations of about 0.6 g for the 0.6 seconds spectral ordinate with 5% critical damping. The corresponding response accelerations at the SMTC station about 11 km from the bridge were significantly less at about 0.4 g. Liquefaction or other site features may also have reduced the ground motions at the bridge to nearer the levels recorded at SMTC.
- The return period of the shaking that the bridge experienced is estimated to be 180 to 220 years, compared with the current design standard of at least 1000 years. It is therefore quite likely that the pier columns would be damaged at their lower ends during stronger shaking. NZTA is in the process of arranging installation of additional column confinement for these elements by fibre wrapping.

TRAM ROAD UNDERPASS (OVER SH1)

Inspection by: J H Wood and H E Chapman **Date of Visit**: 14 October 2010

Details of Tram Road Underpass

SH, Region, RP & BSN	Over SH1, Region 6; RP 317/9.36; BSN 3264.				
Location	11 km north of Christchurch city centre.				
Distance to EQ Epicentre	41 km Darfield EQ 4 Sep 10; 20 km Chch EQ 22 Feb ’11; 19 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	29 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.26 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1966. Year built 1967.				
Geometry	Length: 62 m	No Spans: 4	Max Span: 18.3m	Max. Ht: 6m.	Width over deck slab: 13.7 m
Alignment, Skew, Grade	Straight and level. No skew.				
No of Lanes	2 Lanes, 2 shoulders and 2 footpaths.				
Superstructure	RC multi-cell box girder with cantilevered footpaths.				
Piers	2 x 838 mm diameter RC columns per pier monolithic with box girder diaphragm.				
Abutments	RC abutment block and backwall supported on 6 - 430 mm square vertical PSC piles. Friction slab attached to each abutment.				
Pier Foundation	2.6 m x 2.13 m x 1.07 m thick pile cap under each pier column, each supported on 6 x 430 mm square PSC piles and linked by tie beam to other at the pier.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	No information seen.				
Hold-down System	Piers: None required as columns are monolithic with box girder.	Abutments: 5 x 19 mm holding down dowels.			
Linkage System	Continuous superstructure between abutments.	Abutments: 38 mm linkages cast into each end diaphragm of box girder and passed through the abutment backwalls.			
Bearings flexible in shear?	Piers: Monolithic construction.	Abutments: 6 - Freyssinet 4-layer rubber bearings 380 mm x 254 mm x 40 mm thick at each abutment.			
General Condition	Good with no deterioration observed.				
Other Features	None.				

Tram Road Underpass, looking west.

RESULTS OF INSPECTION

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following minor points were noted:

- On the northern column of the west pier there was a single visible fine flexural type crack near the top of the column.
- The 150 mm diameter cast iron drainage connection pipes between the bridge end diaphragms and the abutment walls were fractured. Drainage connections are located at the four corners of the bridge and it appeared that all four pipes had been broken by transverse movement of the superstructure relative to the abutment seats. Relative movements exceeding 20 mm would be expected because of flexibility in the 55 mm high rubber bearings supporting the ends of the box girders.
- Minor settlement and kerb spalling damage was evident at the interfaces between the approach fill and the abutments. Settlement and transverse movements had caused minor misalignment of the handrail joint between the bridge and approach ramp at the south-west corner of the bridge.
- Near-vertical fine cracking was very evident on the outer web faces of the box girder near the abutments. Apparently this cracking has been present for a long period of time and was not related to the earthquake.

(Subsequent investigation by the Regional Bridge Consultant concluded that the cracking was not related to gravity shear overload. It is also unlikely to be seismic related.)

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

A comparison of the design details of the Tram Road Underpass with the Cam and Ohoka Road Underpass details was carried out by Opus International Consultants in May 2011. They also investigated the transverse response of the bridge using an inelastic static push-over analysis. They concluded that the periods of vibration would be similar to the Cam Road Underpass resulting in similar response accelerations and that the flexural capacity at the base of the pier columns would be about 20% higher. The push-over analysis indicated that the flexural capacity would be reached in the base of the columns at a response acceleration of about 0.38 g (compared to 0.36 g for Cam Road). As for the Cam Road Underpass, conservative assumptions were made in the Opus analysis regarding the yield stress in the reinforcement. If a more likely value for yield stress is used (see Discussion on Cam Road Underpass), plastic hinges would be expected to develop in the base of the columns at a response acceleration of about 0.5 g. This is about the response acceleration level that was estimated to have occurred in the Darfield earthquake.

The push-over analysis indicated that plastic hinging would occur in the transverse tie beams that are located between the pile foundation caps at each column at a response acceleration of about 0.39 g (based on a conservative yield stress). That is, at a response acceleration slightly greater than required to form the hinges in the pier columns. Shear failures were predicted to occur in some of the columns at a response acceleration of about 0.6 g.

Conclusions

- Based on the analyses carried out by Opus it appears that the bridge performed a little better than expected.
- As for the Cam Road Underpass, it is possible that the force actions on the pier columns were less than estimated by the analyses. The conclusions presented above for the Cam Road Underpass regarding the distribution of superstructure loads and the proximity of the recording stations apply equally to the Tram Road Underpass.
- The Tram Road Underpass is about 2.3 km further from the KPOC station than the Cam Road Underpass and it may have experienced less intense shaking as there appeared to be a significant fall in intensity moving away from this SMA (see Figure 2).
- The return period of the shaking that the bridge experienced is estimated to be between 180 to 220 years, compared with the current design standard of at least 1000 years. It is therefore quite likely that the pier columns would be damaged at their lower ends during stronger shaking. NZTA is in the process of arranging installation of additional column confinement for these elements by fibre wrapping.

OHOKA ROAD UNDERPASS (OVER SH1)

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
22 September 2010

Details of Ohoka Road Underpass

SH, Region, RP & BSN	SH1; Region 6; RP 317/6.89; BSN 3239.				
Location	16 km north of Christchurch city centre.				
Distance to EQ Epicentre	42 km Darfield EQ 4 Sep 10; 22 km Chch EQ 22 Feb ’11; 21 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	30 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.26 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1966. Year built 1969.				
Geometry	Length: 62m	No Spans: 4	Max Span: 18.3m	Max. Ht: 7.4m.	Width over deck slab: 17.7 m
Alignment, Skew, Grade	Straight and level. No skew.				
No of Lanes	3 Lanes plus median, 1 shoulder and 2 footpaths.				
Superstructure	RC multi-cell box girder with cantilevered footpaths.				
Piers	2 x 990 mm diameter RC columns per pier monolithic with box girder diaphragm.				
Abutments	RC abutment block and backwall supported on 8 x 430 mm square vertical PSC piles. Friction slab attached to each abutment.				
Pier Foundation	3.35 m x 3.35 m x 1.14 m thick pile cap under each pier column, each supported on 8 x 430 mm square PSC piles and linked by tie beam to other at the pier.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	No information seen.				
Hold-down System	Piers: None required as columns are monolithic with box girder.	Abutments: 7 x 19 mm holding down dowels.			
Linkage System	Continuous superstructure between abutments.	Abutments: 6 x 32 mm linkages cast into each end diaphragm of box girder and passed through the abutment backwalls - assumed.			
Bearings flexible in shear?	Piers: Monolithic construction.	Abutments: 8 Freyssinet 4-layer rubber bearings 457 mm x 254 mm x 56 mm thick at each abutment.			
General Condition	Good with no deterioration observed except significant near-vertical cracking in girder web near abutment.				
Other Features	None.				

Ohoka Road Underpass, looking west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following minor points were noted:

- On the northern column of the west pier there was a single visible fine flexural type crack near the top of the column.
- There was a small concrete spall at the top of the southern column on the east pier but it is not clear whether this was related to earthquake loading.
- The 150 mm diameter cast iron drainage connection pipes between the bridge end diaphragms and the abutment walls were fractured. Drainage connections are located at the four corners of the bridge and it appeared that all four pipes had been broken by transverse movement of the superstructure relative to the abutment seats. Relative movements exceeding 20 mm would be expected because of flexibility in the 55 mm high rubber bearings supporting the ends of the box girders.
- Minor settlement and kerb spalling damage was evident at the interfaces between the approach fill and the abutments.
- Near-vertical fine cracking was very evident on the outer web faces of the box girder near the abutments. Apparently this cracking has been present for a long period of time and was not related to the earthquake.

(Subsequent investigation by the Regional Bridge Consultant concluded that the cracking was not related to gravity shear overload. It is also unlikely to be seismic related.)

There were no reports of further damage following the 22 February 2011 Christchurch earthquake but spalling damage was observed at ground level in the south column of the eastern pier following the M_L 6.0 aftershock on the 23 December 2011. The epicentre of this 7 km deep event was located 18 km to the south of the bridge. At the time of preparing this report records were not available for this event from the KPOC SMA, which was the closest recorder. The SMTC SMA located 9.3 km to the south of the bridge recorded a PGA of 0.07 g.

Following an initial inspection of the spalling damage the ground surrounding the columns on the two end piers was excavated down to the bases of the columns (tops of pile caps). This detailed inspection revealed:

- The south column of the eastern pier had significant cracking at the base with crack widths in excess of 1 mm. The concrete was drummy and with very little effort the cover concrete was removed exposing an area of approximately 1.2 m high from the base by 0.8 m wide (Figure O1). Removal of the cover revealed poorly compacted concrete surrounding the main bars. There was no evidence of buckling of the vertical reinforcement. A circumferential crack of 0.5 mm in width extended around the column base.
- The south column on the central pier had a spall at the top of the pier.
- The north column of the central pier had a clearly visible circumferential crack at the top of the pier.
- The south column of the western pier had an area of concrete spall at the top of the pier, and a circumferential crack of 0.5 mm in width at the base of the column.
- The north column of the western pier had a clearly visible area of cracking at the top of the column.
- All the columns were found to have extensive fine surface cracking in both principal directions that was probably related to shrinkage during concrete curing.

Discussion

A comparison of the design details of the Tram Road Underpass with the Cam and Ohoka Road Underpass details was carried out by Opus International Consultants in May 2011. The Ohoka Road Underpass was estimated to have slightly lower periods of vibration than the Cam Road Underpass but the difference was not significant and it would have experienced a similar response acceleration of about 0.5 g in the Darfield earthquake. The comparison indicated that the flexural capacity in the bottom of the piers would be reached at response acceleration about 25% higher than for the Cam Road Underpass (about 0.55 g).

Conclusions

- It appears that the bridge performed a little better than expected since at the predicted response acceleration more obvious cracking would have been expected at the base of all the columns.
- The spalling damage at the base of the south column on the eastern pier probably resulted from the accumulation of cracking in the cover concrete during the main events and aftershocks. As cracking progressed the cover would gradually separate from the poorly compacted concrete surrounding the main bars.

- The Ohoka Road Underpass is about 0.6 km south of the Cam Road Underpass and probably experienced a similar intensity of ground shaking.
- As for the Cam Road Underpass, it is possible that the force actions on the pier columns were less than estimated. The conclusions presented above for the Cam Road Underpass regarding the distribution of superstructure loads and the proximity of the recording stations apply equally well to the Tram Road Underpass.
- The return period of the shaking that the bridge experienced is estimated to be between 180 to 220 years, compared with the current design standard of at least 1000 years. It is therefore quite likely that the pier columns would be more seriously damaged at their lower ends during stronger shaking. NZTA is in the process of arranging installation of additional column confinement for these elements by fibre wrapping. Repair of the poor quality concrete at the base of the south column of the eastern pier will be carried out prior to his retrofitting.

Figure O1
Spalling damage at the base of the south column of the eastern pier. The cracked concrete has been removed to reveal poorly compacted concrete around some of the reinforcing.

OLD WAIMAKARIRI RIVER (MAIN NORTH ROAD) BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010
21 September 2010

Details of Old Waimakariri River (Main North Road) Bridge

SH, Region, RP & BSN	Local Authority bridge – not on state highway.				
Location	13 km north of Christchurch city centre.				
Distance to EQ Epicentre	42 km Darfield EQ 4 Sep 10; 19 km Chch EQ 22 Feb ’11; 18 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	29 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.26 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1929. Year built presumably early 1930’s.				
Geometry	Length: 354m	No Spans: 29	Max Span: 12.2m	Max. Ht: 7m.	Width over deck slab: 6.6m
Alignment, Skew, Grade	Straight and level. No skew.				
No of Lanes	2 Lanes plus kerbs.				
Superstructure	178 to 229 mm RC deck on 6 - 610 mm x 184 mm simply supported steel I beams. Beams are interlinked at their ends and mid-span by 203 mm thick RC diaphragm beams cast after beams placed.				
Piers	610 mm thick x 1067 mm deep RC pile cap supporting 457 mm thick solid wall and 762 mm. thick X 914 mm deep integral pier cap.				
Abutments	610 deep x 152 mm thick RC backwall on 1220 mm deep x 610 mm thick RC wall, supported on 5 x 406 mm octagonal RC piles.				
Pier Foundation	6 x 406 mm octagonal RC piles per pier.				
Soils, Borehole info.	None referred to but probably similar to the Waimakariri River Bridge about 500 m upstream, where boreholes show sands and sandy gravels over their full depth up to 30 m, generally compact at 15 to 20 m depths.				
Depth of Sediment	Probably > 30 metres.				
Liquefaction Risk	Not identified.				
Hold-down System	Piers: 32 mm diameter HD bolts, apparently from the drawings and inspection) only through the outer bottom flanges of the outer beams (i.e. only 2 per pier).			Abutments: As at the piers.	
Linkage System	Piers: There are no linkages shown connecting the spans either on the drawings or seen during the inspection.			Abutments: As at the piers.	
Bearings flexible in shear?	Piers: No – beams rest on mortar bedding.			Abutments: No – beams rest on mortar bedding.	
General Condition	Good with no significant deterioration observed.				
Other Features	The minimal holding down bolts and lack of inter-span linkages warrants action to reduce the risk of spans dropping in a major earthquake shake.				

Waimakariri River (Main North Road) Bridge, looking north.
(Railway bridge on the right)

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

Minor structural damage was observed during the inspection, which warrants some follow-up investigation.

- The pavement had been repaired at the southern abutment indicating significant settlement of the approaches relative to the pile supported abutment sill beams. A photograph taken soon after the earthquake showed a step of about 20 mm at the backfill interface with the abutment backwall.
- The small return wall at the east end of the beam seating at the southern abutment had fractured with large vertical cracks in both the return wall and the end of the seating backwall. There had been significant transverse movement of the superstructure relative to the seating and this may have resulted in a handrail post contacting and damaging the end wall. However, the backwall above the seating, behind the beams and diaphragms, may have failed resulting in the damage to the return wall (Figure W1).
- Relative movement between the beams and the abutment was indicated by disintegration of some of the mortar used to seat the beams (Figure W2), and cracking and spalling in the concrete diaphragms between the ends of the beams above their seatings (Figure W3). The cracking was mainly vertical near mid-span of the diaphragms and the spalling from the bottom corners. Similar damage to the mortar and diaphragms was observed at the northern abutment.
- The southern abutment has been underpinned in the past using bags filled with concrete and placed to a height of about 2 metres under the sill beam and across the front of the

piles. The underpinning was presumably carried out to allow an access road to be constructed under the bridge near the abutment. There was cracking in the top of some of this infill concrete indicating significant movement of the abutment relative to the ground under the bridge (Figure W4).

- The southernmost pier (Pier No 2 on the drawings) is on the bank of the river and is in contact with the soil slope on the bank side. It had a horizontal crack of 2.5 mm maximum width extending across part of its width on the river side (Figure W5). The stiffening effect of the soil would have resulted in this pier carrying a larger part of longitudinal load than many of the other piers as at least 14 of the 28 piers have pile tops showing and would be more flexible.
- Horizontal cracking was evident in piers that were marked with numbers 7 and 8 (probably Pier Numbers 23 and 22 on the drawings). The crack in Pier 7 was located about 400 mm above the pile cap and had a maximum width of about 2 mm (Figure W6). Only a fine crack was visible in Pier 8. The pile caps on both these piers are partially covered with bed material, which would have increased their stiffness. Gapping of up to 15 mm wide between the face of the piers and the river bed material, indicating significant longitudinal movement of the tops of the piers, was evident at piers labelled with numbers 2, 3 and 6. The ground completely covered the pile caps on these piers and the other piers between them and the northern abutment. There were cracks about 25 mm wide in the soil on the river side of the pier labelled number 1 (closest pier to the northern abutment) at about 1 m from the pier (Figure W7). A photograph taken shortly after the earthquake indicated that there had been liquefaction near this pier and the cracks could have been related to this or by heaving or settlement in gently sloping weak surface soils.

Damage Following the 22 February 2011 Christchurch Earthquake

Although not inspected by the authors of this report, the bridge subsequently suffered significant local damage during the February 2011 earthquake. The damage included some bulk movement of the southern abutment and associated ground towards the river, and consequent distortion of the southernmost pier. Span beams were also displaced, without losing support. Remedial work has been undertaken.

Discussion relating to Performance in the 4 September 2010 Earthquake

Structure Response: Although the single row of pier piles is flexible in the longitudinal direction the bridge is stiffened by the 1.9 m high abutment walls to give an estimated longitudinal period of vibration in the range of 0.6 to 0.7 seconds. The relatively low wall type piers with six piles spaced uniformly across their width result in high transverse stiffness and a first transverse mode of vibration estimated to be in the 0.13 to 0.2 second range. Assuming simple SDOF response with 5% damping the response accelerations would have been about 0.4 g in both the transverse and longitudinal directions with corresponding displacement responses of about 5 mm and 50 mm respectively

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and 50% of the inertia force from the pier. It was assumed that the pile tops were at river bed level, although there is variation along the length of the bridge. For an assumed steel yield stress of 230 MPa the analysis indicated that the top sections of the outer piles would reach their ultimate flexural strengths at a response acceleration of about 0.6 g. Although the spans are not well anchored to the piers they are relatively light and the combined shear strength of the two holding down bolts at each span end and beam bearing friction resistance would be sufficient to resist a response acceleration of at least 0.7 g. Damage to the bridge from transverse loading would therefore not be expected.

Structure Strength longitudinally: A simplified static analysis was carried out for the longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the relative stiffness of the abutment structures for tension and compression direction loads and the stiffness of the piers. For simplicity, the tops of the pier piles were assumed to be at bed level but there is significant variability with the pile tops generally in the range of 1 m above to 1.5 m below bed level. This analysis indicated that the abutment and pier piles would reach their ultimate strengths at response accelerations of about 0.20 g and 0.25 g respectively. The pier walls would reach their ultimate flexural strengths at a response acceleration of about 0.30 g.

No damage to the piles was observed but the critical section for longitudinal response would be at a depth of about 1 m below ground surface and this section of the piles was not visible at any of the piers. Where the bed was above the pier pile tops, and at the abutments, no part of the pile length was visible and undetected damage could be present particularly at the abutments. An inspection of some of the abutment piles should be carried out to confirm their condition.

The cracked piers are stiffened by soil against their pile caps and in this situation the bending moments in the piers might be more critical than in the piles. Bond failure in the plain round bars may have been a factor and could have caused the single wide horizontal cracks observed in two of the piers.

Under a longitudinal response acceleration of 0.25 g (corresponding to a displacement of the bridge superstructure of about 30 mm) the inertia load transmitted to the abutment being pushed against the soil matches the maximum combined passive resistance of the abutment seating and backwall and piles of about 3500 kN. The shear friction capacity of the backwall is about 1300 kN and the combined shear resistance of the hold-down bolts and bearing friction is about 500 kN. Additional resistance to the superstructure load comes from the direct passive pressure force on the backwall of about 350 kN but there is clearly insufficient resistance to meet the full 0.25 g response acceleration demand. The holding down bolts and backwall would therefore be expected to fail at a response acceleration of less than 0.25 g. Simplified calculations indicate that the failure level could be as low as 0.15 g. The observed damage to the return wall at the east end of the southern abutment may therefore be related to a shear failure at the base of the backwall where it joins the seating sill beam. Excavation and inspection of part of the eastern end of the backwall needs to be carried out to confirm its condition.

At the abutment being pulled away from the soil the combined shear strength of the hold-down bolts and the resistance from beam bearing friction are probably sufficient to develop plastic hinging in the abutment piles so failure of the hold-down bolts (and backwall) is only likely when the abutment is being pushed against the soil.

Conclusions

- The bridge performed very much better in the longitudinal direction than predicted by a simple static analysis assuming 5% damping. However, the response in the longitudinal direction would be influenced by travelling ground wave effects that result in a phase lag between the input motions at the piers along the length. The reduction from this effect has not been estimated but considering that the bridge is 354 m long and that the shear wave velocity in the upper soil layers is probably less than 200 m/s the input motions could be strongly out of phase over the length of the bridge resulting in a significant reduction in the longitudinal inertia forces. Travelling wave effects and the lack of uniformity in the pier foundation stiffness might also reduce the response in the transverse direction.

- Investigations are recommended for the following:
  - Some of the abutment piles to confirm their condition;
  - Excavation and inspection of part of the eastern end of the abutment backwall needs to be carried out to confirm its condition.
- The minimal holding down bolts and lack of inter-span linkages warrant action to reduce the risk of spans dropping in a major earthquake shake.

Figure W1
Damaged abutment return wall
at south abutment.

Figure W2
Damaged mortar seating under beam
at south abutment.

Figure W3
Cracked and spalled diaphragm
at south abutment.

Figure W4
Cracked underpinning at south abutment.

Figure W5
Horizontal crack in Pier 2.

Figure W6
Horizontal crack in Pier 23 400 mm above pilecap.

Figure W7
Cracking in ground around Pier 29.

GROUP 2 BRIDGE:

- **The Response Spectra**
- **Description of Bridge, Observations Made, and Discussion**

THE RESPONSE SPECTRA FOR 4 SEPTEMBER 2010: 5% DAMPING

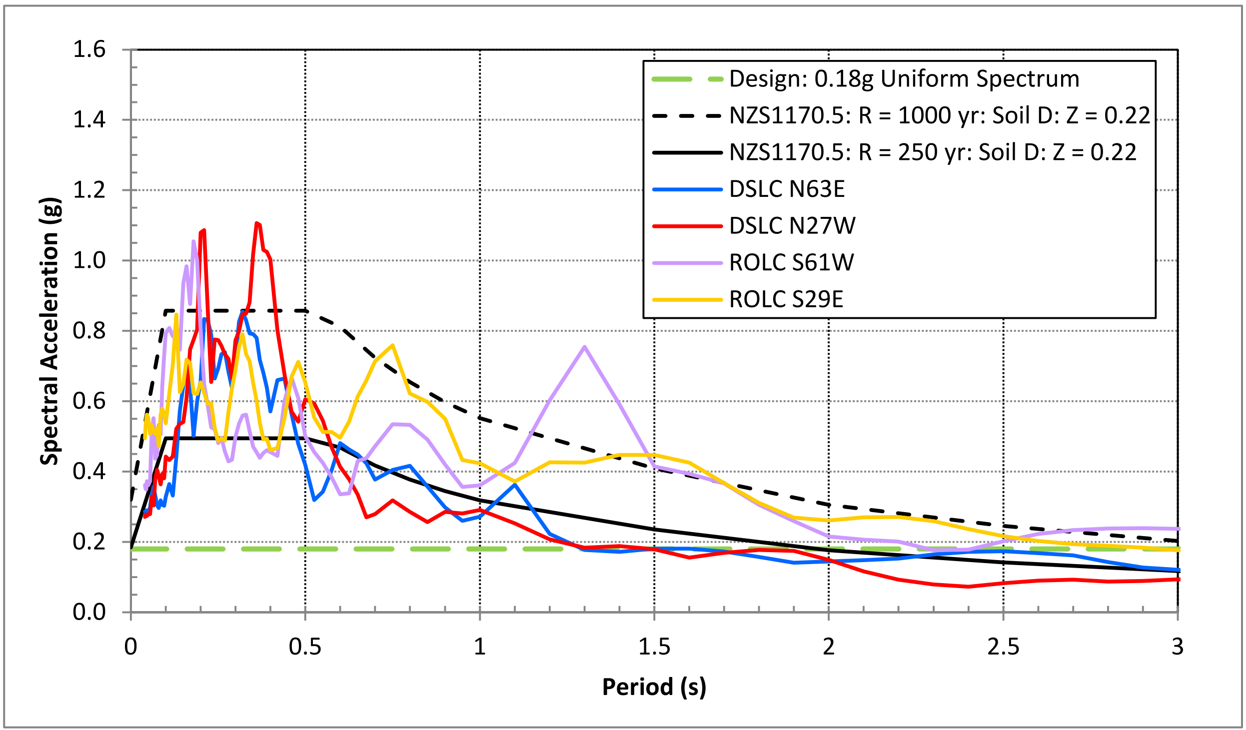


Figure B3: Acceleration response spectra from two nearest SMA's to Group 2 bridge - see Table 2.

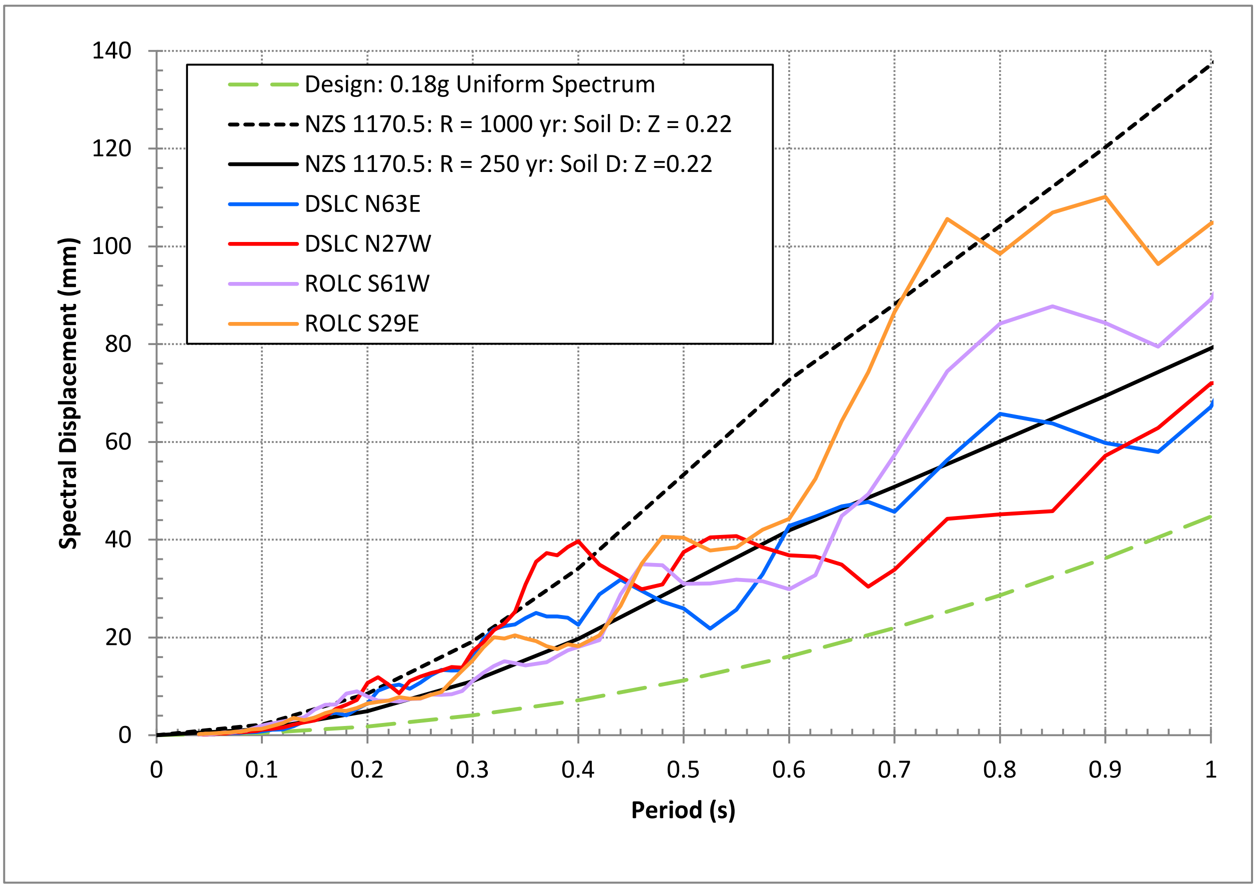


Figure B4: Displacement response spectra from two nearest SMA's to Group 2 bridge - see Table 2.

SH1 SELWYN RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 15 October 2010
23 September 2010

Details of SH1 Selwyn River Bridge

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH1; Region 6; RP 391/0.0; BSN 3810.</td></tr>
<tr><td>Location</td><td colspan="5">35 km west south-west of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">11 km Darfield EQ 4 Sep 10; 38 km Chch EQ 22 Feb ’11; 42 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">6 km Darfield EQ 4 Sep 2010.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1920. Year built: presumably early to mid 1920’s; Deck widened 1984; Seismic retrofit with linkages 2006.</td></tr>
<tr><td>Geometry</td><td>Length: 320m</td><td>No Spans: 35</td><td>Max Span: 9.14 m</td><td>Max. Ht: 7m.</td><td>Width over deck slab: 8.54m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and practically level (drops 300 mm over its length); no skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus kerbs.</td></tr>
<tr><td>Superstructure</td><td colspan="5">180 to 220 thick RC deck slab composite with 6 RC T-beams, simply supported at all piers. Spans fixed to abutments and either both fixed or both sliding at alternate piers.</td></tr>
<tr><td>Piers</td><td colspan="5">530 mm thick x 6.5 m wide x 2.2 m high RC wall thickened and widened as pier cap.</td></tr>
<tr><td>Abutments</td><td colspan="5">530 mm thick x 6.5 m wide x 1.2 m high RC wall carrying 304 mm thick RC backwall to deck level and supported on 4 x 406 mm vertical octagonal RC piles per abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">4 x 406 mm vertical octagonal RC piles per pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">1 borehole record at south end of bridge to 11 metres depth shows compact sandy gravel over full depth.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">>11 metres.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">Not identified.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers:
Fixed beams: 2 x 16 mm vertical bars per beam end into pier
Sliding beams: 2 x 19 mm HD bolts per beam end.
“New” outer girders: No hold-downs.</td><td colspan="2">Abutments:
Inner two girders: 3 x 22 mm 45º sloping bars anchoring beam to abutment
Former outer girders: 2 x 22 mm 45º sloping bars anchoring beam to abutment
“New” outer girders: No hold-downs.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers:
Fixed ends: Spans interlinked with 22 mm bars: 3 at inner two girders and 3 at former outer girders
Sliding ends: No linkages but linkage bar retrofit at these in 2006 provided for this.</td><td colspan="2">Abutments:
45º Sloping bars, as above, act also as linkages.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: No – spans fixed at alternate piers and on slide plates longitudinally with HD bolts in slotted holes, fixing it transversely.</td><td colspan="2">Abutments: No bearings.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no significant deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">Bridge was widened in 1984 by adding corbels to the ends of the piercaps and installing two additional beams on each side of the deck.</td></tr>
</table>

SH1 1 Selwyn River Bridge, looking north.
(Northern spans of thirty five.)

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No earthquake-related structural damage to the bridge was observed during the inspection. Spalling was present at the top of one of the pier piles but this appeared to be impact damage and not related to earthquake loading. The pile tops were only visible on about 40% of the piers near the present river channel and were not visible near the ends of the bridge or at the abutments. If the piles had suffered damage from longitudinal response this would be at the maximum bending moment points about 1 m below ground level and would not have been visible. There was little evidence of significant displacements of the bridge at the tops of the piles or at the abutments suggesting that the response was less than indicated by displacement predictions based on a SDOF assumption.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The bridge has relatively short piers (2.1 m from top of piles to underside of beams) and is therefore quite stiff transversely with the piles effectively fixed against rotation at the underside of the pier walls. It is more flexible in the longitudinal direction where stiffness is provided by 2.1 m high abutment backwalls and the cantilever action of the abutment and pier piles. Because of the length of the bridge, the abutment walls do not provide a major contribution to the overall longitudinal stiffness. Elastic periods of vibration were estimated to be between 0.2 to 0.3 seconds transversely and 0.5 to 0.6 seconds longitudinally. Assuming simple SDOF response with 5% critical damping the response accelerations would have been about 0.7 g and 0.5 g in the transverse and longitudinal directions respectively. Corresponding response displacements would have been about 10 and 35 mm.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and that the pile tops were 0.5 m clear of the river bed. For an assumed steel yield stress of 230 MPa the analysis indicated that the pile bars would yield at a response acceleration of about 0.3 g and that the ultimate flexural strength of the piles would be reached at about 0.35 g. It was assumed that the pile bars were ⅞ inch diameter but they may have been ¾ inch diameter. The bridge clearly performed in the transverse direction better than expected. The shaking intensity at the site may have been less than estimated as the PGA recorded at the nearest recorder at Dunsandel (3.6 km to the south) was 0.25 g, which is a little less than the best estimate for the site of 0.3 g. In the transverse direction there is a variation in pier stiffness along the length of the bridge which will result in several modes of vibration with closely spaced periods. These modes can interact to reduce the response estimated using the SDOF assumption. For example, using one of Dunsandel recorded acceleration time histories as input, the combined response of three transverse modes with periods of 0.2, 0.25 and 0.3 s, and equal participation factors, was found to give a peak response of 0.4 g compared with the 0.57 g peak for single mode response. This effect needs further investigation for long bridges.

Structure Strength Longitudinally: An approximate static analysis for the longitudinal direction based on the assumption that all the response was in-phase indicated that piles would reach their ultimate flexural strengths at a structure response acceleration of about 0.25 g. The critical sections would be about 1.0 m below river bed level. Again the bridge performed significantly better than predicted by this simple analysis. The response in the longitudinal direction would be strongly influenced by travelling ground wave effects that result in a phase lag between the input motions at the piers along the length. The reduction from this effect has not been estimated but considering that the bridge is 310 m long and that the shear wave velocity in the upper soil layers is probably less than 200 m/s the input motions could be strongly out of phase over the length of the bridge resulting in a large reduction in the longitudinal inertia forces. Travelling wave effects might also reduce the response in the transverse direction.

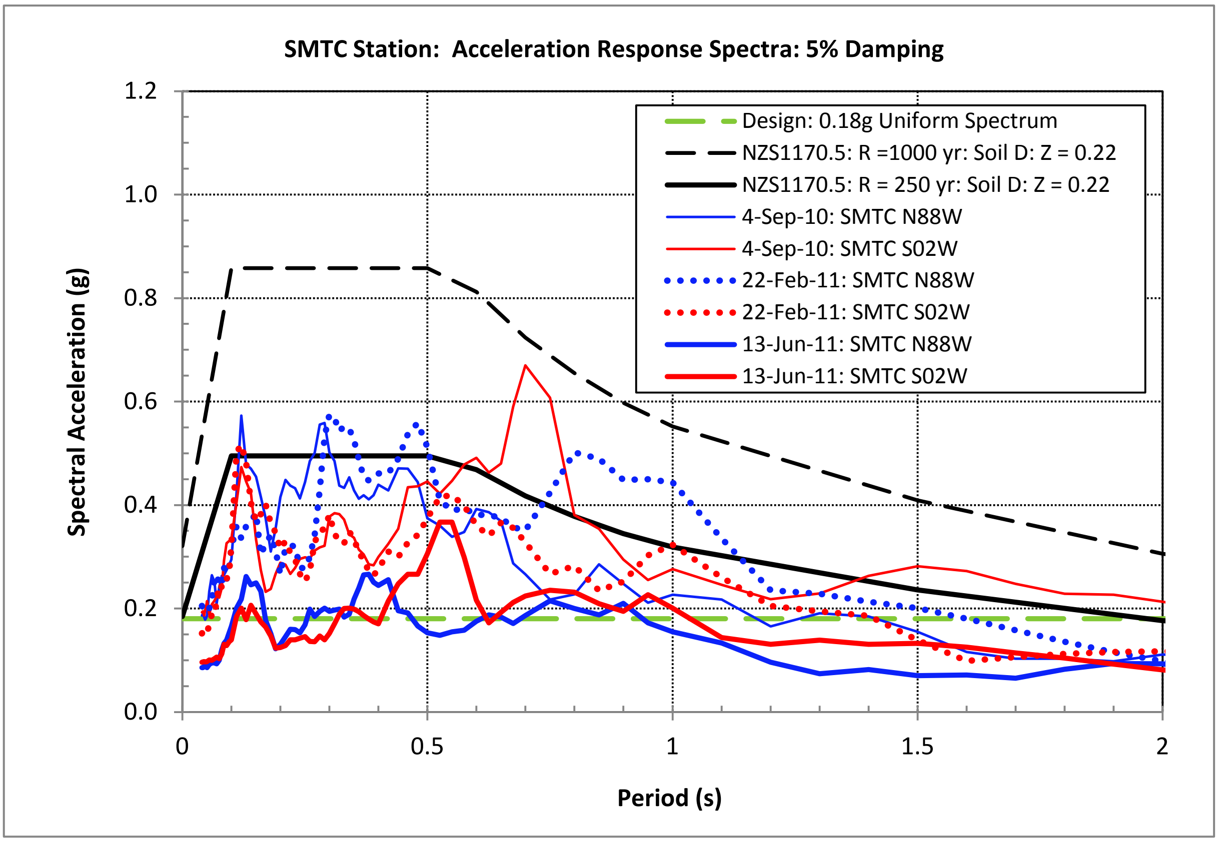
Conclusions

- The bridge performed very much better transversely than predicted by the simple analysis, perhaps because of multi-modal effects reducing the response.
- The bridge performed significantly better longitudinally than predicted by a simple analysis, perhaps because of travelling ground wave effects.
- The effect of travelling ground waves and multi-modal effects on long bridges needs further investigation.
- A more detailed assessment of the performance of this bridge in the earthquake would be warranted.

GROUP 3 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA



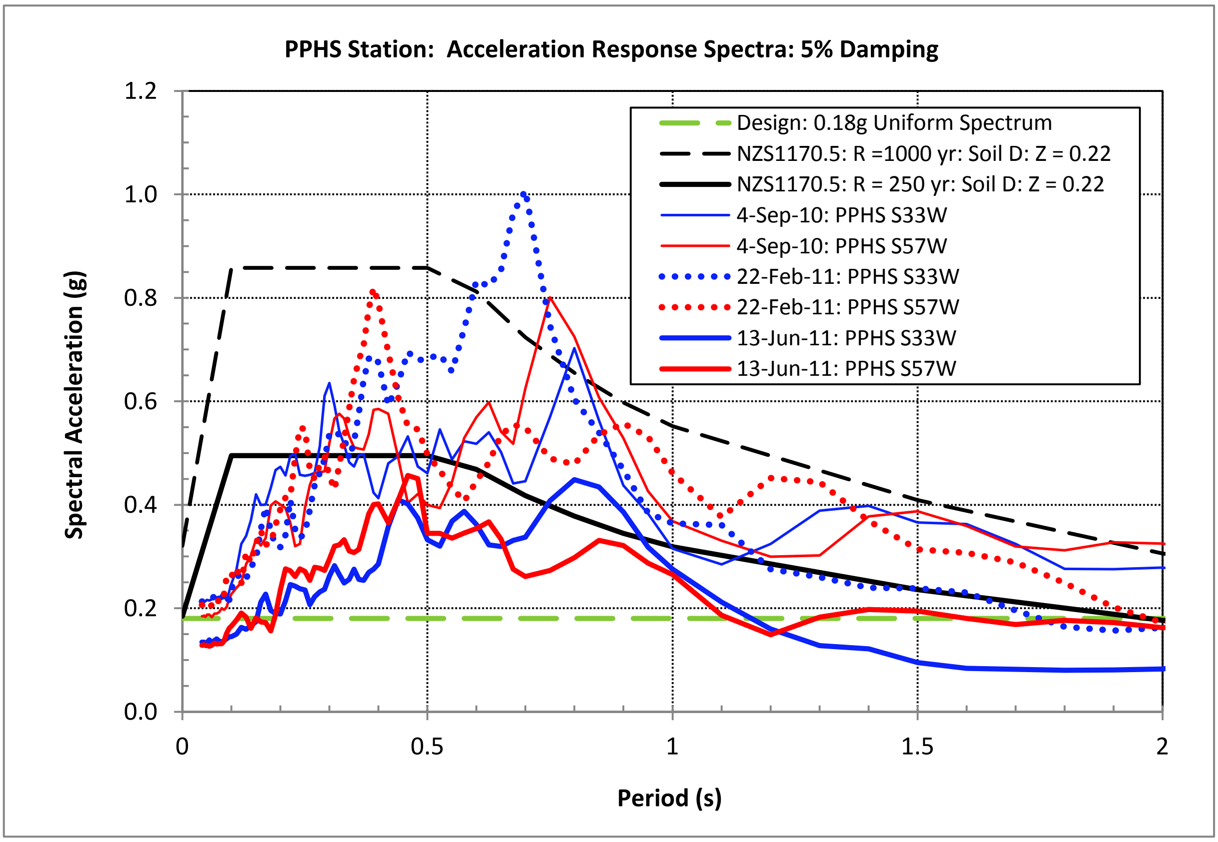
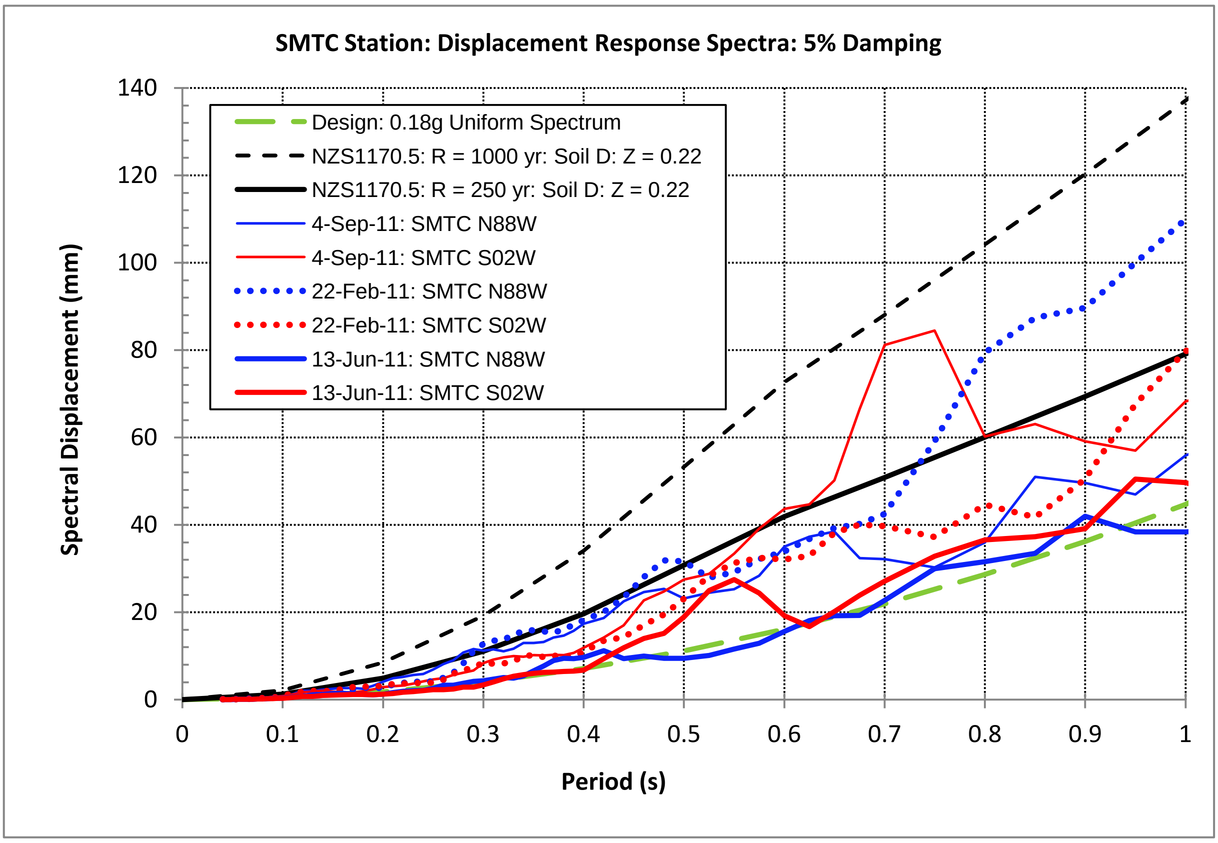


Figure B5: Acceleration response spectra from nearest SMA's to Group 3 bridges – see Table 2.



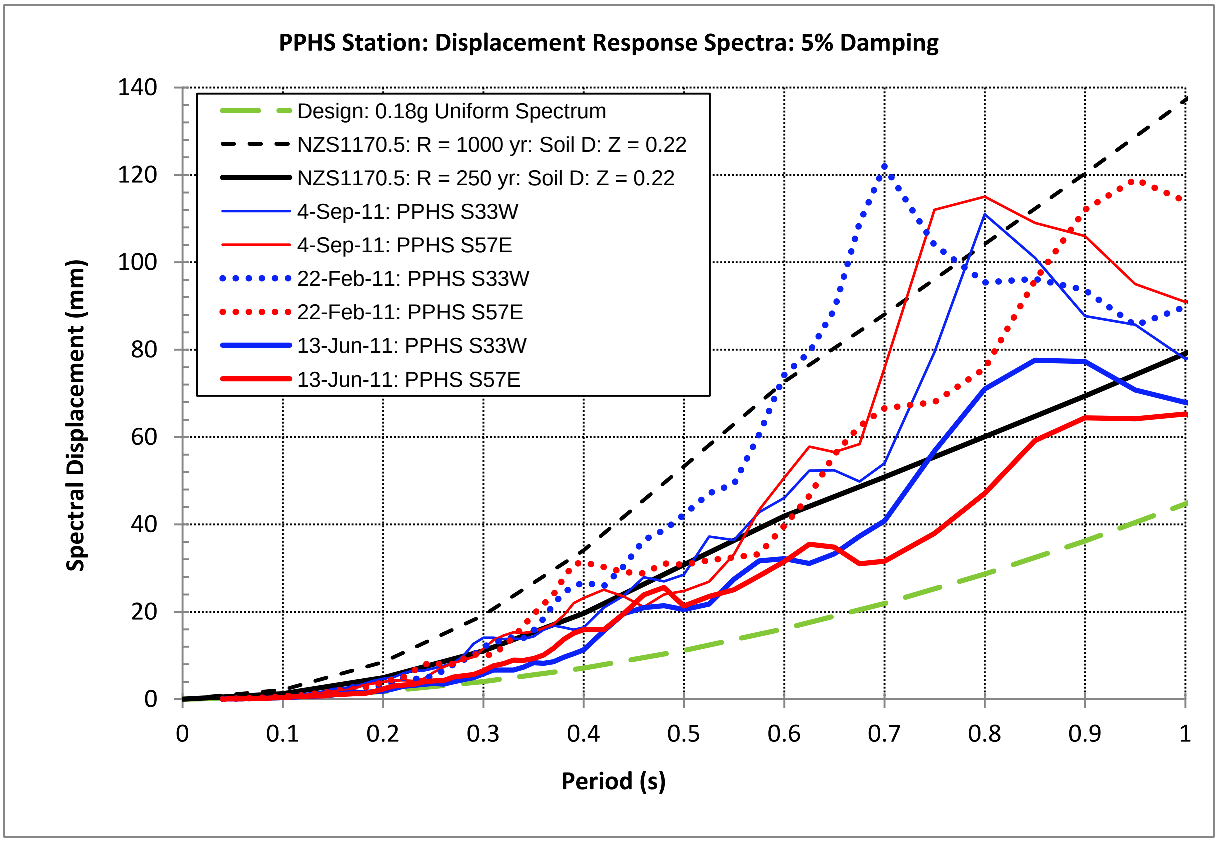


Figure B6: Displacement response spectra from nearest SMA's to Group 3 bridges – see Table 2.

SH74 STYX OVERBRIDGE NUMBER 1

Inspection by: J H Wood and H E Chapman **Date of Visit**: 16 March 2011

Details of SH1 Styx Overbridge Number 1 (Southbound Bridge)

SH, Region, RP & BSN	SH74; Region 6; RP 0/2.14; BSN 21.				
Location	7 km west north of Christchurch city centre.				
Distance to EQ Epicentre	37 km Darfield EQ 4 Sep 10; 14 km Chch EQ 22 Feb '11; 15 km Chch EQ 13 Jun '11.				
Distance to Fault Rupture	29 km Darfield EQ 4 Sep '10.				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1936. Year built: 1937.				
Geometry	Length: 176m	No Spans: 15	Max Span:12.2 m	Max. Ht: 4.8m	Width over deck slab: 13.4m
Alignment, Skew, Grade	Straight; Vertically curved; No skew except for a skew of 50º in span H-J over the railway.				
No of Lanes	2 Lanes plus 2 shoulders plus pedestrian/cycleway.				
Superstructure	Roadway section: 203 mm thick RC slab on 5 simply supported 356 mm wide x 432 mm RC downstand T beams. Cycleway section: 75 mm concrete topping on 300 mm deep simply supported PSC floor units.				
Piers	General piers: 4 – 457 mm square RC columns with flairs top and bottom. Piers vary in height from stubby to flexural. Skewed piers by railway: 5 – 457 mm square RC piers with flairs top and bottom. Piers 4 m high including flairs.				
Abutments	As piers but with very short columns on 1.2 m wide x 914 mm thick x 12.8 m long RC footing.				
Pier Foundation	General piers: 1.2 m wide x 914 mm thick x 12.8 m long RC footing. Skewed piers by railway: As for piers but 16.8 m long.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	Apparently high, with sand boils around the bridge site seen during the visit.				
Hold-down System	Piers: Integral construction.			Abutments: Integral construction.	
Linkage System	Piers: Integral construction. No linkage at "split" Pier D.			Abutments: Integral construction.	
Bearings flexible in shear?	Piers: No - Integral construction.			Abutments: No - Integral construction.	
General Condition	Good with no significant deterioration observed.				
Other Features	None.				

SH74 STYX OVERBRIDGE NUMBER 2

Inspection by: J H Wood and H E Chapman **Date of Visit**: 16 March 2011

Details of SH1 Styx Overbridge Number 2 (Northbound Bridge)

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH74; Region 6; RP 0/2.14; BSN 22.</td></tr>
<tr><td>Location</td><td colspan="5">7 km west north of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">37 km Darfield EQ 4 Sep 10; 14 km Chch EQ 22 Feb ’11; 15 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">29 km Darfield EQ 4 Sep ’10.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 2006. Year built: 2006.</td></tr>
<tr><td>Geometry</td><td>Length: 195m</td><td>No Spans: 9</td><td>Max Span: 21.9 m</td><td>Max. Ht: 6m.</td><td>Width over deck slab: 12.65m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight; Vertically curved, with overall height at piers varying between 4.6 and 6.2 m; No skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus 2 shoulders plus pedestrian/cycleway.</td></tr>
<tr><td>Superstructure</td><td colspan="5">800 mm thick prestressed concrete DHC units with no topping slab. Spans fixed to piers and sliding at abutments.</td></tr>
<tr><td>Piers</td><td colspan="5">Single 1500 mm dia RC column supported on 6.0 m square x 1.5 m thick RC pilecap.</td></tr>
<tr><td>Abutments</td><td colspan="5">1500 mm x 13.65 m wide x 1.5 m deep RC block carrying 250 mm thick RC backwall to deck level and supported on 3 x 1350 mm vertical RC piles per abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">4 x 1350 mm diameter RC piles per pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">No information seen.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">No information seen.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">Apparently high, with sand boils around the bridge site seen during the visi.t</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers:
No hold-downs.</td><td colspan="2">Abutments:
No hold-downs.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers:
Spans interlinked with 32 mm Reidbars – number not found on drawings.</td><td colspan="2">Abutments:
No linkages.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers:
No – spans on 150 x 15 mm thick rubber strip. Movement restrained in all directions by linkage bolts.</td><td colspan="2">Abutments:
Sliding bearings longitudinally; central 5 bearings guided, remaining 6 not.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no significant deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

Bridge No 2 (Northbound), looking South; built in 2006.
Central spans of nine total.

Bridge No 1 (Southbound), looking South; built in 1937.
Central spans of fifteen total. Note extension on far side, added in 2006.

SH74 STYX OVERBRIDGES

RESULTS OF INSPECTION

Damage Following 22 February Christchurch Earthquake (Not Inspected after 4 September 2010 Event.)

Only the spans north of the railway were inspected. The Regional Bridge Consultant reported that no damage was to be seen on the remaining spans to the south of the railway.

Bridge No 2 (Northbound, Built 2006)

- Cracking and spalling at the base of the third and fourth columns from the north end of bridge. The tops of the pile caps were visible at ground level. Fine circumferential cracks up to 2 m above the base of the fourth column and almost to the top of the third column (Figure ST1).
- Settlement of the approach fill relative to the north bridge abutment as evidenced by a strip of repaired pavement adjacent to the abutment.

Bridge No 1 (Southbound, Built 1937)

- Liquefaction sand boils under the Bridge but no obvious damage to this bridge, which is founded on spread footings (Figure ST2).

Figure ST1
Spalling in pier of Bridge No 2.

Figure ST2
Sand boils near Bridge No 1 pier.

Discussion

It was surprising to find that Bridge No 2, built in 2006, showed some spalling at its column bases, while Bridge No 1, built in 1937, showed no signs of damage in either its short stiff columns or its higher, quite slender columns.

Perhaps even more surprising was the presence of sand boils close to the foundations of Bridge No 1, which, from available drawings, apparently comprise quite shallow spread footings. The bridge shows no signs of settlement or distress.

Structure Response: The portal frame type piers of the No 1 Overpass make it very stiff in the transverse direction. Because of rotations expected in the spread footings it is rather less stiff in the longitudinal direction although passive resistance at the abutments provides a stiffening effect. Structural analyses have not been carried out for this bridge but based on other similar structures the periods in both principal directions are likely to be in the 0.2 to 0.3 second range.

The single stem columns of the No 2 Overpass give it greater flexibility in the transverse direction than the No 1 Overpass. A period of vibration for transverse response was calculated to be about 0.45 seconds. Because of the fully embedded large diameter piles at the abutments and the variation in column heights the bridge is much stiffer in the longitudinal direction. A period was not calculated for this direction but based on the transverse analysis it was estimated to be about 0.3 seconds.

SMTC the closest SMA to the Overpasses was located adjacent to the northern abutments. Spectral ordinates from the SMTC records in the period range relevant to the Overpasses were very similar for both the Darfield and Christchurch earthquakes with the component transverse to the bridges generally higher than the component directed along the longitudinal axes. Assuming simple SDOF response with 5% critical damping the maximum response accelerations from both earthquakes for the No 1 Overpass would have been about 0.3 g and 0.4 g in the longitudinal and transverse directions respectively. The corresponding response accelerations for the No 2 Overpass would have been about 0.4 g and 0.5 g.

Structure Strength Transversely: No structural analyses have been carried out for the No 1 Overpass. However, an approximate static analysis was carried out for transverse loading on the highest section of the No 2 Overpass. This indicated that the highest columns of the piers would reach their ultimate flexural strengths (at their bases) under a response acceleration of about 0.4 g. The observed cracking and spalling, which was predominantly on the transverse sides of the highest two columns was consistent with this prediction of flexural strength.

Structure Strength Longitudinally: No structural analyses have been carried out for either of the two Overpasses for loading in this direction.

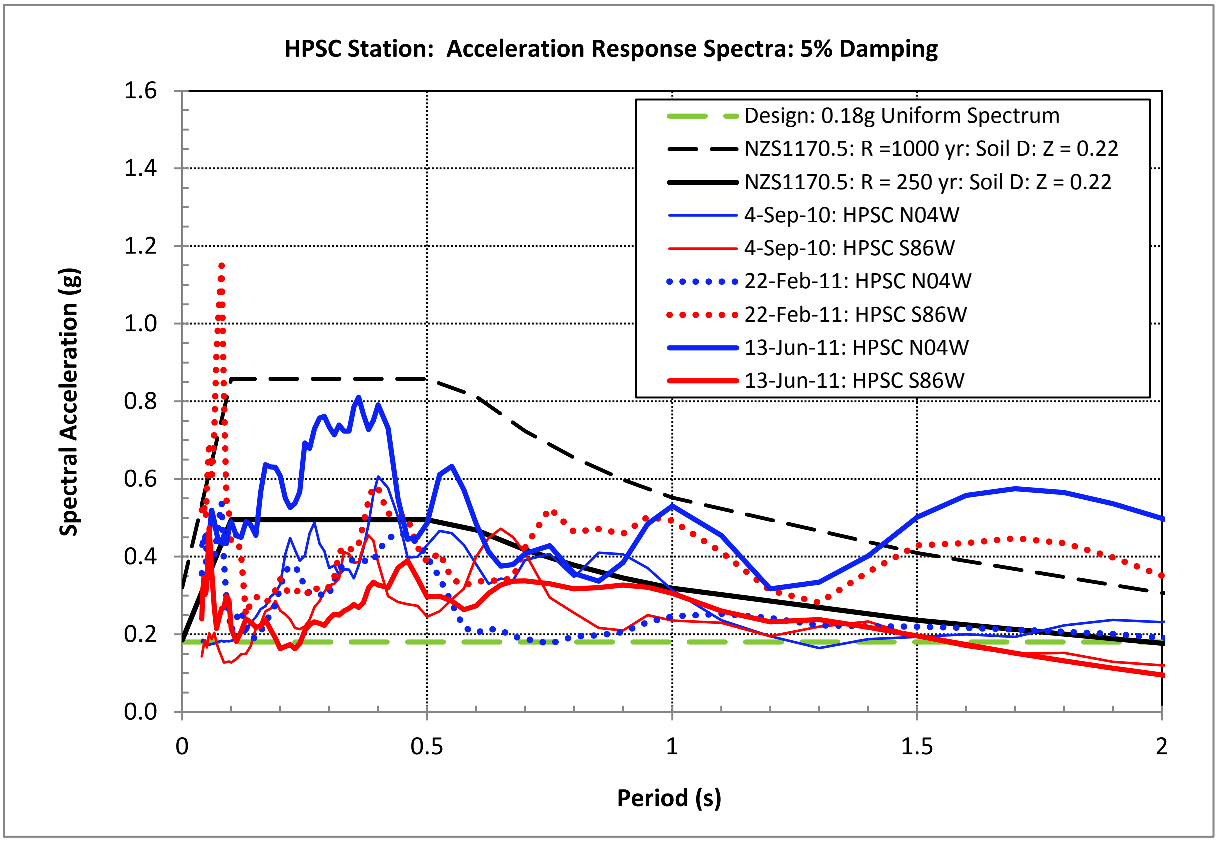
Conclusions

- The performance of the No 2 Overpass in the transverse direction was consistent with the predictions of a simple static analysis.
- The No 1 Overpass performed surprisingly well considering its age and the possibility that liquefaction could have caused differential settlement of the piers, although no structural evidence of settlement has been found.
- A more detailed assessment of the performance of the two Overpasses in the earthquakes would be warranted. The close proximity of the SMTC recording station to the bridges provides a reliable estimate of the input ground motions required for a back analysis assessment of their earthquake performance.

GROUP 4 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA



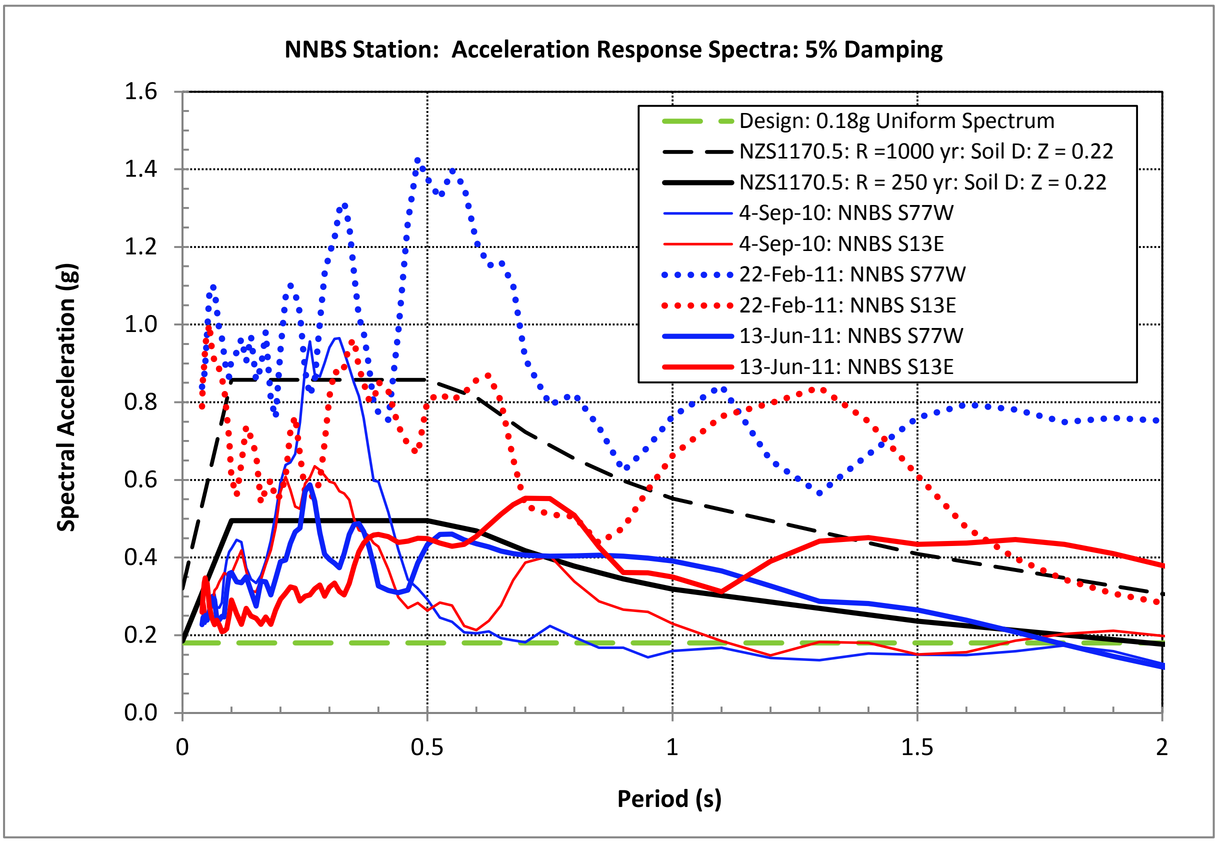
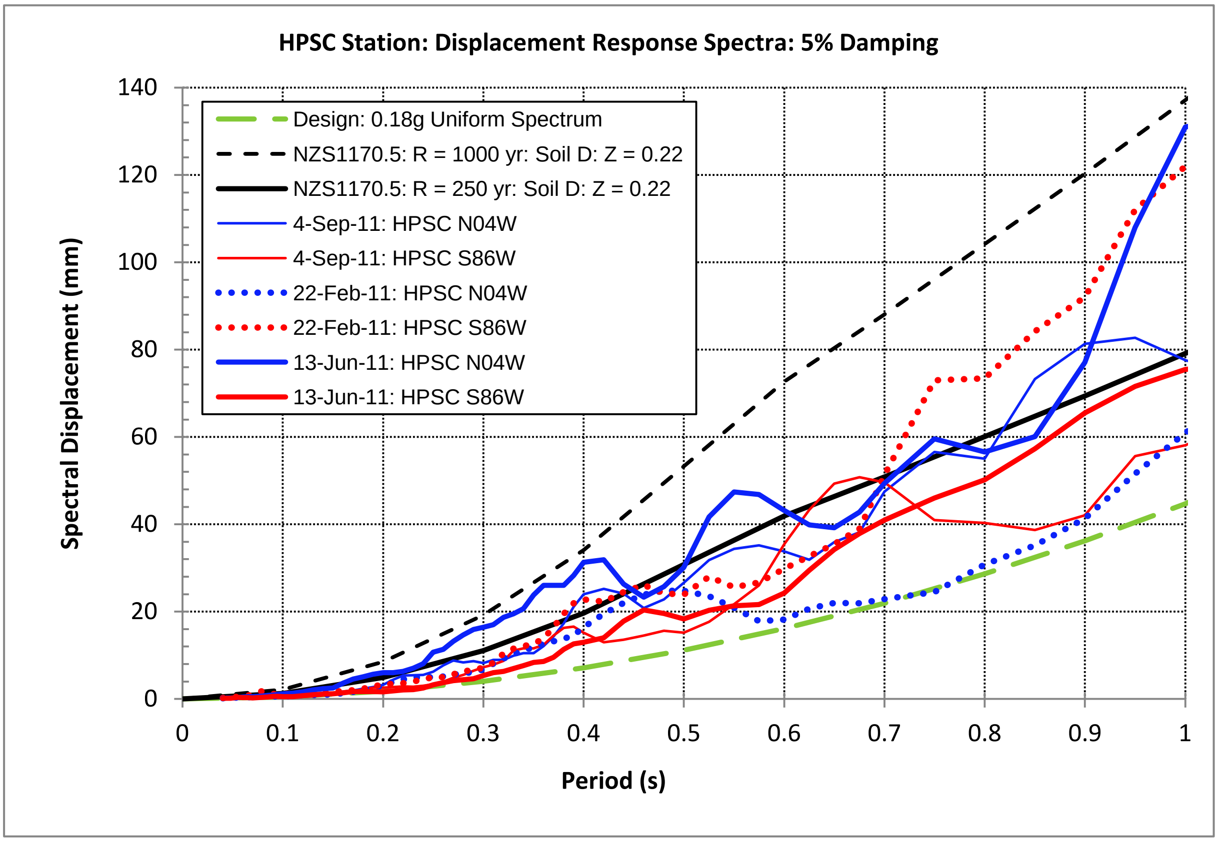


Figure B7: Acceleration response spectra from nearest SMA's to Group 4 bridges – see Table 2.



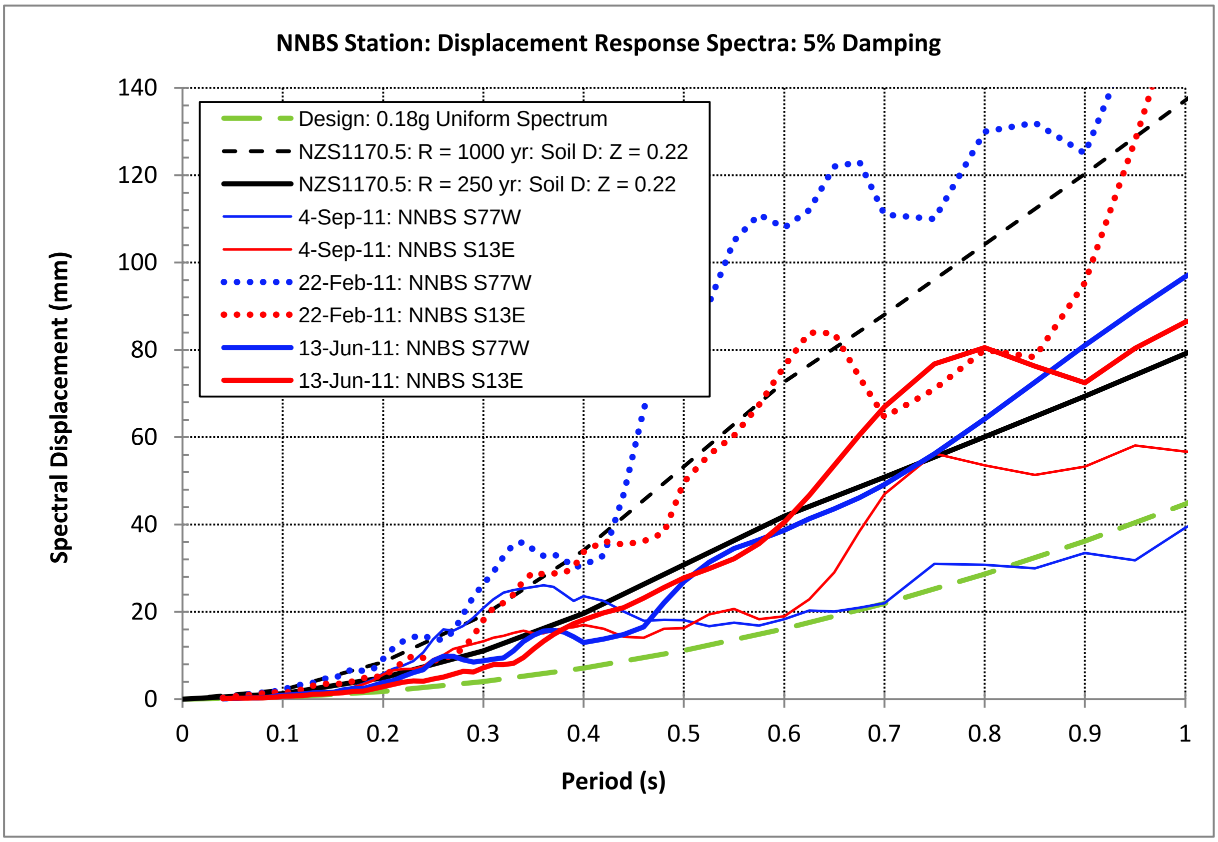


Figure B8: Displacement response spectra from nearest SMA's to Group 4 bridges – see Table 2.

SH74 ANZAC DRIVE BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010 and 16 March 2011
22 September 2010 and 3 March 2011

Details of SH74 Anzac Drive Bridge

SH, Region, RP & BSN	SH74; Region 6; RP 0/11.8; BSN 119.				
Location	7 km east north-east of Christchurch city centre.				
Distance to EQ Epicentre	43 km Darfield EQ 4 Sep ’10; 9 km Chch EQ 22 Feb ’11; 8 km Chch EQ 13 Jun ’1.1				
Distance to Fault Rupture	28 km (Darfield EQ 4 Sep ’10).				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1999. Year built 2000.				
Geometry	Length: 49m.	No Spans: 3	Max Span: 18.6m	Max. Ht: 5m.	Width over deck slab: 21.3 m
Alignment, Skew, Grade	Straight and practically level. 13º skew.				
No of Lanes	4 Lanes with central barrier, plus footpath on west side.				
Superstructure	18 x 650 mm deep x 1144 mm wide DHC PSC standard units per span.				
Piers	4 x 1 m x 1 m trapezoidal RC pier columns carrying 1.29 m deep x 1 m wide precast RC crosshead beam notched at half depth to 400 mm wide to provide 300 mm bearing for precast deck beams. Knee joints at outer columns include steel plate units to connect columns to crossbeams.				
Abutments	North: 1 m thick x 1.2 m deep RC pile cap 24.2 m long, carrying a 300 mm thick x 650 mm deep backwall and 3 m long settlement slab. All supported on 16 vertical steel H piles. South: 1 m thick x 1.2 m deep RC pile cap 22.4 m long, carrying a 300 mm thick x 650 mm deep backwall and 3 m long settlement slab. All supported on 15 vertical steel H piles.				
Pier Foundation	4 x 1.5 m diameter steel shelled RC bored/driven piles per pier.				
Soils, Borehole info.	2 boreholes recorded to 28 metres – one on each bank - plus CPTs to 18 metres. Generally loose to medium sands or silty sands down to very dense sands at about 16 metre depth.				
Depth of Sediment	>28 metres.				
Liquefaction Risk	Extensive liquefaction evident in surrounding areas of river bank.				
Hold-down System	Piers: No vertical tie-down but dowel action of linkages through pier cap provides some tie-down.		Abutments: No vertical tie-down but dowel action of linkages through pier cap provides some tie-down.		
Linkage System	Piers: 24 mm diameter linkage bars placed between each DHC deck unit.		Abutments: 24 mm diameter linkage bars placed between each DHC deck unit and anchored into abutments.		
Bearings flexible in shear?	Piers: No – hollow core units sit on 150 x 15 mm thick rubber strips and are anchored through a shear key.		Abutments: No – hollow core units sit on 150 x 15 mm thick rubber strips and are anchored through a shear key.		
General Condition	Good with no deterioration observed.				
Other Features	Precast concrete plinths along sides, carrying decorative metal artwork.				

SH74 Anzac Drive Bridge, looking north-west – 14 October 2010.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following points were noted:

- There had been soil liquefaction at the site that had caused significant ground subsidence and cracking in the topsoil. The bridge piers are founded on 1.5 m diameter piles extending to a depth of about 22 m. Because of their depth and robustness damage to them from the liquefaction would not be expected.
- The abutments are founded on 310 UC piles, which would be vulnerable to damage from lateral spreading but there was no evidence that they had been displaced.
- A concrete balustrade on the walkway under the southern abutment had settled about 50 mm relative to the precast concrete abutment apron, which appeared to be attached to the abutment structure and had not settled.
- Sections of pavement on the walkway near the southern abutment had been damaged by ground subsidence and lateral spreading. Ejected sand was lying on the ground within 100 metres of the bridge.
- The bridge is about 100 metres from the Hulverstone Drive Pumping Station, which was damaged by liquefaction. It appeared that the underground tank or foundation associated with the pump house had been lifted by flotation in liquefied soil. The nearest strong motion recorder to the bridge is housed in the pumping station and liquefaction may have reduced the recorded peak accelerations.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- Pressures from liquefaction induced lateral spreading of the approaches had caused the abutment piles and beam seatings to rotate about 5^{o} (Figures AD1 and AD2).
- There was cracking and spalling at the beam-column joints in both piers. Damage was more pronounced at the outer of the four columns in each pier. The columns in the southern pier had significant permanent displacement with up to a 2^{o} tilt (Figures AD3 and AD4) apparently caused by liquefaction lateral spreading.
- There was significant lateral spreading and settlement of the approach pavements and embankments. The settlements near the abutments were of the order of 300 mm.
- The concrete walkway structures under each end of the bridge had moved towards the river about 500 mm at the south end (Figures AD1 and AD2) and about 300 mm at the north end.

Figure AD1

Figure AD2

The south abutment showing the extensive ground settlement, lateral spreading, rotation of the abutment and displacement of the walkway. The north abutment showed similar damage.

Figure AD3
Spalling damage at the beam/column joints in the northern pier.

Figure AD4
East column of northern pier.

Photographs taken 16 March 2011.

Discussion

Structure Response: In the longitudinal direction the bridge is rigidly linked to 2.1 metre high abutments, which have 3 metre long settlement slabs tied to them with closely spaced reinforcement. Passive resistance from the abutment back face and the closely spaced abutment piles stiffens the bridge in the longitudinal direction, producing a short longitudinal period of vibration estimated to be in the range of 0.25 to 0.3 seconds. The portal frame piers founded on large diameter piles are very stiff in the transverse direction giving an estimated period of between 0.13 to 0.2 seconds for the first transverse mode of vibration. Based on the mean of the four horizontal components of ground motion from the two nearest SMA's and assuming simple SDOF response with 5% critical damping the response accelerations in the Darfield earthquake would have been about 0.45 g and 0.35 g in the longitudinal and transverse directions respectively (see Figure 5). Corresponding displacement responses would have been about 10 mm and 5 mm. In the Christchurch earthquake the response accelerations were higher than in the Darfield earthquake reaching 0.6 g and 0.5 g in the longitudinal and transverse directions respectively.

The bridge is located about 100 m from the HPSC recording station and although the records from this station were affected by liquefaction, because of its closeness to the bridge the response spectra from the HPSC records may give a better representation of the ground motion at the bridge than the means of the components from the two nearest SMA's. Reference to the response spectra from HPSC (Figure B7) shows that in the longitudinal the response acceleration in the Darfield earthquake might have been a little greater than 0.4 g and in the Christchurch earthquake about 0.33 g. Conversely, in the transverse direction the response acceleration in the Christchurch earthquake was about 0.3 g, exceeding the response in the Darfield earthquake where the spectral acceleration was probably about 0.25 g. The SMA recording directions aligned with the principal directions of the bridge making it possible to make more reliable estimates for the response in the two principal directions. Although the intensity of the horizontal ground shaking at the HPSC station was similar in the two earthquakes the vertical component was much stronger in the Christchurch earthquake reaching a PGA of 0.85 g compared to 0.13 g in the Darfield earthquake.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier model assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans, and 50% of the inertia force from the pier. The analysis indicated that the portal frame piers would have remained elastic under the estimated response acceleration in the Christchurch earthquake of 0.3 g. Under combined gravity and earthquake loading the flexural strength of the tops of the centre columns in the portal frames appeared to be the most critical items. These sections would reach their ultimate flexural strength capacities at a response acceleration of about 0.55 g. In the Christchurch earthquake the mean response acceleration computed from the two nearest two SMA's reached about this level although the response acceleration from the HPSC station was significantly less. The high vertical accelerations and pressures on the piers from lateral spreading may have contributed to the damage observed at the beam-column joints in the pier portals.

Structure Strength Longitudinally: A simplified static analysis was carried out for the longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the relative stiffness of the abutment structures for tension and compression direction loads, and the stiffness of the piers. The analysis indicated that at the abutment being pulled away from the soil the linkage bars located between the hollow core deck units would reach their yield strength at a response acceleration of about 0.55 g. Under increasing response accelerations a greater proportion of the load from this abutment would be transferred to the piers and the other abutment (pushed against the soil). At a response acceleration of about 1.1 g and a longitudinal displacement of about 25 mm the base sections of the pier columns would reach their ultimate flexural strength capacities. The maximum passive force on the abutment walls would be reached at about the same response acceleration level. The abutment upstand walls (in contact with the deck units) are 300 mm thick with a large amount of vertical reinforcement and have sufficient strength to transmit the superstructure loads corresponding to the full passive pressure on the abutment walls and piles. The pier columns are detailed with confinement reinforcement and would perform satisfactorily if subjected to much larger longitudinal displacements than yield level.

Conclusions

- Based on the results of our analyses significant damage to the bridge from the inertia loads experienced in either of the earthquakes would not have been expected. The liquefaction induced lateral spreading and the high vertical accelerations in the Christchurch earthquake probably contributed to the damage to the piers observed following this event.
- Although liquefaction was very evident at the site following the Darfield earthquake, the amount of lateral spreading was small and did not appear to damage the abutment piles. In contrast, significant lateral spreading occurred in the Christchurch earthquake, with the cumulative effects of the earthquakes resulting in serious damage to the abutment piles.
- Although the intensity of the horizontal ground shaking in the Christchurch earthquake did not appear to be much greater at the site than in the Darfield earthquake, the lateral spreading and structural damage changed quite markedly. This may have been exacerbated by the cumulative lateral spreading displacements from the two earthquakes. Because the difference in shaking intensity in the two events bounded the damage threshold level the performance of the bridge is of particular interest for more detailed study.

BRIDGE STREET BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Dates of Visits: 14 October 2010 and 16 March 2011
22 and 23 September 2010 and 3 March 2011

Details of Bridge Street Bridge

SH, Region, RP & BSN	Local Authority bridge – not on state highway.				
Location	7 km east of Christchurch city centre.				
Distance to EQ Epicentre	44 km Darfield EQ 4 Sep '10; 7 km Chch EQ 22 Feb '11; 5 km Chch EQ 13 Jun '11.				
Distance to Fault Rupture	29 km (Darfield EQ 4 Sep '10).				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1978. Year built 1980.				
Geometry	Length: 65m	No Spans: 3	Max Span: 22.03m	Max. Ht: 8m.	Width over deck slab: 15.2m
Alignment, Skew, Grade	Straight and practically level. Deck slopes 140 mm from west to east. 25º skew.				
No of Lanes	2 Lanes plus 2 shoulders plus 2 footpaths.				
Superstructure	200mm composite reinforced concrete deck on 5 PSC I beams per span, 1.6 m deep.				
Piers	1.3 m thick x 5.2 m x 6.2 m octagonal RC pile cap supporting a 1.8 m x 1.8 m octagonal RC column x 3.6 m high, which carries a 1.8 m wide x 14.7 m long RC hammerhead that tapers from 2 m to 800 mm depth.				
Abutments	1.9 m wide x 1.5 m deep x 13.7 m long RC abutment block carrying a 350 mm x 2.1 m high RC backwall and wing walls, all supported on 10 x 450 mm raked octagonal PSC piles.				
Pier Foundation	12 x 450 mm raked octagonal PSC piles.				
Soils, Borehole info.	3 borehole records are shown on Drawing Sheet 2, driven to a depth of about 18 to 20 m below bank level. These showed loose to medium dense sands to a depth of 6 to 8 metres and increasing density below that depth.				
Depth of Sediment	>20 metres.				
Liquefaction Risk	Yes – as demonstrated by damage to abutments caused by liquefaction pressures.				
Hold-down System	Piers: None, but 6 x 127 mm square vertical RHS shear keys per pier are cast into pier cap and protrude into deck diaphragms.			Abutments: None.	
Linkage System	Continuous superstructure between abutments.			Abutments: 6 x 50 mm diameter linkage bolts per abutment, through deck diaphragm and abutment backwall.	
Bearings flexible in shear?	Piers: Minimal, due to 127 mm square RHS shear keys connecting deck to piers. RHS is wrapped in 25 mm thick rubber all round. Bearings are 406 x 280 x 115 mm elastomeric – 1 per beam end per span (i.e.6 total per span end).			Abutments: Yes. Bearings are 406 x 280 x 115 mm elastomeric – 1 per beam end per span (i.e.6 total per span end).	
General Condition	Good except for severe earthquake damage to abutments and possible damage to the piers below water or ground level.				
Other Features	None.				

Bridge Street Bridge, looking west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

Bridge Street Bridge was the most seriously damaged of the bridges inspected. In summary the visible damage included:

- Extensive liquefaction induced lateral spreading of the ground and damage to approach embankments and gabion walls. Reports of ongoing subsidence and lateral spreading even a few weeks after the main 4 September event.
- Settlement of the eastern approach embankment. A photograph taken a few days after the earthquake showed a step of approximately 100 mm at the abutment backwall.
- Lateral spreading and subsidence beneath the pile-supported abutment seating beams (Figure BS1). A maximum settlement of about 500 mm was observed at the west abutment and about 300 mm at the east abutment. The piles comprise 450 mm wide octagonal piles arranged in forward and back raked pairs.
- Closing of the gaps at the abutment deck joints (Figure BS2). There was tight contact at both the north-east and north-west corners of the deck and spalling damage to the deck end at the north-west corner. The design joint gaps were 50 mm and creep and shrinkage would have increased this distance, so the sum of the forward movement of the abutments was more than 100 mm. Most of this movement may have occurred at the west abutment which had rotated more than the east abutment.

- Lateral spreading of the abutment slopes leading to rotation of the abutments about their horizontal transverse axes. The west abutment was inclined at an angle of about 0.1 radians (6^o) measured relative to the ends of the beams at the north side of the abutment. This rotation was apparently caused by liquefaction of sand layers below the water table producing lateral spreading towards the river and resulting in large lateral pressures on the raked piles. The tops of some of the piles supporting the west abutment had fine flexural cracks near their intersection with the bottom of the abutment seating beam.
- Apparent rotation of the superstructure about a vertical axis (Figure BS3). Transverse misalignments of the edge of the deck and abutment wing walls were about 30 mm and 50 mm at the east and west abutments respectively. The abutments had clearly moved forward so some of the misalignment may have resulted from their movement rather than that of the superstructure. The direction of the apparent rotation was consistent with the direction of any shear force that would develop as a result of the closing of the abutment joint constructed on a 25^o skew.
- Sliding of the 115 mm thick elastomeric bearing pads supporting the beams at the abutments (Figure BS4). The bearings had moved back towards the abutment backwalls with a maximum movement of about 100 mm at the north-west corner. Presumably most of the sliding of the bearings was related to the forward displacement and rotation of the abutment structures.
- The six 50 mm diameter linkage bolts at each abutment were loose (Figure BS5). This was clearly related to the forward movement of the abutments.
- The 1800 mm wide octagonal column of the west pier had a clearly visible horizontal crack located in the tidal range about 1600 mm above the top of the pile cap. This crack was covered by water during the inspection but the photo published by the NZ Natural Hazard Platform Bridge Research Group indicated that it was perhaps about 1 mm wide and was more open on the north side. Clearly the longitudinal reinforcement had yielded near the base of the column. Moments from both inertia loads and the possible rotation of the bridge about a vertical axis may have exceeded the section yield strength. The crack may have been held open by the permanent rotational restraint applied at the abutment joint contacts.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- Additional lateral spreading and rotation occurred at both abutments (Figure BS6). The rotation at the west abutment had increased from 0.1 to 0.16 radians (6^o to 9^o) (Figure BS7). The total rotation at the east abutment was about 0.1 radians (6^o).
- The apparent rotation of the superstructure about a vertical axis had increased to produce offsets in the deck edges at either abutment of about 150 mm (Figure BS8).
- Deformation and sliding of the beam elastomeric bearings on the abutments had increased.
- There was increased impact damage at deck level at the abutments and damage to the deck joint rubber seals.

Discussion

Soil Effects on Abutment Piles: Bore logs included with the bridge drawings show that there are medium dense sand layers extending for a depth of about 5 metres below the water table, which is about 5 metres below the road level. These sand layers have Standard Penetration Test N values in the range of 12 to 19 and would be susceptible to liquefaction in the earthquake. The slopes under the bridge and on the approaches are inclined at about 2 horizontal: 1vertical, so slope failures may have occurred without a fully liquefied condition

Figure BS1
Settlement at east abutment.

Figure BS2
Joint closed at north-west corner.

Figure BS3
Horizontal rotation at north-east corner.

Figure BS4
Bearing displacement at east abutment.

Figure BS5
Loose linkage bolts at east abutment.

Figure BS6
Lateral spreading on west approach.

Figure BS7
Settlement and rotation at west abutment.

Figure BS8
Horizontal rotation at south-west corner.

in the soil layers. A simple analysis of the abutment raked pile system with passive pressures from lateral spreading of the sand layers applied to the piles was carried out to estimate their performance. Although the soil input parameters for such an analysis are very uncertain it was possible to calibrate the model against the measured rotation at the abutment level. The results indicated that plastic hinges would develop in the piles within the liquefied layer and near the interface with the dense sand which formed the founding layer beneath the liquefied zone.

Structure Response: The bridge has a continuous deck over the piers and is quite rigidly connected to them by RHS shear keys sheaved with 25 mm thick rubber. The 115 mm high elastomeric bearings supporting the beams at the abutments are relatively flexible and assuming that the deck joints at the abutments maintain their design gap of 50 mm the superstructure response is not strongly influenced by the stiffness of the abutment structures. The continuous deck results in very effective diaphragm action with a similar but relatively small part of the superstructure inertia loads being transferred to the abutments regardless of the direction of loading. Periods of vibration are similar in both the transverse and longitudinal direction and were estimated to be in the range of 0.3 to 0.4 seconds. Assuming simple SDOF response with 5% critical damping the response accelerations in both the longitudinal and transverse directions would have been about 0.45 g and 0.6 g in the Darfield and Christchurch earthquakes respectively. The corresponding displacement response in the Darfield earthquake would have been about 15 mm, which slightly exceeds the displacement that would cause yielding of the piers, as indicated by the analyses described below.

Following closure of the abutment joint gaps the bridge would not have responded in the manner assumed in the response analyses. If the movement of the abutments was caused mainly by liquefaction then the gaps probably closed after the strongest ground shaking had occurred. In this case the analyses would provide a good estimate of the performance. After the gaps closed the periods of vibration would have been reduced and the horizontal inertia load would have been carried mainly by the abutments, particularly for loads in the longitudinal direction.

Structure Strength Transversely: A simplified static analysis was carried out for the transverse direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the relative stiffness of the piers and abutment bearings. The deck was assumed to act as a rigid diaphragm spanning between the abutments. The analysis indicated the base sections of the pier columns would reach their ultimate flexural strength capacities at response acceleration of about 0.3 g. Displacements of the superstructure at yield in the pier columns were estimated to be in the range of 8 to 12 mm.

Structure Strength Longitudinally: A simplified static analysis was carried out for longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the relative stiffness of the piers and abutment bearings. The analysis indicated the base sections of the pier columns would reach their ultimate flexural strength capacities at response acceleration of about 0.4 g. The better performance in the longitudinal direction is a consequence of the lateral inertia load being applied at the beam bearing level rather than at the centre of gravity of the superstructure. Displacements of the superstructure at yield in the pier columns were estimated to be in the range of 8 to 12 mm. The pier columns are detailed with confinement reinforcement and would perform satisfactorily if subjected to much larger longitudinal displacements than yield level.

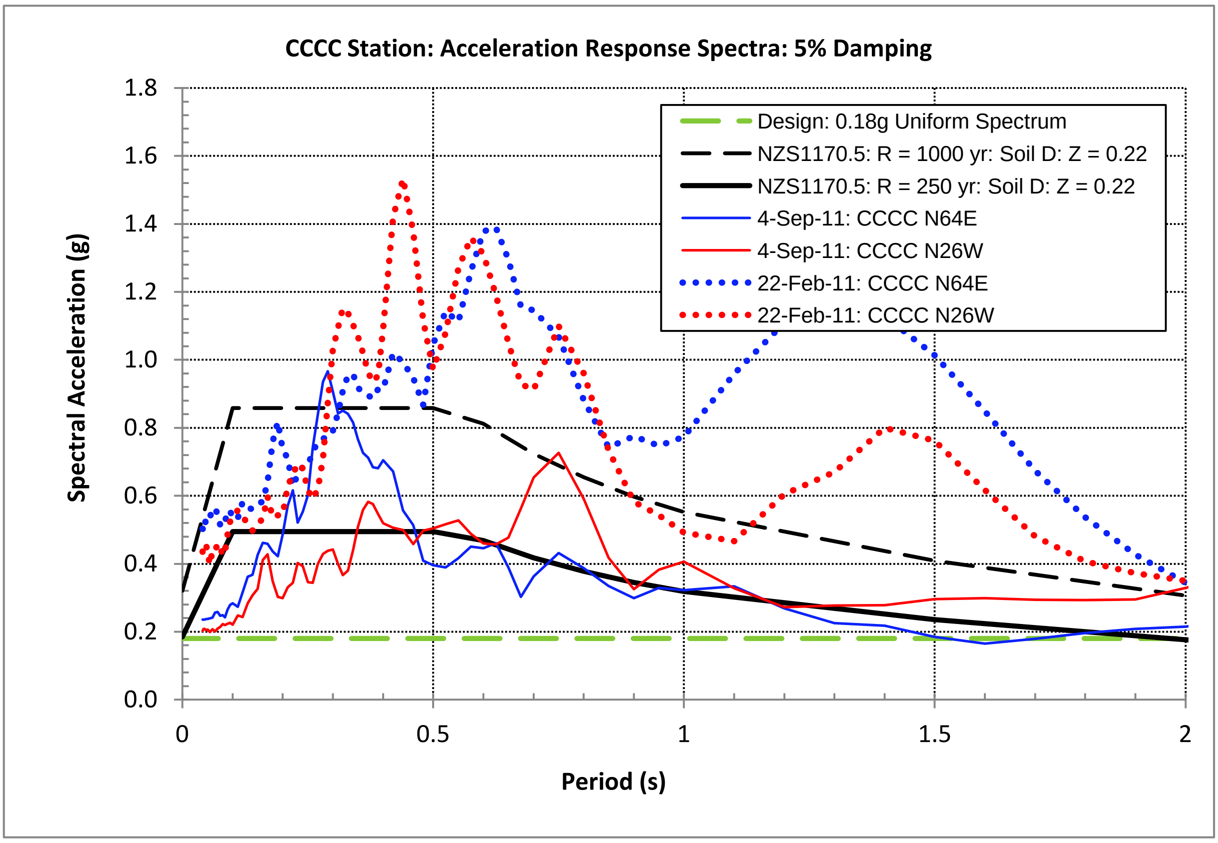
Conclusions

- The prestressed concrete piles that support the abutments have been bent by soil movements and are probably significantly cracked below ground level, some 10 metres below road level. The Darfield earthquake caused the initial large deformations and cracking damage to the abutment piles. The pile deformations increased significantly in the Christchurch earthquake which probably produced stronger shaking at the site than the Darfield earthquake.
- The analyses indicate that significant cracking and spalling may have occurred at the bottoms of the pier columns prior to the joint gaps closing. However, the bases of the pier columns and their pile caps are covered by water at low tide. The columns’ long-term integrity should be assured by inspection and appropriate repair, which would require coffer dams approximately 3.5 metres deep to be installed on top of the pilecaps.

GROUP 5 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA



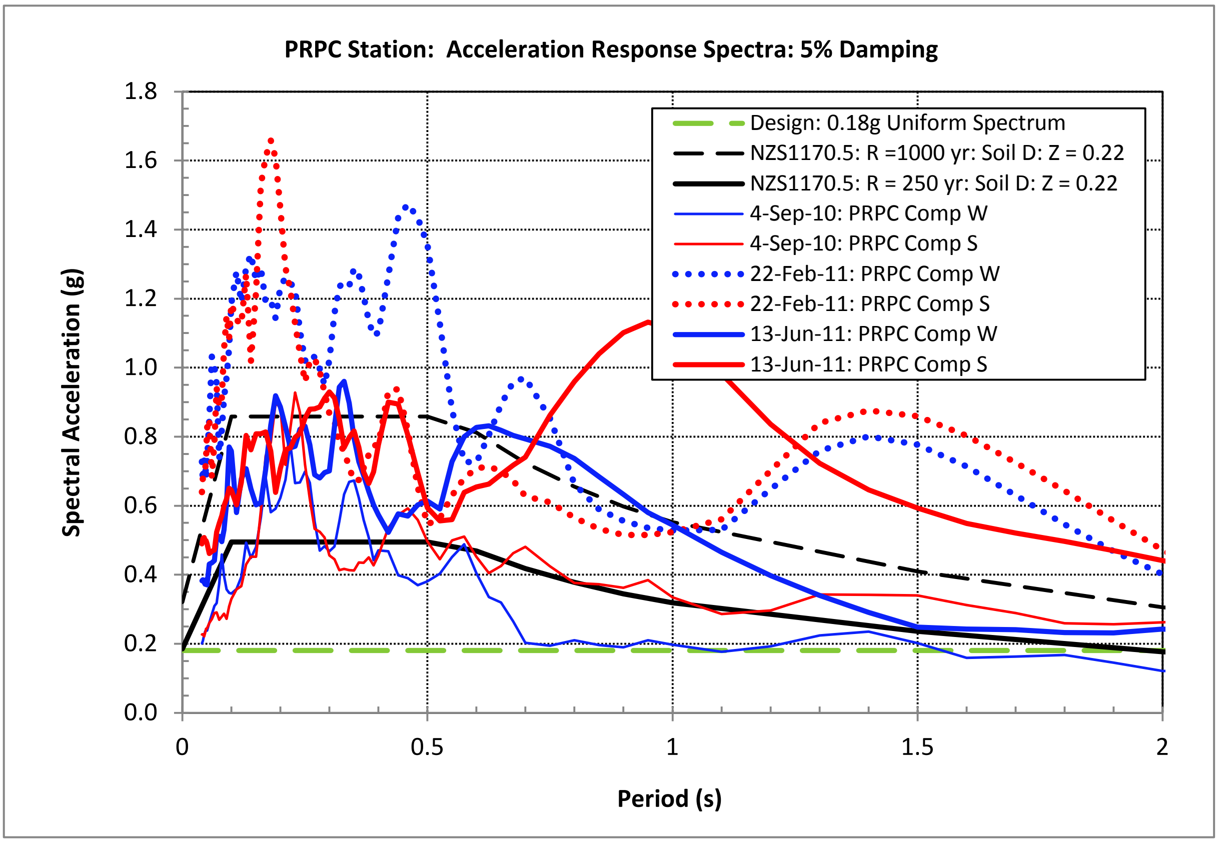
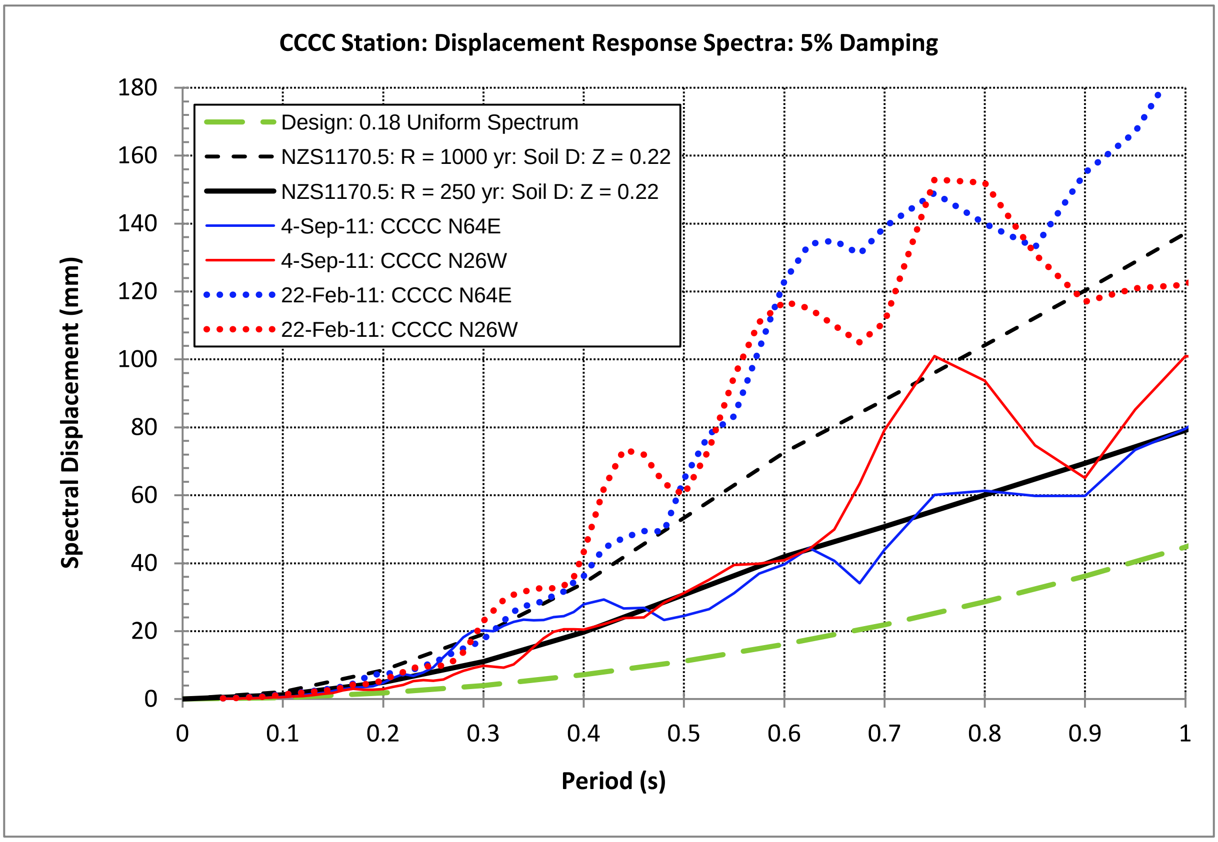


Figure B9: Acceleration response spectra from two nearest SMA's to Group 5 bridges - see Table 2.



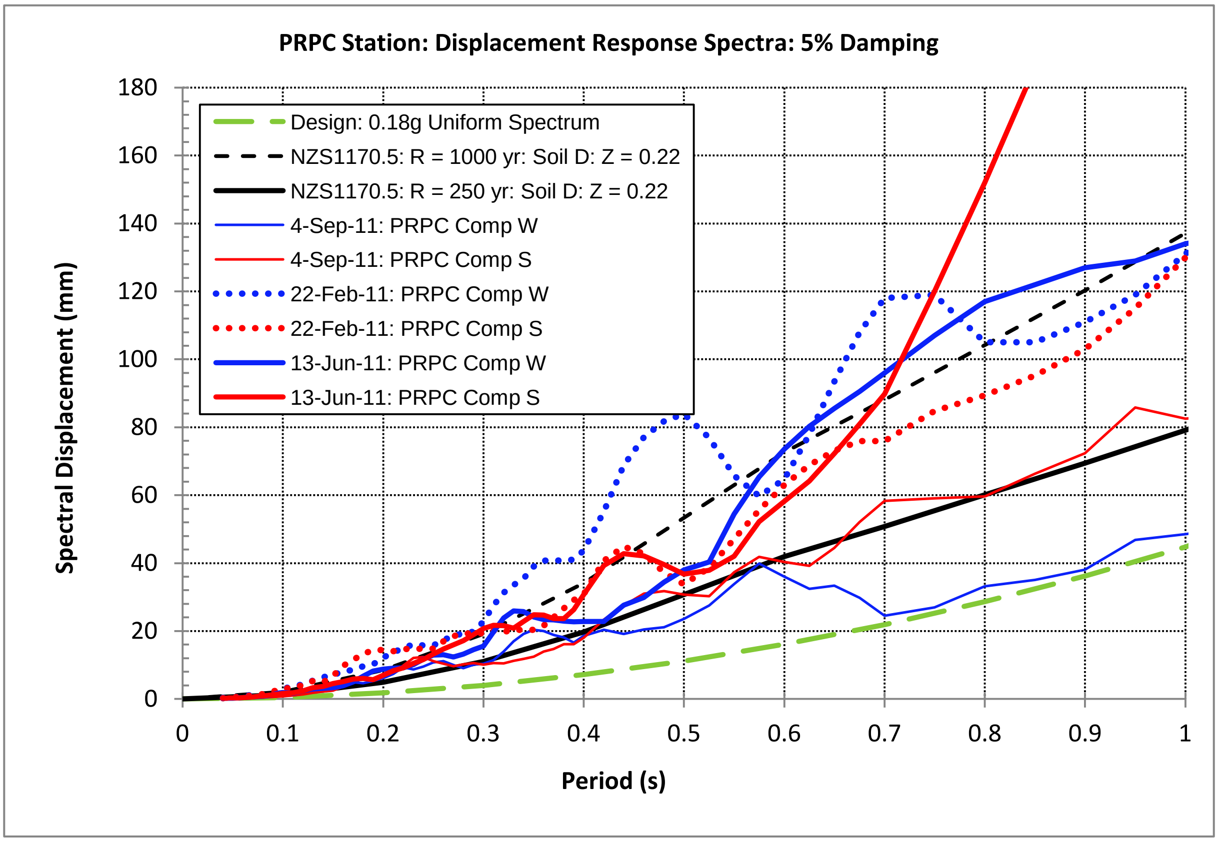


Figure B10: Displacement response spectra from two nearest SMA's to Group 5 bridges - see Table 2.

SH73 HEATHCOTE RIVER BRIDGE (OPAWA)

Inspection by: J H Wood and H E Chapman **Date of Visits**: 16 March 2011

Details of SH74 Heathcote River Bridge (Opawa)

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH73; Region 6; RP 3/0.68; BSN 33.</td></tr>
<tr><td>Location</td><td colspan="5">4 km south-east of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">42 km Darfield EQ 4 Sep ’10; 2 km Chch EQ 22 Feb ’11; 4 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">24 km Darfield EQ 4 Sep ’10.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1989. Year built 1990.</td></tr>
<tr><td>Geometry</td><td>Length: 58 m</td><td>No Spans: 3</td><td>Max Span: 23.6m</td><td>Max. Ht: 6 m.</td><td>Width over deck slab: 11.13 m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and practically level (on minor vertical curve). No skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus shoulders.</td></tr>
<tr><td>Superstructure</td><td colspan="5">250 mm composite reinforced concrete deck on 4 PSC precast I beams 1.6 m deep per span.</td></tr>
<tr><td>Piers</td><td colspan="5">1.5 m x 1.5 m octagonal RC column supporting 1.8 thick x 1.75 max depth tapered cantilever cap x 8.7 m long.</td></tr>
<tr><td>Abutments</td><td colspan="5">1.5 m wide x 1.1 m deep x 11.13m long RC pile cap carrying 500 mm thick x 2.05 m high RC backwall. All supported on 10 vertical 200 x 146 kg/m steel UBP per abutment. Actual level of pile toes unclear from drawings. Settlement slab attached to each abutment.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">1 – 2 m diameter x 14 m deep RC steel-cased pile, carried on 7 vertical 200 x 146 kg/m steel UBP per pier. Actual level of pile toes unclear from drawings.</td></tr>
<tr><td>Soils, Borehole info. (Ref drawing Sheet 2)</td><td colspan="5">West bank: soft sands & silts to 25m; firm to compact gravels to 31m end of borehole.
East bank: soft sands and clays down to 18.3 m end of borehole.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">No information seen.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: None.</td><td colspan="2">Abutments: None except linkage bolts in shear.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: Span linkages provided by deck longitudinal reinforcing bars made continuous and common diaphragm for beams over the piers.</td><td colspan="2">Abutments: 6 x 42 mm linkage bolts through beam diaphragm and each abutment backwall.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: No – RHS shear keys into diaphragms prevent translation. Bearings are 380 x 300 x 115 thick multilayer elastomerics – 1 each per beam end.</td><td colspan="2">Abutments: No – tight linkage through diaphragms prevent translation. Bearings are 380 x 300 x 115 thick multilayer elastomerics – 1 each per beam end.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH73 Heathcote River Bridge (Opawa) looking south-east.

RESULTS OF INSPECTIONS

The bridge was not visited after the 4 October 2010 Darfield earthquake as there were no reports of damage at that time.

Damage Observed Following the 22 February 2011 Christchurch Earthquake

No major structural damage to the bridge was observed during the inspection but the following points were noted:

- Significant longitudinal movement of the bridge was indicated by damage to the approach pavement kerbs, cracking in the pavement at the back of abutment walls, and gaps in the soil at ground level on the timber posts supporting the approach guardrails.
- Settlement of the concrete aprons under both abutments.
- A small rotation of the south-east abutment as shown by the gaps at the ends of the beams.
- The outer pair of elastomeric bearings of the four at the north-west abutment showed residual shear displacements of 15 to 20 mm each, with the tops displaced outwards in opposite directions, indicative of transverse “stepping” as the bridge vibrated transversely.
- At the north-west end the superstructure had displaced transversely about 20 mm in a westerly direction relative to the abutment.
- It is reported by the Regional Bridge Consultant that, since 22 February 2011, both abutments have settled in the order of 65 mm relative to the piers. This is generating significantly increased longitudinal moments over the piers because the superstructure is continuous. It seems that the abutment piles are founded at a much shallower depth than the pier piles.

Discussion

Structure Response: The bridge has a continuous deck over the piers and although the beams are supported on 115 mm high elastomeric bearings at both the piers and abutments the superstructure is rigidly located on the piers by RHS shear keys. The bearings supporting the beams at the abutments are relatively flexible; however longitudinal displacements are limited by tight linkage bolts and the small 25 mm gap between the span end diaphragms and the abutment backwalls. The 40 mm diameter linkage bolts have sufficient shear strength to also restrict transverse displacement at the abutments. Each abutment is founded on 10 steel H piles that result in the abutments being stiffer than the piers for transverse loading and much stiffer than the piers in the longitudinal direction when the passive resistance at the abutment pushed against the soil becomes effective. Sufficient passive resistance can be generated on the 3.15 m high back face of the abutments to resist the total superstructure inertia load from response accelerations greater than 1.0 g.

The continuous deck results in very effective diaphragm action transferring a significant part of the superstructure inertia loads to the abutments regardless of the direction of loading. Based on the assumption that there was no significant displacement of the superstructure on the abutment bearings (restrained by the linkage bolts and backwalls) periods of vibrations were estimated to be about 0.2 and 0.3 seconds in the longitudinal and transverse directions respectively. Assuming simple SDOF response with 5% critical damping the response accelerations in both the longitudinal and transverse directions would have been about 0.55 g and 1.0 g in the Darfield and Christchurch earthquakes respectively. The displacement responses in the Christchurch earthquake would have been between 10 mm and 20 mm.

Structure Strength Transversely: A simplified static analysis was carried out for the transverse direction based on the tributary mass assumption for each of the piers and abutments. This indicated the base sections of the pier columns would reach their ultimate flexural capacities at a response acceleration of about 0.4 g. From estimates of the relative stiffness of the piers and abutments it was concluded that about 70% of the transverse inertia load would be carried on the abutments indicating that the tributary mass assumption did not provide a good estimate of the transverse load on the piers. If only 30% of the transverse load is carried on the piers the ultimate flexural strength capacity of the base of the piers would be reached at a response acceleration of about 1.0 g or at about the level that was estimated for the Christchurch earthquake based on the 5% damping assumption. Although the bases of the piers were covered by about a 900 mm depth of water at the time of the inspection there was no evidence of cracking above water level indicating that the performance may have been rather better than expected. On the other hand it was evident from the permanent displacements and the disturbance to the beam bearings that the abutments had carried very high transverse loads and these may have been higher than estimated. The linkage bolts would be expected to fail in shear at a response acceleration of about 0.9 g. However, it was evident that the gaps between the beams and abutment backwalls had closed-up and significant load could have been transferred by friction on contact points.

Structure Strength Longitudinally: **A** simplified static analysis was carried out for the longitudinal direction based on the assumption that all the loading was in phase along the length of the bridge and taking into account the relative stiffness of the piers and abutment. The analysis indicated the base sections of the pier columns were not critical as it would require response accelerations greater than 2.0 g for them to reach their ultimate flexural strength capacities. The abutment backwalls are 500 mm thick and are very robust with 20 mm diameter bars in both faces spaced at 200 mm centres. It would require response accelerations of greater than 1.5 g to lead to their failure in shear friction from the loads applied by the ends of the superstructure. They also have sufficient flexural strength to resist the yield load from the linkage bolts, which were predicted to fail in tension at a response acceleration of about 1.1 g.

Conclusions

- The bridge was subjected to very strong shaking in the Christchurch earthquake, which was estimated to have an equivalent return period of 1300 years, and performed well with no significant structural damage.
- Small permanent transverse displacements at the abutments and pavement cracking in the road surface indicated that the abutments had been subjected to high lateral loads. The large number of abutment piles and the passive resistance of the backfill resulted in stiff abutment structures which carried a large part of the horizontal earthquake loads.
- The bases of the columns should be inspected for possible flexure cracking.
- It would be informative to carry out further investigation and back analyses on this bridge as it is has a simple geometry and is typical of bridges constructed in the 1990's.

SH74 HEATHCOTE RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
15 June 2011

Details of SH74 Heathcote River Bridge

SH, Region, RP & BSN	SH74; Region 6; RP 10/0.47; BSN 195.				
Location	5 km south-east of Christchurch city centre.				
Distance to EQ Epicentre	42 km Darfield EQ 4 Sep '10; 3 km Chch EQ 22 Feb '11; 4 km Chch EQ 13 Jun '11.				
Distance to Fault Rupture	26 km (Darfield EQ 4 Sep '10).				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1962. Year built 1963.				
Geometry	Length: 52m	No Spans: 3	Max Span: 17.1m	Max. Ht: 3.7m.	Width over deck slab: 10.6m
Alignment, Skew, Grade	Straight and practically level (rises 150 mm from north to south). No skew.				
No of Lanes	2 Lanes plus shoulders/kerbs.				
Superstructure	178 mm composite reinforced concrete deck on 10 PSC precast I beams per span.				
Piers	1.7 m deep x 760 mm wide RC pile cap supporting 3 x 760 mm thick x 1.75 m high RC columns, tapered from 406 mm to 635 mm, which carry a RC pier cap 685 mm deep x 760 mm wide.				
Abutments	1.2 m wide x 1.07 m deep RC pile cap carrying 370 mm thick x 1160 mm high RC backwall. All supported on 10 raked 432 mm octagonal RC piles. No settlement slab shown.				
Pier Foundation	10 x 432 mm vertical octagonal RC piles per pier.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	Liquefaction observed following the Christchurch earthquake.				
Hold-down System	Piers: 9 x25mm vertical HD bolts per span end per pier.			Abutments: 9 x25mm vertical HD bolts per abutment.	
Linkage System	Piers: 9 x38mm linkage bolts through beam diaphragms at each pier.			Abutments: 9 x 38 mm linkage bolts through beam diaphragm and abutment backwall.	
Bearings flexible in shear?	Piers: Minimal - 13mm thick x 100 mm wide neoprene strip x 9.2 m long.			Abutments: Minimal - 13mm thick x 100 mm wide neoprene strip x 9.2 m long.	
General Condition	Good with no deterioration observed.				
Other Features	None.				

SH74 Heathcote River Bridge, looking north-west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No structural damage to the bridge was observed during the inspection but the following points were noted:

- The pile tops at the abutments were not visible but based on the amount of movement indicated by the deck joints and soil gapping at the abutments it seems unlikely that the abutment piles were damaged.
- The pile tops at the piers were not visible. It was not possible to reliably estimate the transverse movements at the piers, and although unlikely, the pier pile tops could possibly have been cracked. Inspection of the pile tops would be difficult as the drawings indicate that they are below the lowest known tide level.
- The soil at the north-east corner of the bridge had moved downslope leaving a 10 mm gap between the face of the wingwall and the soil. There was also soil settlement of about 30 mm at this location. There were cracks in the silty soil near the top of the abutment slope under the bridge at a more central part of the north abutment. These cracks were probably caused by the weak surface soils settling and sliding under strong shaking. Longitudinal movements of the bridge may have also contributed to the gap and cracks in the soil. There were no obvious signs of liquefaction near the site and there were no signs of settlement of the pavement at the contact with the abutment backwall.

- The chip seal pavement had been repaired in strips across the joints at the abutments and piers at some time prior to the earthquake. The drawings show that these joints were about 20 mm wide, filled with Flexcell and sealed at the top with Pliastic. The gaps would have been widened by creep and shrinkage in the deck and beams and repairs were probably carried out to reinstate the pavement that had broken away from the original joint sealant. There was some evidence of new small cracks in the repaired strips of pavement that might have been caused by the earthquake movements. However, it appeared that the longitudinal movements of the bridge during the earthquake were small and did not result in permanent displacements of the backfill and approach pavement of more than a few millimetres.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- There were significant signs of lateral spreading and cracking in the approach soils that had not been pronounced in the 4 September 2010 Darfield earthquake.
- The soil at the north abutment had moved downslope leaving a gap between the face of the abutment and the soil of about 150 mm compared with 10 mm after the 4 September 2010 earthquake.
- There was of up to 1^{o} of rotation of the backwall of the north abutment relative to the beam ends, with the bottom of the abutment moved towards the river.
- There were cracks in the southwest wingwall and a soil gap at the riverside face of this wingwall of about 120 mm.
- The approach fill at the south end had settled about 100 mm over a length of several metres. There was similar settlement at the north abutment although it may have been a little less at that end.

Discussion

Structure Response: The bridge has 2.2 metre high abutment backwalls and would be a very stiff bridge for both longitudinal and transverse response with elastic periods of vibration in the 0.3 to 0.4 seconds range. For 5% critical damping the response accelerations would have been about 0.55 g and 0.9 g in the Darfield and Christchurch earthquakes respectively. The response displacements in the Christchurch earthquake would have been between 20 mm and 30 mm. Because of interaction at the abutment and the relatively soft soils in the surface layer the damping in the earthquake may have been higher than 5% of critical, reducing the displacement response to the lower end of the predicted range.

The bridge is quite well detailed. For example, there are no splices shown in the main longitudinal bars in the pier columns. However, the ductility of the piers would be limited by the widely spaced (300 mm) stirrups in the columns.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier. It was assumed that the pier columns were fixed at their base where they connect to 990 mm deep pile caps, and that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans. For an assumed steel yield stress of 300 MPa the analysis indicated that the main bars at the column bases would yield at a response acceleration of about 0.40 g and that the ultimate flexural strengths of the columns would be reached at about 0.45 g. The piers clearly had much greater transverse strength than would result from applying the 0.1 g lateral loading at working stress specified in the Bridge Manual at the time of the design. Although there was no obvious cracking in the piers following the Christchurch earthquake it seems likely that they were loaded to about the flexural reinforcement yield level, which may have been higher than the 300 MPa assumed in the analyses. The predicted 0.9 g response acceleration of the superstructure in the earthquake is a best estimate based on 5% critical damping. Interaction of the piles in the soft soil and lateral spreading may have resulted in higher damping and diaphragm action in the deck of the tightly linked spans probably distributed part of the transverse loads to the abutments, reducing the loads on the piers.

Structure Strength Longitudinally: A detailed analysis for longitudinal loading that considered the relative stiffness of the piers and abutments was not carried out, however because the abutments have raked piles they are much stiffer than the piers. The superstructure is connected to the abutments by tight linkage bolts with a 20 mm Flexcell filled gap between the span end diaphragms and the abutment backwalls resulting in a relatively rigid connection. The combination of stiff abutment foundations and rigid connections to the superstructure results in the abutments carrying most of the longitudinal load. In the design it was probably assumed that all the longitudinal loading would be carried on the raked abutment piles at both ends of the bridge. However, the passive soil resistance on the 2.2 m deep backwalls of the abutments would have been very effective in providing additional resistance. The ultimate passive resistance of the walls would be sufficient to carry the longitudinal load from the 0.9 g response acceleration expected in the Christchurch earthquake but would require a displacement of about 40 mm to fully develop. The raked piles are quite stiff and the lack of any large permanent displacement at the abutments suggests that the raked piles resisted most of the longitudinal load. Small permanent rotations at the abutments indicated that there had may have been some pull-out movement of the piles when the abutments were loaded by longitudinal loading towards the backfill but the rotations might also have been caused by lateral spreading. Apart from the raked piles, the abutment backwalls would be the most critical component under longitudinal loads and a flexural failure in them from linkage bolts tension loads would be expected at a response acceleration of about 0.85 g. Although it was not possible to inspect the backwalls for tension load cracking there was no obvious indication of flexural yielding.

Conclusions

- The bridge was subjected to very strong shaking in the Christchurch earthquake, which was estimated to have an equivalent return period of 1300 years, and performed well with no significant structural damage.
- It would be informative to carry out further investigation and back analyses on this bridge as it is has a simple geometry and is typical of bridges constructed in the 1960’s.

SH74 RAILWAY OVERBRIDGE

Inspection by: J H Wood and H E Chapman **Date of Visit**: 15 October 2010

Details of SH74 Railway Overbridge

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH74, Region 6; RP 19/1.97; BSN 210.</td></tr>
<tr><td>Location</td><td colspan="5">6 km south-east of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">42 km Darfield EQ 4 Sep ’10; 2 km Chch EQ 22 Feb ’11; 4 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">26 km (Darfield EQ 4 Sep ’10).</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1962. Year built 1963.</td></tr>
<tr><td>Geometry</td><td>Length: 68m</td><td>No Spans: 4</td><td>Max Span:17.1m</td><td>Max. Ht: 7m.</td><td>Width over deck slab: 10.62m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and practically level (rises 160 mm from south to north). 30º skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus shoulders and kerbs.</td></tr>
<tr><td>Superstructure</td><td colspan="5">178 mm composite reinforced concrete deck on 10 x precast PSC I beams per span.</td></tr>
<tr><td>Piers</td><td colspan="5">2 RC pile caps: 2x1.07 m deep x 2.9m wide supporting 2 x 760 mm thick x 4.8 m high RC columns, tapered from 685 mm to 914 mm, which carry a RC pier cap 1128 mm deep x 760 mm wide. Pile caps are linked with 760 mm x 1067 mm deep tie beam.</td></tr>
<tr><td>Abutments</td><td colspan="5">1.37 m wide x 1.07 m deep RC pile cap carrying 305 mm thick x 1160 mm high RC backwall. All supported on 10 raked 406 mm octagonal RC piles. No settlement slab shown.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">6 vertical 406 mm octagonal RC piles per pilecap (i.e. 12 piles per pier).</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">No information seen.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">No information seen.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: 9 x25mm vertical HD bolts per span end per pier.</td><td colspan="2">Abutments: 9 x25mm vertical HD bolts per abutment.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: 9 x38mm linkage bolts through beam diaphragms at each pier.</td><td colspan="2">Abutments: 9 x 38 mm linkage bolts through beam diaphragm and abutment backwall.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: Minimal - 13mm thick x 100 mm wide neoprene strip x full width of pier, with HD bolts.</td><td colspan="2">Abutments: Minimal - 13mm thick x 100 mm wide neoprene strip x full width of abutment, with HD bolts.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH74 Railway Overbridge, looking south.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No structural damage to the bridge was observed during the inspection but the following minor points were noted:

- The pile tops at the abutments and piers were not visible but based on the small amount of movement indicated by the deck joints, and soil gapping at the abutments and pier columns it seems unlikely that the piles were damaged.
- A previous repair at the junction between the kerb on the bridge and abutment at the north-west corner was cracked with pieces of concrete dislodged and there was minor cracking in the pavement at the deck joint. Similar kerb damage and pavement cracking was observed at the south abutment. At the south-east kerb junction an old corrosion related spall had been dislodged and cracking was evident across a section of relatively new seal at about 400 mm behind the abutment wall. There was evidence of twisting action related to the skew with the joint gap slightly more open on the west side. This kerb damage and pavement cracking at both abutments was probably caused by relative movement between the bridge and abutment during the earthquake.
- There was evidence of about 10 mm of backfill settlement at either end of the bridge but this was not sufficient to require pavement levelling following the earthquake. A narrow strip of new seal had been placed along the joint between the span and the abutment at the

south abutment but it was not clear whether this repair had been carried out following the earthquake.

- There were gaps of a few millimetres (less than 5 mm) at both abutments between the contacts of the abutment sill face with the abutment slope soil. There was no obvious soil settlement at this contact point. The gaps could have been caused by longitudinal movements of the abutments relative to the embankments but may also have been related to downslope movement of the embankment fills.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- The linkage bolts at both abutments were loose with gaps of up to 10mm between the bolt heads and bearing washers.
- The superstructure had rotated a small amount in a horizontal plane and this permanent displacement was probably related to soil-structure interaction at the 30º skewed abutments and longitudinal compression at the skewed deck joint at each pier. The deck joints had translated at the abutments and outer piers (Figures RO1 and RO2), but not at the centre pier, leaving a horizontal step between the edges of adjacent spans of approximately 10 mm. The deck joints at the abutments and two outer piers, but not the centre pier, showed signs of having been “worked”.
- Gaps in the soil at the face of the abutments evident following the 4 September 2010 event appeared to have increased to about 10 mm.
- The small amount of settlement in the approach pavement observed following the 4 September 2010 event had increased at both abutments to be of the order of 20 mm. A repair to the pavement adjacent to the abutment backwall had been carried out at the south abutment.
- Spalling damage noted at the abutment kerb junctions following the earlier event had increased.

Figure RO1
Deck joint at north abutment.

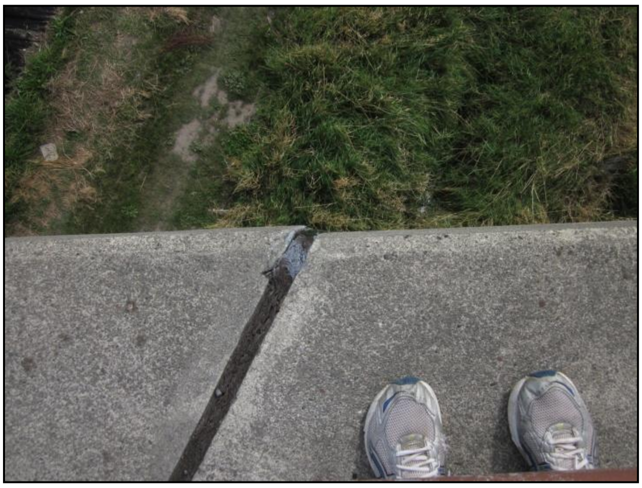


Figure RO2
Horizontal rotation at deck joint.

Discussion

Structure Response: The bridge has relatively high abutment backwalls (2.2 m) and would be a relatively stiff bridge for both longitudinal and transverse response with periods of vibration in the 0.3 to 0.5 seconds range. For 5% critical damping the response acceleration would have been about 0.45 g and 0.9 g in the Darfield and Christchurch earthquakes respectively and the response displacement in the Christchurch earthquake between 20 and 50 mm. Because of interaction at the abutments and the relatively soft soils in the surface layers the damping in the earthquake may have been higher than 5%, reducing the displacement response to the lower end of the predicted range.

The bridge is quite well detailed. For example, the splices shown in the main longitudinal bars in the pier columns are at about mid-height. However, the ductility of the piers would be limited by the widely spaced (300 mm) 9.5 mm diameter stirrups in the columns.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier. It was assumed that the pier columns were fixed at their base where they connect to 1067 mm deep pile caps, and that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans. For an assumed steel yield stress of 300 MPa the analysis indicated that the main bars at the column bases would yield at a response acceleration of about 0.3 g and that the ultimate flexural strengths of the columns would be reached at about 0.35 g. The piers clearly had greater transverse strength than would result from applying the 0.1 g lateral loading at working stress specified in the Bridge Manual at the time of the design. Although there was no obvious cracking in the piers following the Christchurch earthquake it seems likely that they were loaded to about the flexural reinforcement yield level, which may have been higher than 300 MPa assumed in the analyses. The predicted 0.9 g response acceleration of the superstructure in the earthquake is a best estimate based on 5% critical damping. Interaction of the piles in the soft soil may have resulted in higher damping and diaphragm action in the deck of the tightly linked spans may have distributed part of the transverse loads to the abutments, reducing the loads on the piers.

Structure Strength Longitudinally: A detailed analysis for longitudinal loading taking account of the relative stiffness of the piers and abutments was not carried out, however because the abutments have raked piles they are stiffer than the piers and would carry a large part of the longitudinal load. The capacity of the abutment raked pile systems combined with the passive resistance on the abutment backwalls was estimated to be sufficient to resist the longitudinal response loads without damage to the piles and walls. In the design it was probably assumed that all the longitudinal loading would be carried on the raked abutment piles at both ends of the bridge. However, the passive soil resistance on the 2.2 m deep backwalls of the abutments would have been very effective in providing additional resistance although initially the raked piles would carry most of the load because of their greater stiffness.

On the assumption of equal load resisted by each abutment, the abutment linkage bolts were predicted to yield at a response acceleration of about 0.55 g. The slackness observed in the bolts was probably caused by yield in the bolts under the longitudinal loads from response accelerations predicted to be greater than this level.

Conclusions

- The bridge was subjected to very strong shaking in the Christchurch earthquake, which was estimated to have an equivalent return period of 1300 years, and performed well with no significant structural damage.
- For transverse loading the bridge performed better than predicted by a simple static analysis assuming 5% damping. Although higher damping and horizontal diaphragm action might have reduced the earthquake loads on the piers, column yielding and abutment movements would have been expected in the Christchurch earthquake.
- In view of its very good performance it would be useful to carry out a detailed analysis of the bridge.

SH74A RUTHERFORD STREET BRIDGE

Inspection by: J H Wood and H E Chapman **Date of Visit**: 15 October 2010

Details of SH74A Rutherford Street Bridge

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH74A, Region 6; RP 0/0.71; BSN 7.</td></tr>
<tr><td>Location</td><td colspan="5">4.5 km south-east of Christchurch city centre.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">41 km Darfield EQ 4 Sep ’10; 3 km Chch EQ 22 Feb ’11; 5 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">25 km (Darfield EQ 4 Sep ’10).</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1983. Year built 1983.</td></tr>
<tr><td>Geometry</td><td>Length: 39.7m</td><td>No Spans: 3</td><td>Max Span: 13.5m</td><td>Max. Ht: 5.5m.</td><td>Width over deck slab: 18.6m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and level. 13º skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus median, 2 shoulders and 2 footpaths.</td></tr>
<tr><td>Superstructure</td><td colspan="5">13 precast PSC T beams 864 mm deep per span with longitudinal insitu concrete joints between 178 mm deep flanges, which form the deck. Deck post-tensioned transversely and made continuous over piers.</td></tr>
<tr><td>Piers</td><td colspan="5">3.5 m wide x 1.15 m thick x 14 m long RC pilecap supporting 2 RC walls, each 5 m wide x 3.7 m high, which taper from 530 to 750 mm thick. These carry the RC pier cap, 1.5 m wide 850 mm deep x 18.7 m long.</td></tr>
<tr><td>Abutments</td><td colspan="5">3.5 m wide x 1 m deep x 19.1 m long pilecap, supported on 12 x 450 mm raked octagonal PSC piles and carrying the 500 mm thick x 4.2 m high abutment wall/350 mm thick backwall and wing walls.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">12 x 450 mm raked octagonal PSC piles per pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">4 boreholes reported on drawing Sheet 2 – one at each end of each abutment. 1 borehole to 20 metres, 3 to 3 metres.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">3 metres of brown clay overlying fine to coarse soft sands until piles founding at about 19 metres depth.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: None, but 6 x 127 mm square vertical RHS shear keys per pier are cast into pier cap and protrude into deck diaphragms.</td><td colspan="2">Abutments: None.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: Superstructure is continuous.</td><td colspan="2">Abutments: 6 x 50 mm diameter linkage bolts through beam diaphragm and abutment backwall.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: No – although on 280 x 230 x 28 mm elastomeric bearings, shear keys enclosed in rubber prevent significant movement.</td><td colspan="2">Abutments: Yes - on 280 x 230 x 28 mm elastomeric bearings, but movement affected by linkage bolts through backwall.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH74A Rutherford Street Bridge, looking north-east.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No damage to the bridge was observed but the bases of the piers and abutment walls were covered by water at the time of the inspection. Damage was not reported by other inspectors who presumably saw the bridge at other tide levels.

The designed gap between the end of the deck and the abutment backwall was 50 mm. At the time of the earthquake this gap would have been increased by creep and shrinkage in the beams. There was no evidence of any distress or damage at the joint which is bridged by a proprietary Wabo Maurer rubber seal. It appeared that the longitudinal displacements at the joints were less than the predicted 40 mm.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- The 50 mm joint gaps at both abutments had closed up with pounding damage evident at the south-east and north-east corners of the bridge (Figures R1 and R2).
- The beam elastomeric bearings on the abutments had slid 40 to 50 mm from their original locations at the north-east and south-east and south-west corners.
- The approach pavement had settled by about 100 mm at the north abutment and was being repaired at the time of our inspection (Figure R3).
- The linkage bolts were loose at the south abutment. Although not readily visible, the linkage bolts were probably loose at the other abutment (Figure R4).

Discussion

Structure Response: The superstructure is mounted on 28 mm and 43 mm thick rubber pads on the piers and abutments respectively and these provide significant flexibility. The first mode elastic periods were estimated to be in the range of 0.5 to 0.6 seconds transversely and 0.6 to 0.7 seconds longitudinally. Assuming 5% critical damping the response accelerations in the Darfield earthquake would have been about 0.45 g and 0.35 g for the transverse and longitudinal directions respectively. Corresponding response displacements would have been about 25 mm and 40 mm. The response accelerations in The Christchurch earthquake were estimated to be about 0.9 g and 0.8 g for the transverse and longitudinal directions respectively - at least double those in the Darfield earthquake. Corresponding displacements would have been about 70 mm and 100 mm.

Structure Strength Transversely: No transverse analysis was carried out. The piers each comprise two walls supported on a long pilecap, and it is unlikely that transverse earthquake actions would have caused yield stresses in the substructure members.

Structure Strength Longitudinally: A simple analysis based on an assumed yield stress for the reinforcement of 300 MPa indicated that the main steel at the base of the piers would yield at a longitudinal response acceleration of about 0.4 g with the ultimate flexural strength reached at a response acceleration of 0.45 g. Damage would therefore not be expected in the piers at the predicted response acceleration levels in the Darfield earthquake. Although the predicted response acceleration in the Christchurch earthquake was significantly greater than the level expected to cause flexural yield level in the piers it is likely that forward movement of the tops of the abutment walls closed the abutment joint gaps transferring horizontal load from the piers to the abutments. Locking-up of the abutment joints would have reduced both the longitudinal and transverse loads on the piers.

At the time of the Christchurch earthquake the abutment joint gaps would have been increased by creep and shrinkage in the beams to about 60 mm. Had the bridge been able to respond freely without contact at the abutments the peak displacement response in the longitudinal direction would have been about 100 mm. So even if the abutments had not moved forward there may have been impact damage.

Conclusions

- The abutments are 4.3 m high walls supported on a pile cap with raked concrete piles. High lateral soil pressures on the walls and, perhaps, lateral spreading in the Christchurch earthquake resulted in these walls rotating and translating sufficiently to contact the superstructure and prevent it from responding freely on its bearings. Although there was impact damage to the abutments and deck, the contact prevented damage to the piers.
- The bridge performed within its elastic limits in the Darfield earthquake, but is estimated to have experienced only about a 220-year return period shaking in this event.
- Because the deformation performance of the abutment walls was markedly different in the two earthquakes it would be informative to carry out back analysis of the pressures acting on them and to estimate their deflections.

Figure R1
Closed deck joint at north abutment.

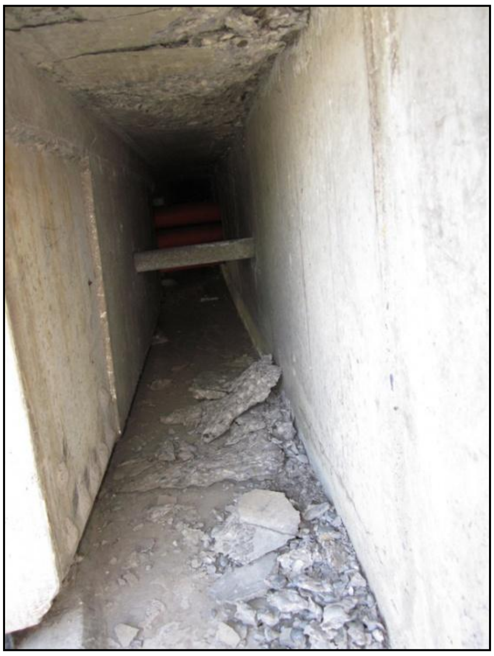


Figure R2
Pounding damage at north abutment.

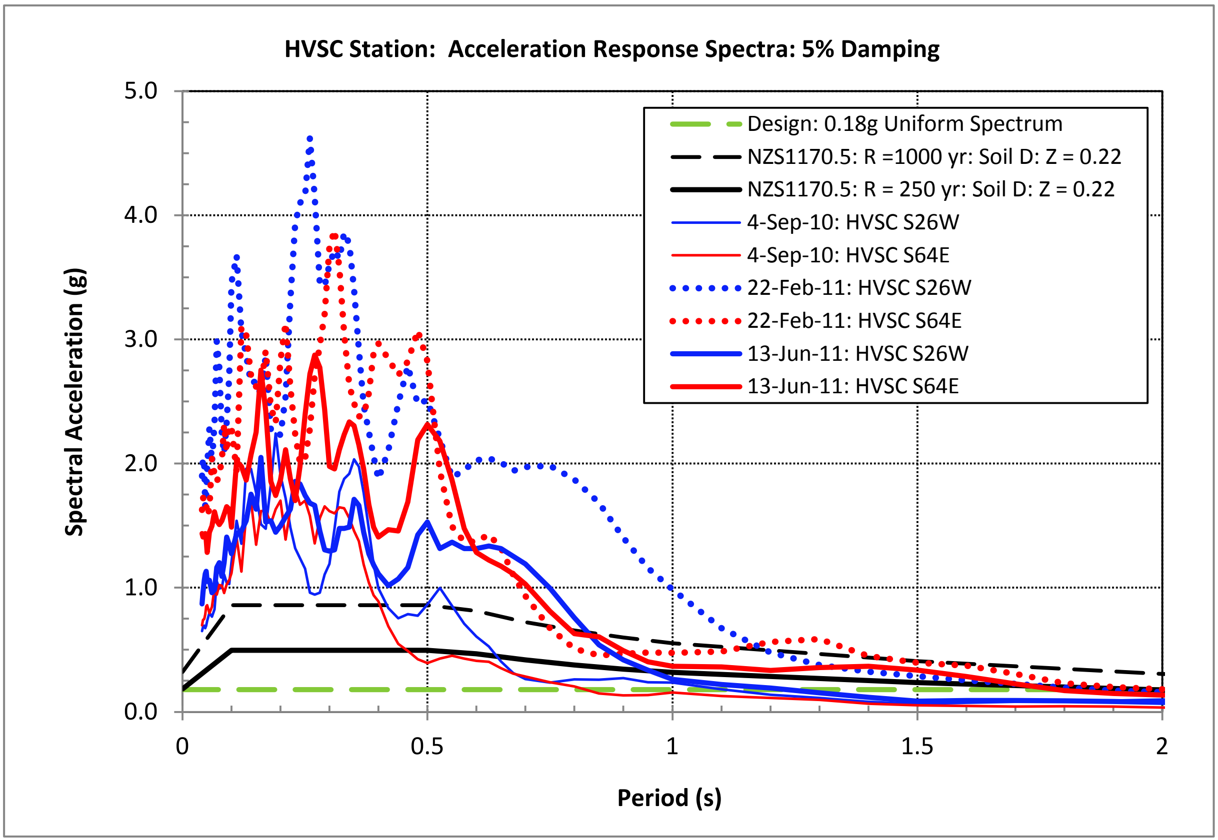
Figure R3
Settlement at north abutment.

Figure R4
Loose linkage bolts at south abutment.

GROUP 6 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA



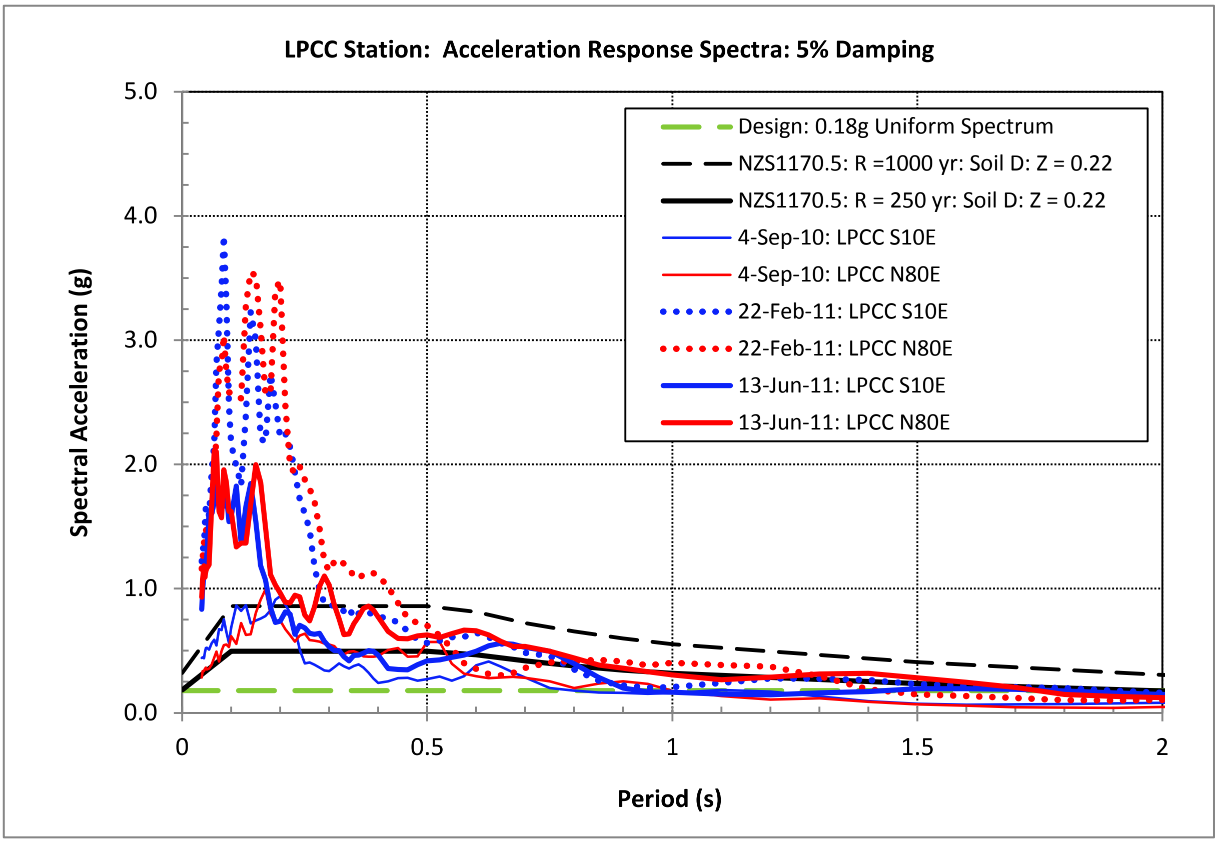
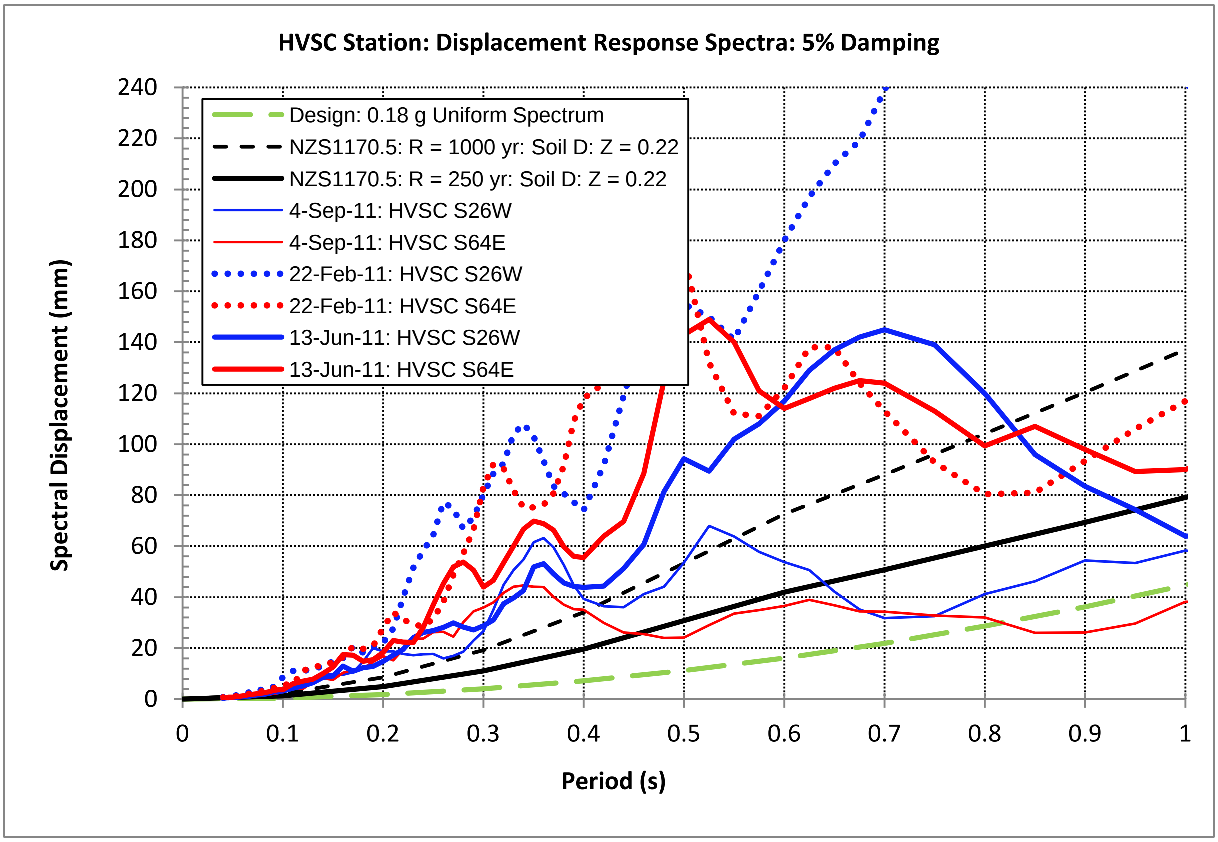


Figure B11: Acceleration response spectra from two nearest SMA's to Group 6 bridges - see Table 2.



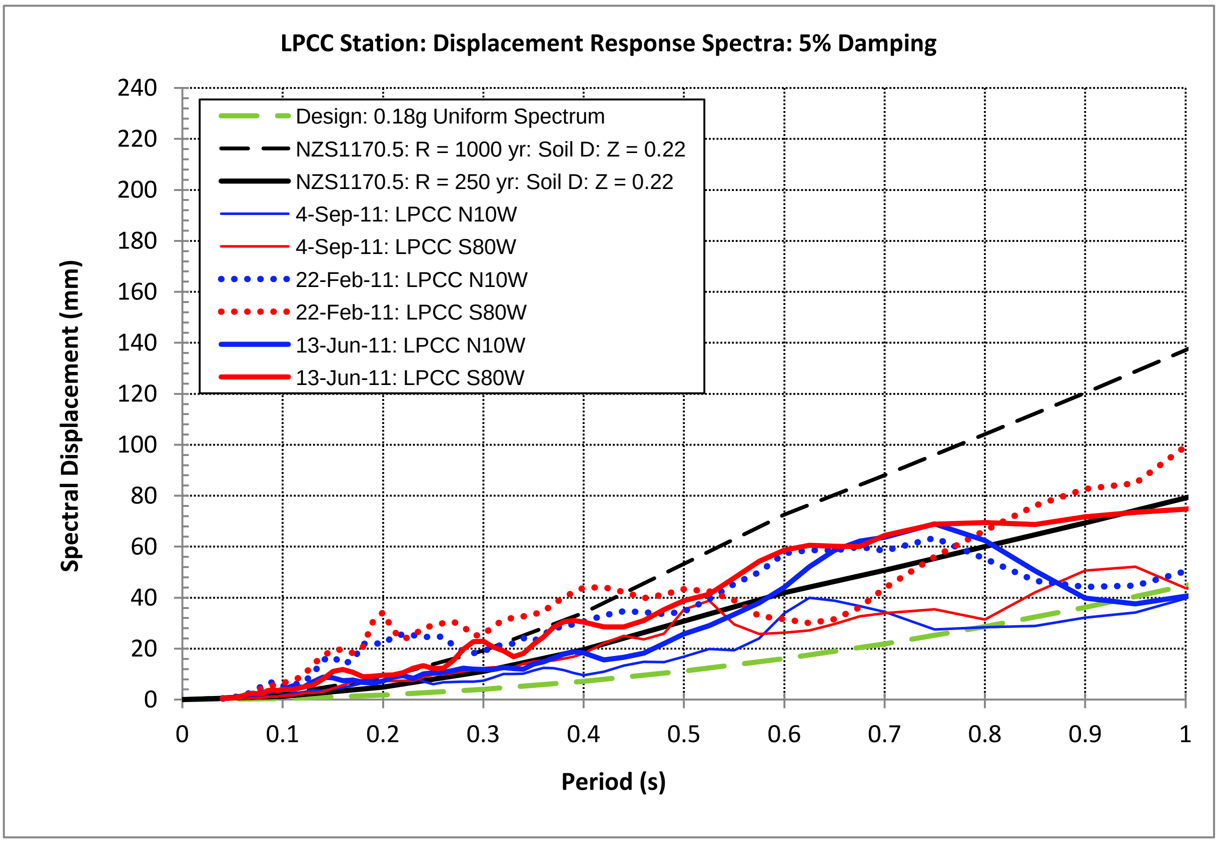


Figure B12: Displacement response spectra from two nearest SMA's to Group 6 bridges - see Table 2.

SH74 PORT HILLS ROAD UNDERPASSES NOS 1 AND 2

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
6 April & 24 August 2011

Details of SH74 Port Hills Road Underpasses Nos 1 and 2

SH, Region, RP & BSN	SH74, Region 6; RP 19/2.48; BSN 215 and 216.
Location	2 km from the Christchurch portal entrance of the Lyttelton Road tunnel.
Distance to EQ Epicentre	42 km Darfield EQ 4 Sep ’10; 1 km Chch EQ 22 Feb ’11; 4 km Chch EQ 13 Jun ’11.
Distance to Fault Rupture	26 km (Darfield EQ 4 Sep ’10).
Hazard Factor Z	0.22 (NZS 1170.5:2004).
Year Designed/Built	Designed 1962. Year built 1963.
Geometry	Lengths: No: 1 72.4m No: 2 69.3m No Spans: 6 Max Span: 12.65m Max. Ht: 8.3m. Width over deck slabs: No 1: 9.3 m No 2: 5.6 m
Alignment, Skew, Grade	430m (No 1) and 442m (No 2) radius curves, no skew, 2.5% grade.
No of Lanes	No 1: 2 lanes plus kerbs. No 2: 1 lane plus kerbs.
Superstructure	Prestressed concrete log beams with cast insitu RC deck.
Piers	Single RC rectangular column with RC hammerhead on spread footing, for each pier.
Abutments	Reinforced concrete stub wall on spread footing.
Pier Foundation	Spread footing. Piers in approach fills provided with annulus to prevent excessive stiffness and consequent excessive seismic loading (retrofit in 2009).
Soils, Borehole info.	Two boreholes were drilled in 1961 showing sandy silt and silty sand below ground water level - grading and relative density not available; two CPT tests carried out in 2003 to 12m depth, with samples tested for grading and other properties - one near each bridge.
Depth of Sediment	Approximately 30m.
Liquefaction Risk	Yes - down to 12m depth in 1,000 year return period event, initiating with a 50 year return period event. No evidence of liquefaction in the Christchurch earthquake.
Hold-down System	Main Piers: 10 pairs of 12.7mm dia HD dowels per span end, in rubber sheathing. Abutments: 10 pairs of 12.7mm dia HD dowels in rubber sheathing.
Linkage System	Main Piers: 18 - 12.7mm dia linkage bars in rubber sheathing joining spans at each pier. Retrofitted in 2009/10: Span to span linkages installed. Transverse shear keys installed. Abutments: None (no backwall). Retrofitted in 2009/10: Span to abutment linkages installed. Transverse shear keys installed.
Bearings flexible in shear?	Piers: No – retrofitted shear keys allow only nominal movement in any direction. Abutments: No – retrofitted shear keys allow only nominal movement in any direction.
General Condition	Good.
Other Features	None.

SH74 Port Hills Road Underpasses Nos 1 and 2, looking west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following minor points were noted:

- There was cracking in the seal above the soil interface at the abutment walls which may have been caused by longitudinal movements. Small steps in the white lines indicated permanent horizontal displacements of the bridge at the south-east abutments.
- The new brackets at the south-east abutment of the No 2 Bridge were tight against the abutment sill beam indicating permanent displacement of the abutment in the longitudinal direction towards the span, as a nominal 10 mm gap was set during retrofitting.
- Cracks at the edge of the pavement along the top of the approach at this location were also evident indicating a relatively small amount of downslope soil movement on the approach embankment.
- Although it was not possible to inspect the base of the piers there was no evidence of soil gapping or concrete cracking at ground level and it therefore seems unlikely that piers were loaded to exceed the reinforcement yield level.
- The retaining walls, of concrete crib blocks, with a maximum height of approximately 5 metres were briefly inspected. It was assessed in the Detailed Seismic Assessment Report that the walls would show serious distress from a 40-year return period earthquake but there was no sign of significant displacement, either locally or overall. Some spalling of the blocks due to corrosion of reinforcement was observed.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- Flexural hairline cracking up to two metres above ground level in the three central piers of the southbound (No 1) Bridge and in the second and third piers from the south abutment in Bridge No 2. The lower halves of one of the piers on each bridge, adjacent to an abutment, were not visible, being enclosed in the retrofitted steel sheath within the abutment batter.

- Soil gapping at ground level surrounding most of the piers with separation cracks of up to 15 mm.
- Spalling damage and exposed reinforcement at ground level at the south-east and north-east corners of the centre pier of the southbound (No 1) Bridge (Figure PH1). The top of the pier footing is estimated to be approximately half a metre below ground level. Following the strong aftershock event of 13 June 2011 it was reported that spalling damage had also occurred to the corner of the next pier to the north on the same bridge.
- The nominal 10 mm gaps between the recently retrofitted shear keys and the abutment face at the Lyttelton abutment of the No 1 bridge had closed up, with no clearance on two of the four keys. The gaps at this abutment on No 2 bridge had closed up following the 4 September 2010 event.
- Spalling and crushing of some of the approach roadway kerbs where they were in contact with the abutments.
- Minor settlement at the Lyttelton abutments which appeared to have been repaired with sections of new pavement after the earthquakes.
- Wide cracks in the soil slope and at the contact between the soil and abutment face under the Lyttelton abutments resulting from downslope soil movement.
- Wide cracks in the soil running parallel to the roadway at the top of the approach embankment on the east side of the approach fill at the Lyttelton end.

Figure PH1
Spalled concrete and buckled column reinforcement.

Discussion

Retrofitting: The Overpass bridges were strengthened in 2009 by fixing fabricated steel shear keys to the underside of the beams at both the abutments and piers to resist longitudinal earthquake loads (Figures PH2 and PH3). Linkage bolts have been fitted between brackets located on either side of the piers by drilling through the tops of the piers to form a tight linkage between adjacent spans. The down-stand of the brackets prevents relative movement between the spans and the piers. Linkage at the abutments has been provided by bolts extending between the brackets and the soil face of the abutment seating, with a 10 mm gap left to allow for temperature movements.

Figure PH2
Pier shear keys and annulus around pier.

Figure PH3
Abutment shear keys.

New shear keys fixed to the end faces of the abutments and piers provide resistance to transverse loads (Figures PH2 and PH3). The spans were originally held down to the abutments and piers with 12 mm diameter dowels anchored into the infill concrete between the log beams. Spans were originally linked by longitudinal 12 mm diameter bars anchored into the infill concrete. The new shear keys and linkage bolts provide a large increase to the resistance of the original linkage system which was considered inadequate for current design loads.

A steel annulus was built around each of the two piers (one per bridge) that were located in the approach fills, so that their stiffness and seismic loading were equalised with the other piers (Figure PH2).

Structure Response: The lateral sliding resistance of the footings and the passive resistance of walls at the abutments result in the bridges being relatively stiff in the longitudinal direction with periods expected to be in the 0.25 to 0.3 seconds range. Most of the longitudinal load is carried on the abutments with only a small proportion resisted by the tall and moderately flexible single stem piers founded on spread footings. The abutments only have a minor influence on the transverse response of the piers with the first mode period estimated to be in the range 0.45 to 0.6 seconds. Records from the closest SMA (HVSC at 1.6 km from the bridges) were thought to have been unduly influenced by subsurface topographic effects and the estimated ground motions were based on the second closest SMA (LPCC). The mean PGA recorded at this station was 0.28 g in the Darfield earthquake and 0.83 g in the Christchurch event. For 5% damping the estimated response accelerations in the Darfield earthquake would have been about 0.6 g and 0.35 g in the longitudinal and transverse directions respectively. In the Christchurch earthquake the corresponding response accelerations would have been about 1.2 g and 0.6 g.

The bridge is quite well detailed. For example, there are no splices shown in the main longitudinal bars in the pier columns. However, the ductility of the piers would be limited by the 9.5 mm diameter stirrups widely spaced at 300 mm in the columns.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier. It was assumed that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans. For an assumed steel yield stress of 300 MPa the analysis indicated that the main bars at the column bases would yield at a response acceleration of about 0.25 g and that columns would reach their ultimate flexural strengths at about 0.3 g. These damage levels are significantly lower than the estimated response acceleration in the Christchurch earthquake of 0.6 g and also lower than the 0.35 g predicted in the Darfield earthquake. The spalling damage in the Christchurch earthquake indicated some inelastic behaviour of the central piers but the overall performance in the two earthquakes was rather better than predicted. Damping from rocking on the spread footings was probably higher than 5% and distribution of load through deck diaphragm action might also in part explain the discrepancy between the estimated response accelerations and the damage threshold level. A simple transverse beam analysis indicated that diaphragm action could reduce the response based on the tributary mass assumption by about 20%. The degree of the reduction is sensitive to the axial stiffness in the linkages at the piers and the retrofitted linkages help in this respect.

Structure Strength Longitudinally: A simplified longitudinal earthquake analysis indicated that the passive and sliding resistance available at the abutments would resist the longitudinal inertia force from a response acceleration of about 0.5 g. With small movements at the abutments additional resistance would be provided by the piers so significant movement in the longitudinal direction would not be expected in response accelerations up to about 0.6 g. The longitudinal response acceleration in the Darfield earthquake was about 0.6 g so the lack of damage in this event was consistent with the analysis results. Only small longitudinal movements occurred in the 1.2 g response acceleration in Christchurch earthquake so overall the longitudinal performance was rather better than predicted.

Without the new shear keys and linkage bolts at the abutments, but including the passive resistance on the beam ends and bearing friction resistance, failure of the original hold-down dowels would have occurred at a response acceleration of about 0.25 g. The retrofitted shear keys at the abutments may have prevented significant damage as the response accelerations in the Christchurch earthquake would have produced loads on the original hold-down dowels significantly greater than their strength capacity.

Comparison of Performance with Predictions in Detailed Seismic Assessment of 2004: The detailed seismic assessment for the bridge before retrofitting predicted the bridge performance in up to a 1000 year return period event. A summary of the performance predicted up to the 500 year return period level is given in Table PH1 below.

Table PH1. Earthquake Performance and Damage Assessment (By Consultant, January 2004)

Vulnerable Component	Event Return Period (yrs), & PGA (g)	Damage
Longitudinal Direction		
Abutments	10 <0.05	Holding down bolts fail – carriageway at abutments damaged.
Pier F column: Bridge 1	40 0.1	Potential pier failure and damage to two spans.
Pier B column: Bridge 2	140 0.21	Potential pier failure and damage to two spans.
Pier columns: Bridge 1	500 0.33	Potential failure of columns and collapse of superstructure.
Transverse Direction		
Pier linkages	5 0.05	Linkages ineffective (rubber encased); spans pull apart. HD bolts damaged; linkages elongate, softening span-span connection. Superstructure slides down pier caps.
Pier footing	100 0.16	Severe damage to footing; possible collapse of superstructure at higher return periods.
Pier columns: Bridge 1	350 0.29	Pier flexure to ductility factor = 2; potential failure of column and collapse of superstructure.
General		
Bridge founding soil	50 0.12	Soil liquefaction initiates, causing minor bridge settlement and damage.
Approach embankments and underlying soil	50 0.12	Deformation/slumping of embankment, with lateral spreading.
Bridge founding soil	150 0.21	Soil liquefaction more extensive; bridge foundations settle significantly, resulting in major bridge damage.
Approach embankments and underlying soil	150 0.21	Deformation/slumping of embankment, with significant lateral spreading.
Bridge founding soil	450 0.32	Pier footing bearing capacity exceeded; superstructure probably collapses.
Approach embankments and underlying soil	450 0.32	Significant deformation/slumping of fill affecting both lanes; loss of highway support. SH 74 closed.

Conclusions

- The structural damage predictions made by the Consultant were too pessimistic, although all the weaknesses listed were protected by the subsequent retrofitting.
- The linkage retrofitting appeared to be beneficial and probably prevented significant damage to the span to pier connections and deck joints, and to the hold-down dowels and approach pavement at the abutments.
- The soil liquefaction and embankment failures predicted to initiate at a PGA of 0.12 g and lead to a bridge collapse and significant embankment deformation at a PGA of 0.32 g were obviously too conservative as the PGA's in the Darfield and Christchurch earthquakes were estimated to be about 0.3 g and 0.8 g respectively. It would be informative to review these soil and embankment assessments.

SH74 HOROTANE VALLEY OVERPASSES NOS 1 AND 2

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
6 April & 24 August 2011

Details of SH74 Horotane Valley Overpasses Nos 1 and 2

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH74, Region 6; RP 19/2.68; BSN 217 and 218.</td></tr>
<tr><td>Location</td><td colspan="5">2 km from the Christchurch portal entrance of the Lyttelton Road tunnel.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">42 km Darfield EQ 4 Sep ’10; 1 km Chch EQ 22 Feb ’11; 4 km Chch EQ 13 Jun ’11</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">26 km (Darfield EQ 4 Sep ’10).</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.22 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1962. Year built 1963.</td></tr>
<tr><td>Geometry</td><td>Length: 40m</td><td>No Spans: 3</td><td>Max Span: 13.9m</td><td>Max. Ht: 10.3m.</td><td>Width over deck slab:
No 1: 7.7m
No 2: 8.6-9.7m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Practically straight; negligible skew; 1 in 40 grade; 1 in 12 crossfall.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus shoulders, each bridge.</td></tr>
<tr><td>Superstructure</td><td colspan="5">Simply supported standard prestressed concrete I beams with cast insitu RC deck.</td></tr>
<tr><td>Piers</td><td colspan="5">Single RC rectangular column with RC hammerhead.</td></tr>
<tr><td>Abutments</td><td colspan="5">Reinforced concrete stub wall on spread footing – continuous between Bridges 1 and 2.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">Spread footing for each pier.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">Two boreholes were drilled in 1961 showing sandy silt and silty sand below ground water level – grading & relative density not available; one CPT test carried out to 12m depth, with samples tested for grading and other properties.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">Approximately 30m.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">Yes - down to 12m depth in 1,000 year return period event, initiating with a 50 year return period event. No obvious evidence of liquefaction in the Christchurch earthquakes.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Main Piers: 6 - 25.4mm dia HD dowels per span end.</td><td colspan="2">Abutments: 5 - 25mm HD dowels.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Span Hinges: 6 - 38mm dia linkage bars and rubber buffers per span end.
Retrofitted in 2009/10:
Linkage bars added to outer beams.</td><td colspan="2">Abutments: 5 - 38mm dia linkage bars and rubber buffers.
Retrofitted in 2009/10:
Transverse shear keys installed, which also act as longitudinal stoppers.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: Minimal as beams rest on 304 x 152 x 13 mm thick elastomeric pads and are restrained by linkage bars and HD bolts.</td><td colspan="2">Abutments: No – retrofitted shear keys allow only nominal movement in any direction.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH74 Horotane Valley Overpasses Nos 1 and 2, looking west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

Minor structural damage was observed during the inspection:

- All the abutment linkage bolts on both bridges were loose with the slackness under the washers varying between 5 to 10 mm.
- The Lyttelton abutment wall on the southbound lanes (No 1 Bridge) had cracks close to the outer beams on the inland side. Maximum crack widths were 2 to 3 mm with the cracks almost vertical in direction. There were also cracks in the same abutment wall on the seaward side of the city bound lanes (No 2 Bridge).
- The city abutment wall between the two bridges looked as if it had been forced, by excessive backfill pressure, to crack and move towards the space where there was no support from the bridges.
- The city abutment had a very wide vertical crack (maximum width about 8 mm) in the wall section between the bridges but this had obviously pre-existed the earthquake but may have widened during the earthquake.
- There was minor concrete spalling at junctions between the kerbs on the bridge and the kerbs on the abutment (particularly at the Lyttelton abutment on the inland side of the city bound lanes) that may have been caused by earthquake movements.

- There was also minor spalling from the ends of several beams where they seated on the abutments and piers which may have been caused by sliding or gaps closing during the earthquake.
- Four of the seven bolts connecting the guardrail (located on the approach embankment) to the north-west corner of the city bound bridge had sheared and there was evidence that the guardrail on the fill had moved towards the bridge. This damage appeared to be earthquake related but could have pre-existed the event.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- Fine horizontal cracking on the lower half of all four piers, mainly on the northern sides. Crack widths were up to 0.2 mm and the cracks were located between about 500 to 2600 mm above ground level. The cracks were more pronounced on the column faces nearest to the roadway under the bridge on the southern piers but away from the roadway on the northern piers. Comparison of the measurement by subsequent survey between the piers of each bridge and the drawings indicated that these dimensions had shortened by 200 mm on the No 1 Bridge (southbound bridge) and 260 mm on the No 2 Bridge with the north and south pier bases moving approximately the same amount.
- The kerbs and seal along the edges of the roadway and footpath under the bridges, particularly on the Lyttelton side, were cracked indicating significant forward movement of the toe of the abutment slopes (Figure HO1), compatible with the evident movement of the pier foundations noted above. The kerb and seal were cracked transversely at 3 to 4 metre centres for the full length of roadway under the bridges, with some pieces of the kerb displaced.
- All four abutments had moved forward by up to 20 mm. The city abutment of the southbound (No 1) bridge showed severe shear cracking in the backwall and shearing failure of two of the bolts on the new linkage brackets. Bolts on the new linkage brackets had also sheared on both abutments of the No 2 Bridge.
- At the city abutments of both bridges all the linkage bolts were loose with gaps between the washers and abutment faces of up to 20 mm. Similar gaps were present at the Lyttelton abutments but the maximum gap may have been a little less.
- The city abutments had settled (total over height of approach) by about 60 mm. This was particularly visible on the left side of the northbound (No 2) bridge (Figure HO2). The wide vertical crack in the abutment wall section between the bridges at the city abutments had opened to a width of about 60 mm. (After the 4 September 2010 event it was about 8 mm wide). Similar but narrower cracking was evident at the Lyttelton abutment.
- Surface sliding of soil was evident under the city abutments and approximately 100 mm wide cracks and separation gaps between the soil and abutments were evident at the Lyttelton abutments, indicating significant down-slope movements (Figure HO3).
- There was a fine crack in the diaphragm between the span beams at a linkage bolt location on the city abutment of the northbound (No 2) bridge. A similar crack was observed in the diaphragm at the Lyttelton abutment of the No 1 Bridge.
- The northbound (No 2) bridge has displaced transversely about 100 mm away from the No 1 Bridge at its Lyttelton end, possibly due to movement of the embankment slope that runs parallel to and close by the bridge approach. This has resulted in severe vertical cracking at the junction between the abutment seating and the abutment wall between the two bridges (Figure HO4).

- There was significant differential settlement between the approach pavements and the abutments of both bridges as evidenced by recent repairs to the asphaltic concrete pavement near the abutments.
- The approach roadway kerbs had been damaged in several locations by compression against the abutments, probably from forward movement of the backfill. Buckling of the guardrails and shearing of their connection bolts had occurred at several of the joints between the approach guardrails and the bridge end posts.

Figure HO1
Cracked footpath at Lyttelton end.

Figure HO2
Settlement of abutment at city end.

Figure HO3
Cracking from down-slope movement at abutment at Lyttelton end.

Figure HO4
Cracking in abutment at Lyttelton end.

Discussion

Retrofitting: The Overpass bridges were strengthened in 2009 by fitting fabricated steel shear keys at the abutments, primarily to resist transverse loads. However, each of the nine brackets at each abutment, which are shared by both bridges, is fitted with a 30 mm bolt into the bottom of the beam (Figure HO5). This provides longitudinal restraint in addition to that provided by the original linkage and holding down bolts. Additional linkage bolts were added between the outer beams at each pier (Figure HO6). These were designed to improve the deck diaphragm action.

Figure HO5
Shear key at abutment.

Figure HO6
Linkage bolt at pier.

Liquefaction and Ground Damage: The Geology of the Christchurch Urban Area, Part Sheets M35, M36 and N36, 1:25,000 (Institute of Geological and Nuclear Sciences, 1992) shows the geology of the site to be the Holocene Age, Valley fill and slope wash of loess-volcanic derived colluvium (c). The logs of boreholes (Boreholes 1 and 2 to 16 m and 10 m depth) drilled in April 1961 prior to the construction of the Lyttelton Tunnel Access Road (which is now the SH74 highway to Lyttelton) and labelled as having been positioned for the Horotane Valley Road Overpass have been located. The holes were percussion drilled with no testing carried out. The logs indicate the ground to be firm, stiff and hard sandy silty clay. No water levels were recorded.

One Static Cone Penetration Test (CPT 003 to 12 m depth) carried out about 20 m south of the bridge in 2003 as part of the detailed seismic assessment for the bridge was terminated in weak materials with a cone resistance of up to 5 MPa. The results of mixed samples obtained by mixing soils from depths of 3 m, 5 m and 8 m from CPT 003 indicated soils to be slightly clayey, sandy silt, with over 70% silt, and the Atterberg tests indicated this to be non-plastic.

The nature of the ground conditions are uncertain as the results from the 1961 borehole logs indicate sandy silty clay, and contradict the CPT results, the interpretation of which indicates loose to medium dense silt-sand and silt layers between about 1.5 m and 10 m depth, with silt and clay below. There may have been some difference in interpretation of the soils during logging. The CPT pore pressure measurements indicate that groundwater may be at a depth of about 1 m.

The assessment of liquefaction carried out during the detailed seismic assessment has been reviewed based on all the information available. The 2004 assessment appears to have used a standard analyses approach of the Robertson and Wride method, using the software LiquefyPro (Civiltech Corporation, 1998). The assessment indicated that the full thickness to the 13 m depth penetrated would liquefy with a peak ground acceleration of 0.33g to 0.43g used, which is smaller than the estimated maximum acceleration of 0.8 g in the 22 February 2011 event. Therefore based on the assessment in the detailed seismic assessment report, liquefaction to 13 m depth would be predicted in the 22 February 2011 event.

A review of the approach and parameters used in the LiquefyPro analysis indicates that the percentage fines assessed and used was 15% to 35% in the range of interest between 2 m and 10 m depth. This is inconsistent with the 87% fines recorded from a mix of the samples at 3 m, 5 m and 8 m depths in CPT 003 from a grading test carried out in 2003 as part of the detailed seismic assessment. Although mixing soils from different depths to carry out grading does not give the fines content of specific layers, the 87% fines in the mixed sample does indicate the generally very high fines in the soils and is somewhat more consistent with the fine grained soils indicated in the 1961 boreholes. It is possible that silt may have been recorded as clay in the 1961 boreholes, but it is less likely that sand or silty sand would have been recorded as clay. The actual fines content is therefore likely to be of the order of 87% and is much higher than the 15% to 35% that was used in LiquefyPro for the Robertson & Wride based liquefaction analyses presented in the detailed seismic assessment.

Fines content has been shown to have a significant effect on the resistance of soils to liquefaction, with increasing fines content leading to the soil being more resistant to liquefaction. The liquefaction has been reassessed using the measured fines content of 87%, the Seed et al method and the modified Stark and Olsen method for correction for fines content. A groundwater level at 2 m depth was assumed based on the February 2011 earthquake happening in summer, where the groundwater level is likely to have been lower than the 1 m indicated by CPT 003 in winter (June 2003). This reassessment indicates that liquefaction would occurr in very limited layers in the September 2010 event, and would affect the layer between about 3.5 m and 5.5 m, and a deeper layer at about 10.5 m to 11.5 m in the 22 February 2011 event. The assessment indicates negligible subsidence in the 4 September 2010 and 13 June 2011 earthquake events, and about 25 mm to 50 mm in the 22 February 2011 event.

This reassessment is also consistent with the actual observations following the three earthquake events, when there was no evidence of surface expression of liquefaction at the bridge site. In the February 2011 event, the predicted liquefaction of about 2 m thickness or less at 3.5 m depth, would be inadequate to give rise to ground damage as indicated by Ishihara (1995). The ground damage is inferred to be surface expression of liquefaction in the form of ground rupturing giving rise to sand boils and ejection of sand and water to the surface.

The assessed subsidence of the order of 25 mm to 50 mm in the 22 February 2011 event is likely to be widespread and differential subsidence may be lower. The abutments of the bridge have been surveyed to have settled by some 300 mm to 370 mm following the 22 February 2011 event with further ongoing settlement from aftershocks. The true settlement as a result of the earthquakes will be difficult to assess given that some settlement may have occurred soon after construction of the embankment and bridge abutment on weak ground with shallow foundations. At least part of the settlement recorded and the deformation of the bridge as a consequence was a result of the earthquake, and is inferred to be related to the failure / deformation of the embankment slope during the earthquake. The embankment slope is indicated on the drawings to be at 1.5 horizontal: 1 vertical (34°) and appears to be steepest at the eastern abutment. This slope is likely to have displaced due to it being a steep slope subjected to strong shaking, exacerbated by the liquefaction at 3.5 m depth. It is likely that the abutment embankment slopes have moved towards each other with failure on the liquefied layer at about 3.5 m to 5.5 m depth. This subsurface failure surface could be why there was no significant surface damage observed at the toe of the abutment embankment slopes. This movement would have directly led to the settlement of the abutments, the tilting of the piers and the relative movement of the base of the piers towards each other.

Further geotechnical investigations currently in progress confirm the generally fine grained nature of the soils and the limited potential for liquefaction in the September 2010 and February 2011 earthquakes.

Structure Response: The large spread footings at the abutments of the Horotane Valley Overpass result in the bridges being relatively stiff in the longitudinal direction with periods expected to be in the 0.3 to 0.5 seconds range. The tall cantilever single stem piers result in the bridges being quite flexible in the transverse direction with the first mode period estimated to be in the range 0.7 to 1.0 s. Based on the second closest SMA (LPCC) the best estimates of response accelerations in the Darfield earthquake for the longitudinal and transverse directions, assuming 5% critical damping, were about 0.4 g and 0.2 g respectively. The corresponding acceleration responses in the Christchurch earthquake were about 1.0 g and 0.4 g. Because of the wide variation in the short period ordinates (periods less than 1.0 seconds) of the response spectra computed from the two nearest recorder locations the estimate of response acceleration contains a large degree of uncertainty and may have been higher than these best estimate values.

The bridge is quite well detailed. For example, there are no splices shown in the main longitudinal bars in the pier columns. However, the ductility of the piers would be limited by the 9.5 mm diameter stirrups widely spaced at 300 mm in the columns.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier. It was assumed that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans. For an assumed steel yield stress of 300 MPa the analysis indicated that the main bars at the column bases would yield at a response acceleration of about 0.21 g and that the columns would reach their ultimate flexural capacities at about 0.25 g.

Although it was not possible to inspect the base of the piers there was no clear evidence of soil gapping or concrete cracking at ground level from transverse response and it therefore seems unlikely that piers were loaded sufficiently to exceed the reinforcement yield level. The retrofitted tight linkages between the spans on this relatively short bridge would have been effective in reducing the pier loads by diaphragm action and this was probably the main reason for the transverse performance being better than predicted.

Structure Strength and Performance Longitudinally: The slackness of the linkage bolts was probably caused by the abutment structures sliding a small amount towards the centre of the bridges. The drawings show 19 mm wide gaps filled with Flexcell between the abutment backwalls and the ends of the beams. Creep and shrinkage in the beams and deck would probably have widened these gaps by about 5 mm during the period following completion of construction. This shortening loads the linkage bolts, compressing the rubber washers. During the earthquakes the abutments appear to have slid forward closing any creep and shrinkage gap and compressing the Flexcell to slacken the bolts. Taking into account the flexibility of the beam rubber bearings on both the abutments and the piers, and the linkage bolts at the abutments, it was estimated that the abutments were very much stiffer than the piers in the longitudinal direction and would initially have resisted most of the longitudinal earthquake loads. High tension forces in the abutment linkage bolts may have caused flexural yielding in the backwalls and possibly yielding of the bolts, adding to the slackness in the linkage bolt assemblies. Although the backwalls are not very robust in flexure it appeared that they had translated forward together with the beam seating. If they had yielded there would have been evidence of tilting relative to the seating. It seems unlikely that the 38 mm diameter linkage bolts (11 bolts at each abutment structure) yielded as they have sufficient strength to resist the total longitudinal inertia force from a response acceleration of about 0.3 g on the superstructure (greater than the 0.2 g best estimate of the response level in the Darfield earthquake).

An analysis of the sliding stability of the abutment structures loaded by the bridge, the inertia force from the abutment mass and the backfill static and earthquake pressures indicated that they would slide forward at a ground acceleration of about 0.25 g. The embankment slopes under the bridge are quite steep at 1.5 Horizontal: 1 Vertical (34^{o} to the horizontal) and a detailed seismic assessment of the bridge completed in 2004 predicted slope failures at ground accelerations greater than 0.12 g. The shallow slope failures observed following the Christchurch earthquake were therefore expected although the slopes performed better than predicted. Probably down-slope movement contributed most to the observed forward sliding of the abutments and the resulting linkage bolt slackness but sliding of the abutments relative to the soil may have been a factor. Slope movements apparently caused the pier spread footings to each move 100 to 130 mm towards the centre of the bridge causing the cracking in the piers observed following the Christchurch earthquake.

As the abutments move forward and the Flexcell joint gap material compresses the bridges prop the abutment structures and backwalls. However, the 5 m long wall section between the bridges at each abutment is not propped and differential movement between the propped and unpropped sections probably caused the cracking observed in these wall sections.

Comparison of Performance with Predictions in Detailed Seismic Assessment of 2004: The detailed seismic assessment for the bridge predicted the bridge performance in up to a 1000 year return period event. A summary of the performance predicted up to the 500 year return period level is given in Table H1 below.

The damage predicted to the backwall for longitudinal loading at a PGA of 0.28 g is very sensitive to assumptions made in the longitudinal analysis such as the friction available on the beam rubber bearings, the compression stiffness of the gap filler and the period of vibration of the bridge. The bridge clearly experienced a PGA greater than 0.28 g in the Christchurch earthquake (probably about 0.8 g) so the assumptions made regarding the loading on the backwalls were conservative. The damage to the diaphragms, piers and footings predicted for transverse loading would have been alleviated by the retrofitted linkages to the outer beams at each pier.

Conclusions

- The retrofitting was clearly beneficial and probably prevented the onset of damage to the beam diaphragms.
- The soil liquefaction and embankment failures predicted to initiate at a PGA of 0.12 g were too conservative. These assessments have been reviewed and indicate only localised layers to be susceptible to liquefaction. The localised liquefaction may have exacerbated the displacement from "failure" of the steep approach embankments in the February 2011 earthquake, and are consistent with the observations on site. Further geotechnical investigations currently underway would help to confirm the likely behaviour of the bridges and its approaches during the earthquakes.

Table H1. Earthquake Performance and Damage Assessment (By Consultant)

Vulnerable Component	Event Return Period (yrs), & PGA (g)	Damage
Longitudinal Direction		
Abutment backwall	330 0.28	Wall sheared off, allowing span to further displace relative to abutment; potential for span drop-off under longer return period
Pier footing	350 0.29	Severe flexural/shear damage to footing; large displacements; possible collapse of superstructure.
Transverse Direction		
Diaphragms at piers, HD bolts and linkages.	20 0.07	Pier diaphragm damage; spans pull apart; softening of span – span connection.
Pier linkages	70 0.15	Linkages damaged/broken; spans pull apart; softening of span – span connection; possible loss of span support
Pier footing	210 0.24	Severe flexural/shear damage to footing; large displacements; probable collapse of superstructure.
Pier column	250 0.26	Pier flexure to $\mu = 2$, assuming pier linkages to have failed previously.
Abutments	300 0.27	Soil sliding, deforming soil behind abutment (assuming pier linkages stay effective).
General		
Bridge founding soil	50 0.12	Soil liquefaction initiates, causing minor bridge settlement and damage.
Approach embankments and underlying soil	50 0.12	Deformation/slumping of embankment, with lateral spreading.
Bridge founding soil	150 0.21	Soil liquefaction more extensive; bridge foundations settle significantly, resulting in major bridge damage.
Approach embankments and underlying soil	150 0.21	Deformation/slumping of embankment, with significant lateral spreading; Partial collapse of embankment onto Port Hills Road.
Bridge founding soil	450 0.32	Pier footing bearing capacity exceeded; embankments slump inwards and displace/damage piers; superstructure collapses.
Approach embankments and underlying soil	450 0.32	Significant deformation/slumping of fill affecting both lanes; collapse of embankment onto Port Hills Road (the bypass route) affecting one lane.

SH74 HEATHCOTE VALLEY OVERPASS

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
6 April 2011

Details of SH74 Heathcote Valley Overpass

SH, Region, RP & BSN	SH74, Region 6; RP 22/1.5; BSN 235.				
Location	260m from the Christchurch portal entrance of the Lyttelton Road tunnel.				
Distance to EQ Epicentre	43 km Darfield EQ 4 Sep ’10; 0.5 km Chch EQ 22 Feb ’11; 4 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	27 km (Darfield EQ 4 Sep ’10).				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1962. Year built 1963.				
Geometry	Length: 8.5m	No Spans: 1	Max Span: 7.4m	Max. Ht: 6.5m	Width over deck slab: 18.1m
Alignment, Skew, Grade	Straight; no skew; slight grade.				
No of Lanes	2 Lanes plus tapers.				
Superstructure	Simply supported prestressed concrete log beams with RC cast insitu deck on top.				
Piers	None.				
Abutments	Reinforced concrete slab walls propped apart at base by concrete road slabs, and at top by deck, to the soffit of which anchor brackets were retrofitted in 2009. Crib retaining walls on each side.				
Pier Foundation	None.				
Soils, Borehole info.	Footings stated as probably founded directly on hard sandy silts (Banks Peninsula Loess).				
Depth of Sediment	Loess down to volcanic rock. Rock outcrops were noted on the uphill side indicating that the bridge is on a shallow loess layer.				
Liquefaction Risk	No.				
Hold-down System	Piers: N/A.		Abutments: 12.7mm dowels in pairs at 1,200mm c/c.		
Linkage System	Piers: N/A.		Abutments: No linkages – only dowels. Retrofitted in 2009/10. Strengthening brackets to anchor span to abutment walls.		
Bearings flexible in shear?	Piers: N/A.		Abutments: No.		
General Condition	Good.				
Other Features	High crib walls on approach embankments.				

SH74 Heathcote Valley Overpass, looking south.
(Note retrofitted anchor brackets at top of wall.)

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No damage was observed to the bridge structure although spalls in previous repairs to the abutment nib wall where it was in contact with beams at the north-west corner of the bridge may have been dislodged by earthquake movements. The crib wall supporting the approach embankment on the north-east side of the bridge may have moved outwards a small amount (up to 50 mm) as the upper rows of blocks were out of alignment with the abutment wing wall.

Additional Damage Observed Following 22 February 2011 Christchurch Earthquake

- Damage to previous repairs and some new spalling damage at the ends of the abutment nib walls, deck edges in contact with the nib walls and at the tops of the wing walls. Most of this damage was observed after the first event and was mainly confined to the Lyttelton end of the bridge.
- Loss of mortar between the face of the new shear brackets anchored to the deck and the face of the top of the abutment walls (Figures HVO1 and HVO2). The mortar had not adhered satisfactorily to the concrete or steel brackets and there was no method of retaining the mortar in place. Without it there is a risk of excessive movement leading to damage of the nibs on the abutment walls.
- Relative settlement between the approach pavement and the abutments as evidenced by the strips of recent pavement repair adjacent to the abutments.

- Evidence of outward movement and settlement of the crib walls supporting the approach fills at either end of the bridge. This was particularly noticeable on the north-east side of the bridge. Some of this movement had occurred in the 4 September 2010 event. Outward movement had led to cracking on the east pavement edge near the top of the north-east wall.

Figure HVO1
Shear bracket on underside of deck units.

Figure HVO2
Mortar on ground from brackets above.

Discussion

Structure Response: The short bridge structure is essentially locked into the soil and would have been racked by the ground motion displacements in the soil on either side. The peak ground displacement recorded in the Christchurch earthquake at the LPCC station location about 2.7 km to the south-east of the bridge was 166 mm. The displacement at the closer HVSC recorder was about 230 mm but this station experienced very strong short period ground motions which were unlikely to have occurred at this bridge site.

A detailed analysis requires a better knowledge of the soil properties and geology at the site than was available at the time of writing. Rock is thought to be at a shallow depth below the wall footings. In the unlikely event that most of the recorded peak ground displacement of 166 mm occurred over the height of the structure the shear strain deformation of the walls would be about 3%. Because the walls are pinned top and bottom significant damage to the bridge would not have been expected in the very strong shaking in the Christchurch earthquake although the joint dowels and keys could be damaged by rotations greater than estimated.

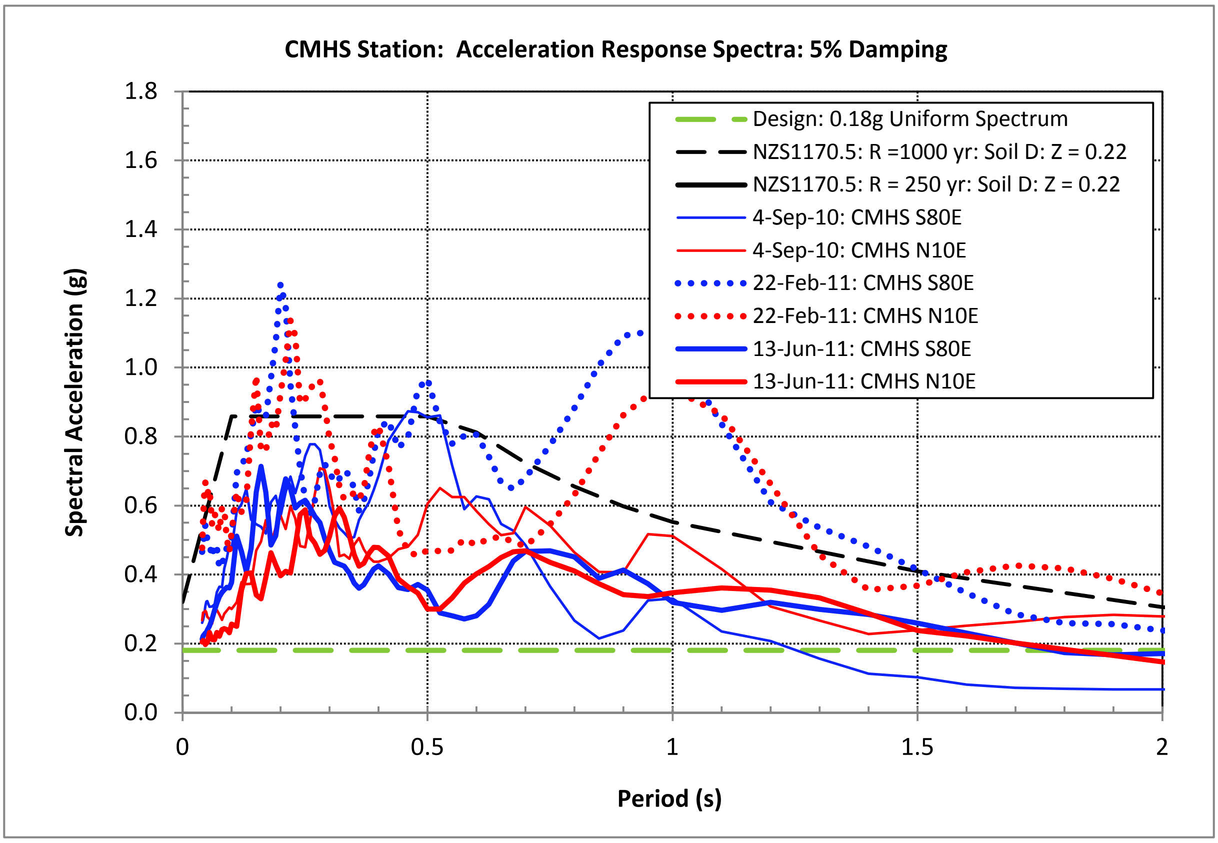
Conclusion

- The assessed return period of the shaking this bridge experienced in the Christchurch earthquake was about 5000 years and it performed well. No firm indication was seen that the retrofitted brackets prevented damage but they are undoubtedly justified to ensure good performance of the connections at the tops of the abutment walls which were subjected to quite large rotations.

GROUP 7 BRIDGE:

- **The Response Spectra**
- **Description of Bridge, Observations Made, and Discussion**

THE RESPONSE SPECTRA



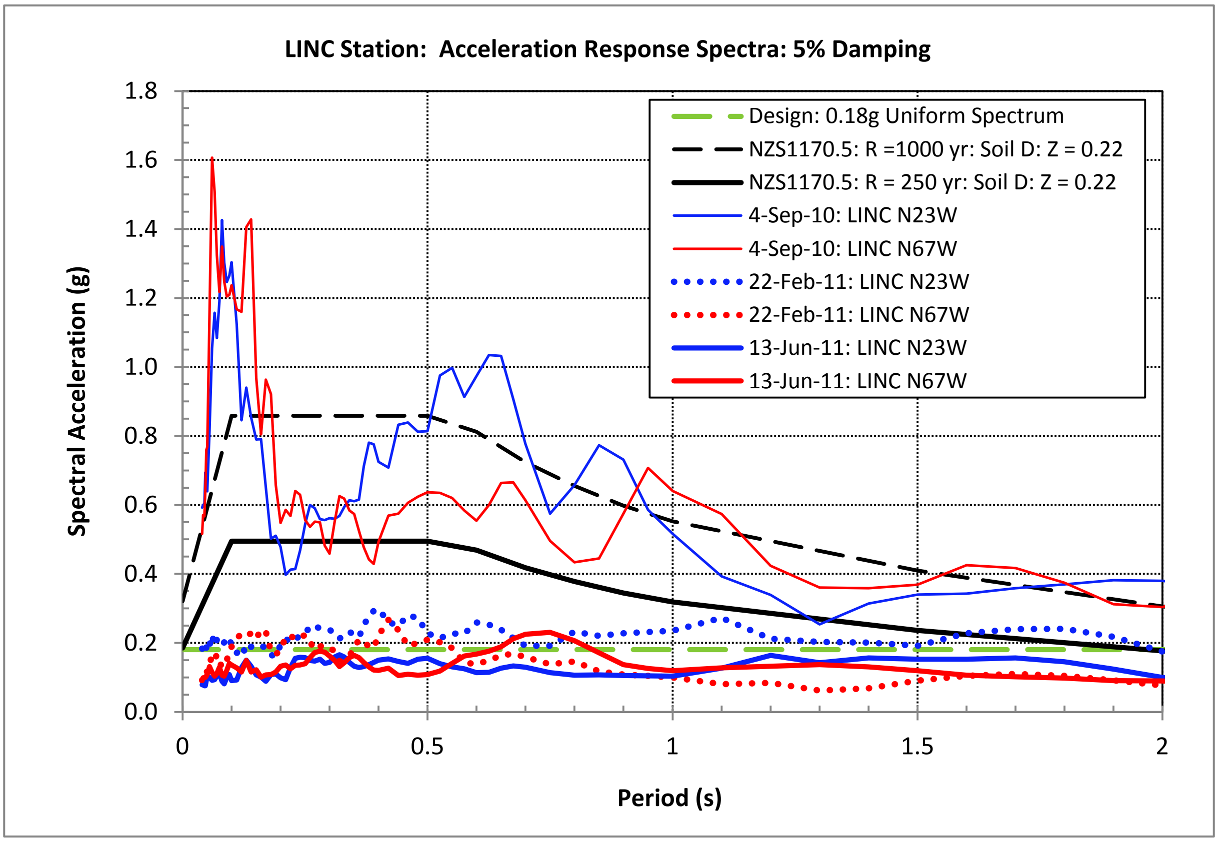
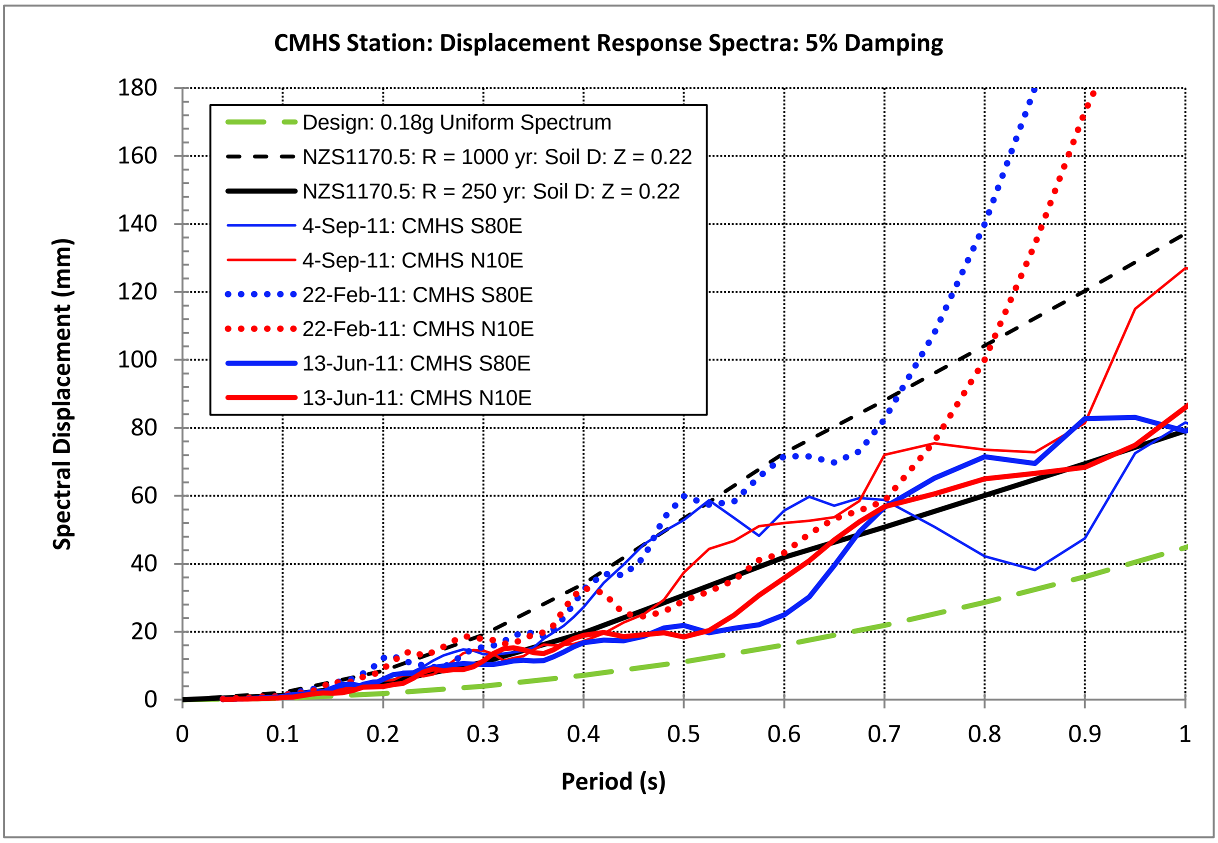


Figure B13: Acceleration response spectra from two nearest SMA's to Group 7 Bridge - see Table 2.



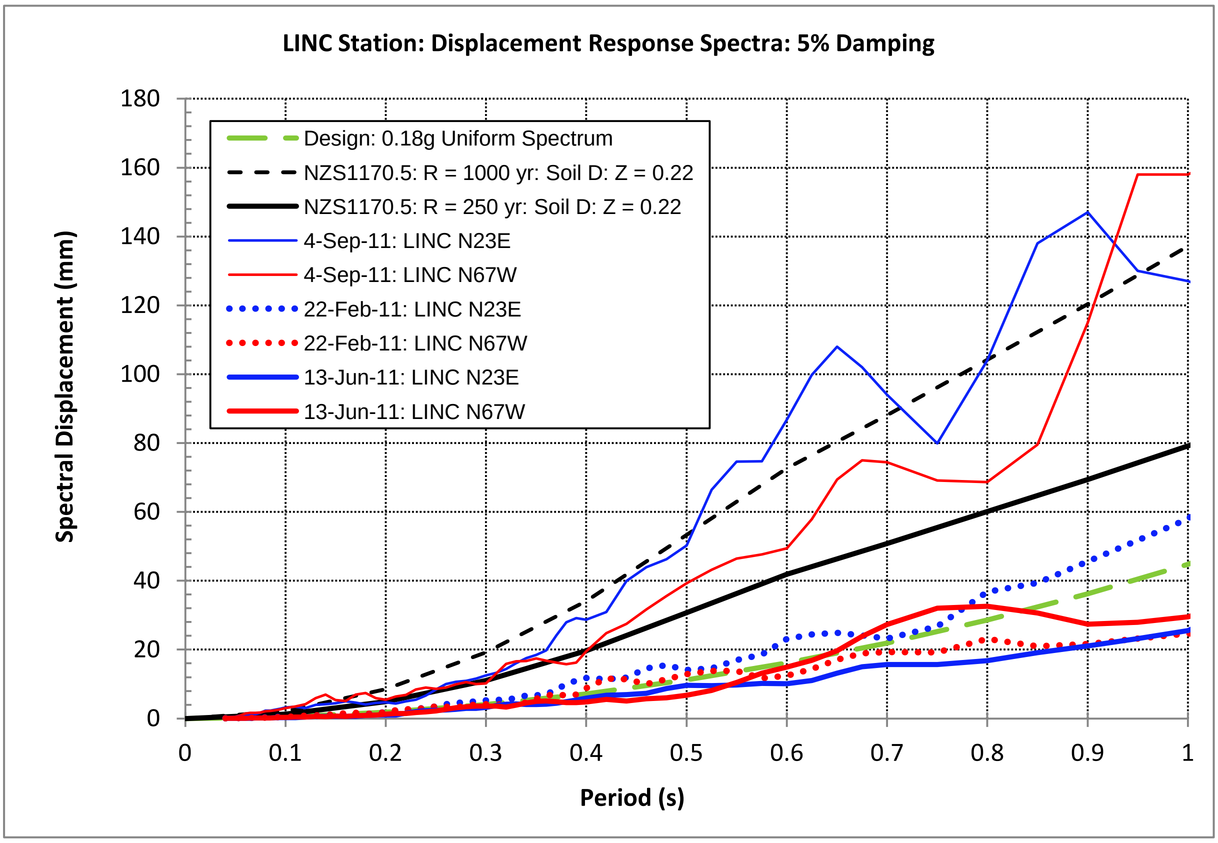


Figure B14: Displacement response spectra from two nearest SMA's to Group 7 Bridge - see Table 2.

SH75 HALSWELL RIVER BRIDGE (LANDSDOWN)

Inspection by: J H Wood and H E Chapman **Date of Visit**: 15 October 2010

Details of SH75 Halswell River Bridge (Landsdown)

SH, Region, RP & BSN	SH75, Region 6; RP 8/0; BSN 80.				
Location	10 km south-west of Christchurch city centre.				
Distance to EQ Epicentre	32 km Darfield EQ 4 Sep '10; 12 km Chch EQ 22 Feb '11; 16 km Chch EQ 13 Jun '11.				
Distance to Fault Rupture	15 km (Darfield EQ 4 Sep '10).				
Hazard Factor Z	0.22 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1935. Year built 1937.				
Geometry	Length: 6.7m	No Spans: 1	Max Span: 6.4m	Max. Ht: 2m.	Width over deck slab: 8.3m
Alignment, Skew, Grade	Straight and level. No skew.				
No of Lanes	2 Lanes plus minimal shoulders.				
Superstructure	203 to 254 mm RC deck slab on 4 composite 381wide x 508 mm downstand RC beams.				
Piers	N/A.				
Abutments	533 mm thick x 2.08 m high RC wall 8.3 m long, carried on 4 x 356 mm octagonal RC piles.				
Pier Foundation	N/A.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	No information seen.				
Hold-down System	Piers: N/A	Abutments: Integral construction.			
Linkage System	Piers: N/A.	Abutments: Integral construction.			
Bearings flexible in shear?	Piers: N/A.	Abutments: No - Integral construction.			
General Condition	Reasonable for its age, but walls cracked by earthquake actions.				
Other Features	None.				

SH75 Halswell River Bridge (Landsdown), looking south.

RESULTS OF INSPECTION

Damage Observed Following the 4 September 2010 Darfield Earthquake

Liquefaction induced lateral spreading of the soil approaches resulted in flexural failures in the abutment walls at the underside of the beams (Figures H1 and H2). Wide flexural cracks developed on the soil side of the walls and were visible at the wall ends at three corners of the bridge. The widest crack was at the north-east corner and was about 15 mm wide. At the time of the inspection there was no crack visible at the south-west corner. Following a magnitude 5.0 aftershock on 18 October 2010, which had an epicentre about 2.1 km south of the bridge, a fine crack was also observed at this corner.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The relatively deep abutment walls essentially lock the short span into the ground and it tends to respond with the ground displacements rather than vibrating freely. Based on the mean of the response spectra from the two closest SMA's located at approximately equal spacing on either side of the bridge, and aligned in an approximately east-west direction, the PGA's and spectral accelerations were generally a little larger in the Darfield earthquake than the Christchurch earthquake (see Figures 4, 5 and 7). The mean of the peak ground displacements recorded in the two horizontal components of both records were 350 mm and 90 mm in the Darfield and Christchurch earthquakes respectively.

Structure Strength Transversely: The structure is robust transversely, comprising an integral T-beam deck on abutment walls and piles.

Structure Strength and Performance Longitudinally: There was clear evidence of soil liquefaction at the site with cracks in the stream banks adjacent to the bridge indicating lateral spreading of the banks towards the stream. The approach fills had settled about 100 mm relative to the bridge with the step at the backfill interface with the abutment walls repaired with new asphaltic seal after the earthquake.

Figure H1
North-east corner.

Figure H2
North-west corner.

Liquefaction presumably occurred in sandy soils below the stream bed level, which is about 400 mm below the bottom of the walls. The drawings show the piles founded in gravel at a depth of about 2.2 m below the stream bed so the liquefied layer may not have been very deep. The pile tips are at about 4.6 m below bed level and as there was no obvious settlement of the bridge it is unlikely that liquefaction reached this depth. Because the soil profile is not known in any detail it is not possible to reliably calculate the effects of liquefaction on the bridge. Calculations showed that the walls had insufficient flexural capacity to resist passive Rankine earth pressures. During lateral spreading pressures against restrained structures can be up to about four times greater than Rankine pressures.

The lateral pressures on the piles may have also loaded the base of the walls and contributed to the failures. Failure of the walls from the lateral spreading pressures would therefore be expected. Plastic hinging in the tops of the piles may have also occurred as their combined flexural strength is lower than the flexural strength of the wall.

Conclusion

- Since it is likely that the bridge will need to be replaced, and in view of its small size, no follow-up investigations or analysis are recommended.

GROUP 8 BRIDGES:

- **The Response Spectra**
- **Description of Bridges, Observations Made, and Discussion**

THE RESPONSE SPECTRA FOR 4 SEPTEMBER 2010: 5% DAMPING

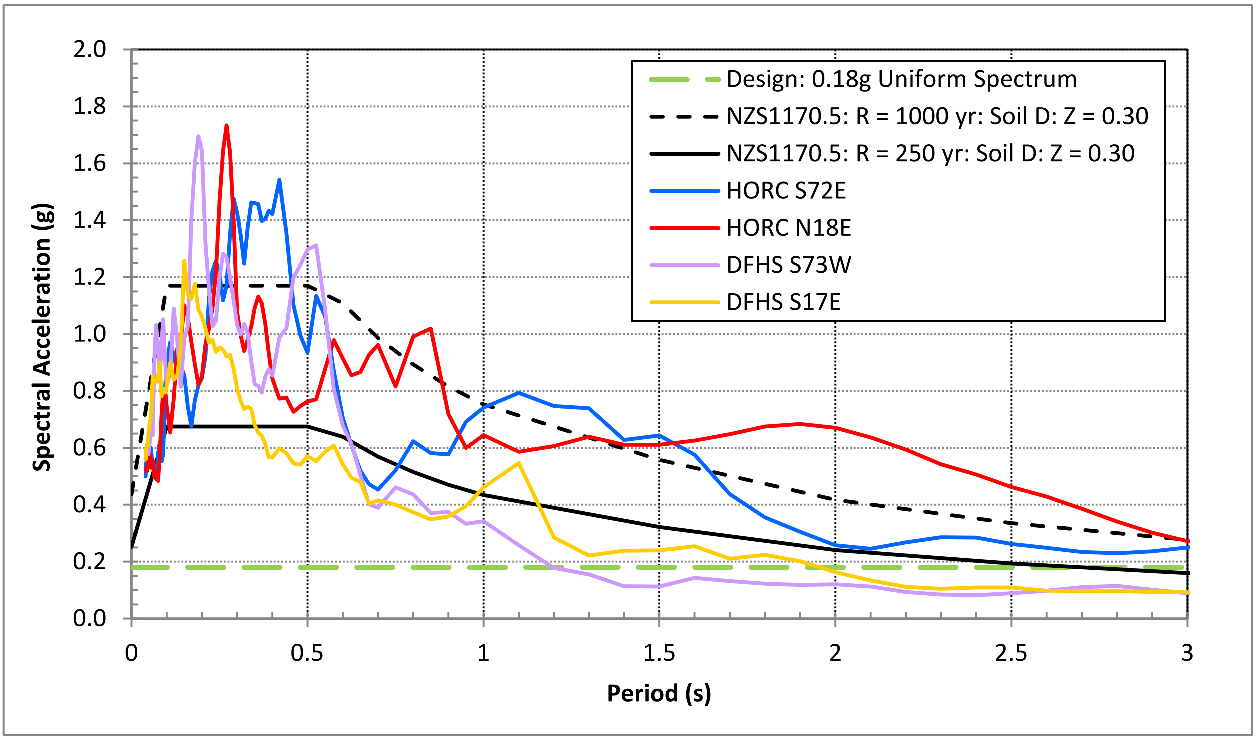


Figure B15: Acceleration response spectra from two nearest SMA's to Group 8 bridges - see Table 2.

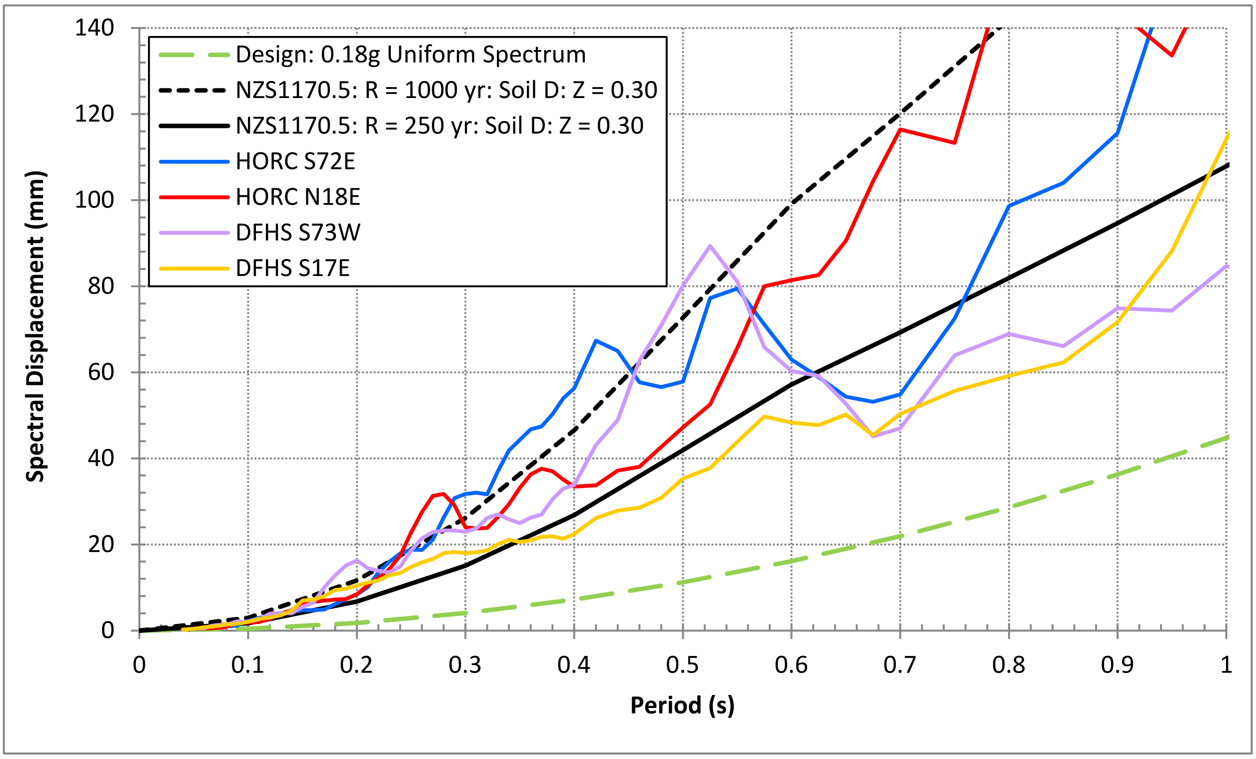


Figure B16: Displacement response spectra from two nearest SMA's to Group 8 bridges - see Table 2.

SH77 WAIRIRI STREAM BRIDGE

Inspection by: J H Wood and H E Chapman — Date of Visit: 15 October 2010

Details of SH77 Wairiri Stream Bridge

SH, Region, RP & BSN	SH77, Region 6; RP 67/9.91; BSN 769.				
Location	18 km west of Darfield.				
Distance to EQ Epicentre	22 km Darfield EQ 4 Sep 10; 65 km Chch EQ 22 Feb ’11; 68 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	14 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.30 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1962. Year built 1964.				
Geometry	Length: 16.5m	No Spans: 1	Max Span: 15.8m	Max. Ht: 4m.	Width over deck slab: 8.58m
Alignment, Skew, Grade	Straight and level. No skew.				
No of Lanes	2 Lanes plus minimal shoulders.				
Superstructure	152mm composite reinforced concrete deck on 5 PSC I beams.				
Piers	N/A.				
Abutments	508 mm x 914 mm deep pilecap supporting the 406 mm x 2 m high x 8.2 m long abutment wall, the 1.23m high backwall and 2 angled wing walls. All are carried on 6 x 406 mm octagonal RC piles. Abutment wall has 304 mm thick counterfort wall attached centrally on its buried face.				
Pier Foundation	N/A.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	No information seen.				
Hold-down System	Piers: N/A.	Abutments: 8 x 25 mm HD bolts per abutment.			
Linkage System	Piers: N/A.	Abutments: 8 x 38 mm diameter linkage bolts per abutment.			
Bearings flexible in shear?	Piers: N/A.	Abutments: 127 mm x 610 mm x 13 mm thick neoprene pad under each beam but movement is restricted by linkage bolts through backwall.			
General Condition	Good with no deterioration observed.				
Other Features	None				

SH77 Wairiri Stream Bridge, looking north.

RESULTS OF INSPECTION

Damage Observed Following the 4 September 2010 Darfield Earthquake

No structural damage to the bridge was observed during the inspection. Except for piles on the north side of the west end of the bridge the piles were covered by soil and could not be inspected.

The pavement had recently been repaired at both abutments. These repairs smoothed out settlement steps at the abutment interfaces which probably occurred during the earthquake. Although it was not possible to obtain a direct measure of the amount settlement it appeared to be significant and of the order 50 mm.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: Because of its short length and high abutment walls the bridge is essentially locked into the soil for longitudinal response. With pinned connections of the span to the abutments the bridge is flexible in relation to the surrounding soil and will tend to follow the shear strain deformation in the upper soil layers.

In the transverse direction the bridge may respond more freely but the long wing walls splayed at about 45^o to the abutment wall face provide significant resistance to transverse displacements.

Structure Strength Transversely: A simple analysis check for the transverse direction indicated that the abutment piles would be below their flexural yield stress under a horizontal response acceleration of 0.5 g.

Structure Strength Longitudinally: An approximate static analysis was carried out for longitudinal loading using a simple finite model of a long-section of the bridge and applying inertia loads to the bridge structure and soil for both vertical and horizontal accelerations. Under longitudinal ground acceleration, the walls tend to deform with the soil but the superstructure inertia load and lack of compatibility of the wall and soil deformations results in significant soil pressures against the wall and flexure of the wall. Vertical accelerations add to the at-rest gravity pressures on the walls. Earthquake loads, assumed to be concurrent 0.5 g ground accelerations in both the vertical and horizontal directions, combined with gravity loads, produced moments in the walls and piles that were less than yield moments based on an assumed reinforcement yield stress of 300 MPa. The walls are 406 mm thick at their mid-section with 16 mm diameter bars at 300 centres so they are reasonably robust. In addition, they have a small counterfort at the transverse mid-span but this was neglected in the analysis.

Conclusion

- Although the bridge was undamaged it would be informative to carry out a more detailed analysis of it. Inexact and very conservative methods of estimating the earthquake pressures on the abutments of small locked-in bridges have been used in the past and this bridge could be used as an example to illustrate the use of more representative procedures based on ground surface soil strains.

SH77 SELWYN RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
23 September 2010

Details of SH77 Selwyn River Bridge

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH77, Region 6; RP 79/0; BSN 790.</td></tr>
<tr><td>Location</td><td colspan="5">14 km west of Darfield.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">22 km Darfield EQ 4 Sep 10; 64 km Chch EQ 22 Feb ’11; 67 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">14 km Darfield EQ 4 Sep 2010.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.30 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1929. Year built 1931.</td></tr>
<tr><td>Geometry</td><td>Length: 91.5m</td><td>No Spans: 10</td><td>Max Span: 9.2m</td><td>Max. Ht: 2.5m.</td><td>Width over deck slab: 4.37m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and level. No skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">1 Lane plus minimal shoulders.</td></tr>
<tr><td>Superstructure</td><td colspan="5">178 to 203 mm RC deck on 4 composite insitu RC T beams.</td></tr>
<tr><td>Piers</td><td colspan="5">533 mm thick x 1.68 m high x 4.7 m long RC wall carrying 425 mm deep x 762 mm thick integral pier cap.</td></tr>
<tr><td>Abutments</td><td colspan="5">530 mm x 1.2 m high x 4.7 m long abutment wall and 970 mm high x 304 mm thick backwall, supported on 3 x 406 mm octagonal RC piles.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">3 x 406 RC octagonal piles.</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">No information seen.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">No information seen.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers:
Sliding ends of spans:
2 x 19 mm dia HD bolts.
Fixed ends of spans:
2 vertical 16 mm dia bars (both beams fixed at pier) or:
2 sloping 22 mm dia bars & 2 x 19 mm dia HD bolts (1 beam fixed & 1 sliding at pier).</td><td colspan="2">Abutments:
Spans are fixed at both abutments:
2 sloping 22 mm dia bars.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers:
Sliding ends of spans: None.
Fixed ends of spans:
2 x 19 mm dia bars (both beams fixed at pier) or:
2 sloping 22 mm dia bars into fixed beam only (1 beam fixed & 1 sliding at pier).</td><td colspan="2">Abutments: Sloping bars (see above) act also as linkages.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: Sliding steel plates with very limited, or no room for movement at fixed ends.</td><td colspan="2">Abutments: Sliding steel plates with no room for movement as ends are fixed.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good for its age with no significant deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH77 Selwyn River Bridge, looking south-west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No significant structural damage to the bridge was observed during the inspection but the following minor points were noted:

- A spall was observed on the top of one of the piles at the west abutment. This spall, or perhaps pile top cracking, may have been present prior to the earthquake. The pile tops, which are the most critical area for damage under transverse response, were not visible at five of the nine piers and at the abutments except for several at the west abutment.
- If the pier piles had suffered damage from longitudinal response this would be at the maximum bending moment points about 1 m below ground level and this location was not visible.
- There was no evidence of significant displacements of the bridge at the soil interface with the visible piles or at the abutments suggesting that the response was less than indicated by displacement predictions based on a SDOF assumption and 5% critical damping.
- There were a number of wide cracks in the soil on the river bank slope at the west end of the bridge and gaps between the soil at down-slope sides of the abutment, adjacent pier and the pier closest to the river (the third from abutment). Although the slopes were not steep the surface soil appeared to be very soft. The cracking had probably been caused by lateral spreading from liquefaction but it may have been caused by a shallow slope failure in the soft soil.

- At the west abutment, steps in the concrete kerb lines, repaired pavement, and settlement marks on the front face of the abutment wall and at guardrail posts indicated that the approach fill had settled by about 50 mm at the soil interface with the abutment backwalls. This settlement may have been related to a local area of liquefaction also indicated by the soil spreading on the slope to the river. Settlement of about the same order was evident at the east abutment but it was not clear how much of this had existed prior to the earthquake.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The Bridge has quite short piers but in the spans across the main river channel, degradation of the bed has resulted in the tops of the piles at the underside of the pier walls being up to 1.5 metres above bed level. Because the bridge is single lane the spacing between the outer piles is only 4.0 metres. Overall it is moderately stiff transversely with the piers on the banks that have piles fully embedded in soil being significantly stiffer than the piers in the river. It is quite stiff in the longitudinal direction where restraint is provided by 2.2 m high abutment walls.

Elastic periods of vibration were estimated to be between 0.25 to 0.4 seconds for the transverse direction and between 0.3 to 0.4 seconds in the longitudinal direction. Assuming simple SDOF response with 5% critical damping the response accelerations would have been about 0.9 g in both the transverse and longitudinal directions with a corresponding displacement response of about 25 mm.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and that the pile tops were 1.5 m clear of the river bed. For an assumed steel yield stress of 230 MPa the analysis indicated that the piles would reach their ultimate flexural strengths at a response acceleration of about 0.35 g. The bridge clearly performed in the transverse direction better than expected. The shaking intensity at the site may have been less than estimated but both the nearest two recorders to the site recorded high PGA's (0.45 g at HORC and 0.49 g at DFHS).

In the transverse direction there is a variation in pier stiffness along the length of the bridge which will result in several modes of vibration with closely spaced periods of vibration. These modes can interact to reduce the response estimated using the SDOF assumption. Damping in the strong shaking, both within the structure and between the ground and the foundations, might also have been higher than the assumed 5% of critical and travelling wave effects might also reduce the response in the transverse direction.

Structure Strength Longitudinally: An approximate static analysis for the longitudinal direction, based on the assumption that all the response was in-phase, indicated that piles would reach their ultimate flexural strengths at the maximum moment location below ground level at a response acceleration of about 0.5 g. At this response level significant resistance is provided by passive pressures at the abutments with the longitudinal displacement being about 25 mm. As for the transverse analysis, the bridge appeared to perform better than predicted by this simple analysis. Soil-structure interaction at the abutments might have resulted in damping higher than 5% but the most likely reason for the better than predicted performance would be travelling ground wave effects that result in a phase lag between the input motions at the piers along the length. The total length of the bridge is 91 m and the input motions could be significantly out of phase over this length resulting in a reduction in longitudinal response.

Conclusions

- The bridge seems to have performed significantly better then calculations predict, due perhaps to ground wave effects or structure or foundation damping exceeding the 5% critical assumed.
- More detailed analysis of this bridge could be useful to investigate the extent of, and reasons for, the apparently enhanced performance.

SH77 WAIANIWANIWA RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
23 September 2010

Details of SH77 Waianiwaniwa River Bridge

SH, Region, RP & BSN	SH77, Region 6; RP 79/6.44; BSN 854.				
Location	6 km west of Darfield.				
Distance to EQ Epicentre	16 km Darfield EQ 4 Sep 10; 58 km Chch EQ 22 Feb ’11; 60 km Chch EQ 13 Jun ’11.				
Distance to Fault Rupture	12 km Darfield EQ 4 Sep 2010.				
Hazard Factor Z	0.30 (NZS 1170.5:2004).				
Year Designed/Built	Designed 1933. Year built 1934, widened 1991.				
Geometry	Length: 27.3m	No Spans: 3	Max Span: 9.1m	Max. Ht: 7m.	Width over deck slab: 9.0m
Alignment, Skew, Grade	Straight and practically level. No skew.				
No of Lanes	2 Lanes plus shoulders.				
Superstructure	216mm reinforced concrete deck on 457 mm x 152 mm non-composite steel beams.				
Piers	457 mm thick x 3.66 m high x 8 m wide RC wall.				
Abutments	North: 680 mm thick x 1.7 m high x 8 m wide RC wall carrying 700 mm high x 300 mm thick backwall and wingwalls, supported on 3 x 356 RC octagonal piles plus 1 x 200UBP steel H pile under each side for widening of abutment. South: 765 mm thick x 2.96 m high x 8 m wide RC wall carrying 700 mm high x 300 mm thick backwall and wingwalls, supported on spread footing plus 1 x 200UBP steel H pile under each side for widening of abutment.				
Pier Foundation	3 x 356 RC octagonal piles plus 1 x 200UBP steel H pile under each side for widening of pier.				
Soils, Borehole info.	No information seen.				
Depth of Sediment	No information seen.				
Liquefaction Risk	No information seen.				
Hold-down System	Piers: 1 x M24 HD bolt through outer flange of 3 outer beams only.		Abutments: 1 x M24 HD bolt through outer flange of 3 outer beams only.		
Linkage System	Piers: 230 mm x 80 mm x 12 mm thick fishplate each side of 2 outer beams on each side of the deck, bolted through beam webs with 2 x M24 bolts.		Abutments: None.		
Bearings flexible in shear?	Piers: No - Nominal rigid (mortar pad).		Abutments: No - Nominal rigid (mortar pad).		
General Condition	Good with no significant deterioration observed.				
Other Features	None.				

SH77 Waianiwaniwa River Bridge, looking east.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

No earthquake related structural damage to the bridge was observed during the inspection. The tops of the piles, where damage might have been expected, were covered by soil at the abutments and water and soil at the piers, so were not inspected.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The Bridge has moderate height piers (3.7 m from top of piles to underside of beams) and is reasonably stiff transversely with the piles effectively fixed against rotation at the underside of the pier and abutment walls. It is very stiff in the longitudinal direction where restraint is provided by abutment walls 2.4 m and 3.7 m high at the east and west ends respectively. The superstructure is relatively light and this combined with the stiff substructure results in low periods of vibration. Elastic periods of vibration were estimated to be about 0.2 seconds for the transverse direction and between 0.1 to 0.15 seconds in the longitudinal direction. Assuming simple SDOF response with 5% critical damping the response accelerations in the Darfield earthquake would have been about 1.0 g and 0.8 g in the transverse and longitudinal directions respectively with corresponding response displacements of about 10 and 5 mm.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and that the pile tops were covered by soil to the underside of the pier wall. For an assumed steel yield stress of 275 MPa the analysis indicated that the ultimate flexural capacity of the concrete pile on the tension side of the pier would be reached at a response acceleration of about 0.6 g. The spans are reasonably well linked at the piers with holding down bolts on all beams and fish plate connections between the ends of the new beams in adjacent spans. Deck horizontal diaphragm action would therefore be expected to transfer load from the piers to the abutments, but as the abutments in the transverse direction were of similar stiffness to the piers the reduction in the pier loads would not be very large. A more detailed inspection of the pile tops should be carried out to check for any hidden damage.

Structure Strength Longitudinally: An approximate static analysis for the longitudinal direction, based on the assumption that all the response was in-phase along the length of the bridge, indicated that passive pressures against the abutment walls would provide most of the longitudinal resistance. The abutments are much stiffer than the piers, which are essentially pinned at their tops and cantilever from the piles. Under a longitudinal response acceleration of 0.8 g the displacement was estimated to be less than 5 mm and reinforcement in the piles and walls was well below yield stress level.

Conclusions

- There is a big difference between the predicted transverse response acceleration (1 g assuming 5% critical damping) and the acceleration at which the flexural capacity of the tension-side concrete piles in the piers would be reached (0.6g). This difference could be significantly reduced by damping from the soils retained against the abutments. In this case the pier piles would probably not be at risk of damage.
- The tops of the outer concrete piles at the piers should be inspected for a metre below the pier walls for possible flexural damage.

SH77 HAWKINS RIVER BRIDGE

Inspection by: J H Wood and H E Chapman
P Brabhaharan

Date of Visit: 15 October 2010
23 September 2010

Details of SH75 Hawkins River Bridge

<table>
<tr><td>SH, Region, RP & BSN</td><td colspan="5">SH77, Region 6; RP 79/11.17; BSN 902.</td></tr>
<tr><td>Location</td><td colspan="5">4 km west of Darfield.</td></tr>
<tr><td>Distance to EQ Epicentre</td><td colspan="5">12 km Darfield EQ 4 Sep 10; 53 km Chch EQ 22 Feb ’11; 56 km Chch EQ 13 Jun ’11.</td></tr>
<tr><td>Distance to Fault Rupture</td><td colspan="5">11 km Darfield EQ 4 Sep 2010.</td></tr>
<tr><td>Hazard Factor Z</td><td colspan="5">0.30 (NZS 1170.5:2004).</td></tr>
<tr><td>Year Designed/Built</td><td colspan="5">Designed 1939. Year built 1939.</td></tr>
<tr><td>Geometry</td><td>Length: 82.3m</td><td>No Spans: 6</td><td>Max Span: 13.7m</td><td>Max. Ht: 3m.</td><td>Width over deck slab: 8.33m</td></tr>
<tr><td>Alignment, Skew, Grade</td><td colspan="5">Straight and practically level (Deck rises 140 mm from east to west). No skew.</td></tr>
<tr><td>No of Lanes</td><td colspan="5">2 Lanes plus minimal shoulders.</td></tr>
<tr><td>Superstructure</td><td colspan="5">178 mm composite reinforced concrete deck on 4 x 356 wide x 813 mm o/a depth RC beams.</td></tr>
<tr><td>Piers</td><td colspan="5">Piers B,C, E & F: 610 mm wide x 1 m thick x 7.5 m long RC pile cap carrying a 457 mm thick x 1.88 m high x 7.3 m long RC wall.
Pier D: 762 mm wide x 1 m thick x 7.5 m long RC pile cap carrying two 305 mm thick x 1.88 m high x 7.3 m long “split” RC walls.</td></tr>
<tr><td>Abutments</td><td colspan="5">380 mm thick x 1.37 m high x 10 m long RC wall supported on 4 x 356 mm square RC piles.</td></tr>
<tr><td>Pier Foundation</td><td colspan="5">6 x 356 mm square RC piles (all piers).</td></tr>
<tr><td>Soils, Borehole info.</td><td colspan="5">No information seen.</td></tr>
<tr><td>Depth of Sediment</td><td colspan="5">No information seen.</td></tr>
<tr><td>Liquefaction Risk</td><td colspan="5">No information seen.</td></tr>
<tr><td>Hold-down System</td><td colspan="3">Piers: Integral construction.</td><td colspan="2">Abutments: Integral construction.</td></tr>
<tr><td>Linkage System</td><td colspan="3">Piers: Integral construction. No linkage at “split” Pier D.</td><td colspan="2">Abutments: Integral construction.</td></tr>
<tr><td>Bearings flexible in shear?</td><td colspan="3">Piers: No - Integral construction.</td><td colspan="2">Abutments: No - Integral construction.</td></tr>
<tr><td>General Condition</td><td colspan="5">Good with no deterioration observed.</td></tr>
<tr><td>Other Features</td><td colspan="5">None.</td></tr>
</table>

SH77 Hawkins River Bridge, looking west.

RESULTS OF INSPECTIONS

Damage Observed Following the 4 September 2010 Darfield Earthquake

Minor earthquake related structural damage to the bridge was observed during the inspection:

- Spalling and cracking was present in the tops of the piles at the underside of four of the five pier walls (Figures HA1 and HA2). The pile tops were not visible at the central pier. The damage was more pronounced at the second pier from each abutment than at the piers closest to the abutments and was probably caused by transverse response. If the pier piles had suffered cracking damage from longitudinal response this would be at the maximum bending moment points about 1 m below ground level and would not have been visible.
- At both abutments infill concrete has been placed below the underside of the abutment structural wall and around the piles to prevent scour of the abutment backfill. The maximum height of the infill was about 1 metre and it appeared to completely surround the piles. At the west abutment this infill concrete had wide vertical cracks near the interface with the piles. There was also cracking and spalling at the interface of the infill with the structural abutment wall at the east abutment but this was mainly in plaster that had been placed in the gaps in the horizontal construction joint. The cracking damage to the west abutment infill concrete suggested that there had been significant longitudinal movement of the abutments - perhaps of the order of 10 mm (Figures HA3 and HA4).
- There was fine cracking in the structural wall at the east abutment which may have been earthquake related but it was difficult to distinguish earthquake induced from shrinkage cracking.

Figure HA1
Pier and piles.

Figure HA2
Spalling in pile under pier pilecap, probably due to transverse loading on bridge.

Figure HA3
Infill concrete below west abutment.

Figure HA4
Cracking indicating possible longitudinal displacement.

There were no reports of further damage following the 22 February 2011 Christchurch earthquake.

Discussion

Structure Response: The Bridge has relatively short piers (2.2 m from top of piles to underside of beams) and is therefore quite stiff transversely with the piles effectively fixed against rotation at the underside of the pier and abutment walls. It is also very stiff in the longitudinal direction where restraint is provided by 1.4 m high abutment backwalls and the rigid joints at the superstructure connections to the abutment and central pier. Elastic periods of vibration were estimated to be between 0.2 to 0.25 seconds both transversely and longitudinally. Assuming simple SDOF response with 5% critical damping the response accelerations in the Darfield Earthquake would have been about 0.9 g in both the transverse and longitudinal directions with a corresponding response displacement of about 15 mm.

Structure Strength Transversely: An approximate static analysis was carried out for transverse loading on a single pier assuming that the pier was loaded by the inertia force from the tributary mass of the two adjacent spans and that the pile tops were 0.5 m clear of the river bed. For an assumed steel yield stress of 275 MPa the analysis indicated that the ultimate flexural capacity of the pile on the tension side would be reached at a response acceleration of about 0.5 g. There had clearly been some inelastic action in all the visible piles so this prediction is reasonably consistent with the estimated elastic response acceleration of 0.9 g. The pile cap at the central pier had been underpinned with concrete which presumably covered the tops of the piles. This may have provided significant stiffening and strength to the tops of the piles at this pier. Deck horizontal diaphragm action would probably transfer significant load from the intermediate piers to the central pier as well as the abutments. Therefore the assumption of the transverse load on the intermediate piers being equal to the inertia force from the tributary mass might overestimate the load carried by them.

Structure Strength Longitudinally: An approximate static analysis for the longitudinal direction based on the assumption that all the response was in-phase along the length of the bridge indicated that the ultimate capacity of the pile tops at the abutments would be reached at a response acceleration of about 0.5 g. Ultimate capacities would be reached at about the same load level in the abutment wall (moving away from the soil) and in the central pier wall at the underside of the beams. There was no visible damage at these locations so clearly the bridge performed better in this direction than predicted by this simple analysis. One obvious reason was that only the stiffening from the structural walls at the abutments was included in the analysis model and clearly the infill below the walls had contributed significantly to the longitudinal stiffness of the bridge. The response in the longitudinal direction may have been influenced by travelling ground wave effects producing a phase lag between the input motions at the piers and abutments along the length and/or by damping of more than 5% critical within the soil/structure interfaces. Although the bridge is moderately long with an overall length of 82 m the wave velocities in the firm site gravels would be quite high so it seems unlikely that any reduction from this phase lag effect would be large. Further investigation of travelling ground wave effects on the response of this bridge would be informative.

Conclusions

- There is a big difference between the predicted longitudinal response acceleration (0.9 g assuming 5% critical damping) and the acceleration at which the flexural capacity of the concrete piles in the abutments would be reached (0.5g). Further investigation of travelling ground wave effects and/or by damping of more than 5% critical within the soil/structure interfaces on the response of this bridge would be informative.

APPENDIX C

ASSESSMENT OF BRIDGE STRENGTHS

APPENDIX CONTENTS

Notes: The text in this appendix refers to figures and tables that are located either in the main text or in Appendices A or B. The figure and table numbers are accordingly prefixed with either no prefix, A or B.

ASSESSMENT OF BRIDGE STRENGTHS

1. GENERAL APPROACH

To assess the strength and performance of the critical components of each of the bridges simple static analyses were carried out. These were generally based on spreadsheet computations but in some cases the piles were analysed with a simple two-dimensional computer model employing Winkler springs. For the transverse direction, the analyses were based on a tributary mass assumption for the tallest or most critically loaded pier. For the longitudinal direction, the relative stiffness of the piers and abutments was considered, with passive pressures on the abutments assumed to resist the appropriate load level based on the overall estimated displacement. Generally all the piers of similar height were assumed to be of the same stiffness although in some cases the height between the tops of the piles at the underside of the cap and the bed level varied along the length of the bridge.

2. STRUCTURAL ANALYSIS

Each structure was analysed to determine the critical components and the approximate level of seismic loading required to bring these components to their ultimate strengths in tension (linkage assemblies), flexure and shear. The analyses were approximate and were not carried out to the level of a detailed seismic assessment. The results could differ from those obtained by dynamic or non-linear push-over analyses by about ± 30%.

The seismic coefficient equivalent to the level of loading predicted to bring the critical element to its ultimate strength was compared with the equivalent ultimate strength design coefficient from the design code most likely to have been used for the design. This ratio was calculated for the two principal directions of load and the minimum of these values is listed in Tables 5 and 6 for the Darfield and Christchurch Earthquakes respectively.

Peak response accelerations for the inspected bridges in each of their principal directions were estimated using the periods of vibration obtained from the simplified analyses and the average of the spectral accelerations for 5% critical damping calculated from the time-history acceleration records from the two nearest accelerograph stations to each bridge, as shown in Figures 2 and 3 and Table 2. A minimum ratio of capacity over demand was calculated using the predicted acceleration capacities of the critical elements and the response accelerations for each principal direction. This ratio is listed in Tables 5 and 6 for the Darfield and Christchurch Earthquakes respectively.

3. MATERIAL PROPERTIES

Probable material strengths were used to assess the flexural and shear strengths of the critical components as follows:

- For bridges constructed prior to 1932 (three bridges), the nominal yield strength of reinforcement was taken as the 210 MPa value given in the Bridge Manual (Second Edition, 2003). This was increased by a factor of 1.1 to give a probable yield strength of 230 MPa.
- For bridges constructed after 1932 and prior to 1962 (four bridges), the corresponding nominal and probable strengths were taken as 250 MPa (as given in the Bridge Manual) and 275 MPa respectively.
- For bridges constructed after 1962 the respective values were assumed to be 275 MPa and 300 MPa. The current NZTA Bridge Manual indicates that the change from the nominal

250 MPa to 275 MPa yield strengths occurred at about 1967. However, most of the drawings for the bridges designed between 1962 and 1968 (14 bridges) stated that the design was based on an allowable stress in the reinforcement of 138 MPa (20,000 lb/in^2). In this period, the concrete design codes (NZSS 1900: Chapter 9: 1964, American Concrete Institute: ACI 318-63) used a working stress of 50% (or less) of the yield strength of the reinforcement indicating that the reinforcement used in construction in New Zealand from 1962 onwards had a minimum specified yield stress of about 275 MPa.

4. DESIGN CODES

The ages of the bridges inspected covered a wide range with the oldest designed in 1920 and the newest designed in 2006. Three bridges were designed prior to 1931 and these bridges were probably not specifically designed to resist seismic loads.

4.1 Ministry of Works Bridge Manual (1956)

The first code relevant to seismic design of bridges was the Ministry of Works Bridge Manual. Following the 1931 Napier earthquake it is understood that seismic loading was considered in bridge design and the provisions that appeared in the 1956 Bridge Manual were probably in use for many years prior to its publication. A Public Works Department circular dated April 1933 briefly refers to “Earthquake Stresses”, requiring all structures to be made monolithic or otherwise to be well tied together. Piers were to be designed to resist an acceleration of one-tenth of the weight of the superstructure.

The 1956 Bridge Manual used a working stress design approach and specified a seismic horizontal load coefficient of 0.1 to be applied to the total mass of the bridge. Unfactored seismic and gravity loads were combined with a 33% increase in allowable stress permitted for this combination. Over the period of application of the 1956 Bridge Manual design provisions the working stress permitted in concrete reinforcement was typically taken as 50% of the steel yield stress. Thus the seismic load coefficient required to bring a concrete section to flexural yield in the reinforcement would be $0.1 \times 2 / (1.33) = 0.15$. Ultimate flexural strengths are greater than yield strengths by a factor that varies between about 1.05 and 1.2. This increase is related to yield in the side reinforcement in circular and squat rectangular sections and the change in the neutral axis depth (on all sections) as the concrete properties become non-linear. In current strength assessment work it is usual to assume that the probable yield strength of the reinforcement is 1.1 times the minimum specified yield strength. If factors of 1.1 are applied for both of these expected increases in strength then a concrete section designed to the 1956 Bridge Manual would be expected to reach its ultimate flexural strength under a horizontal seismic design coefficient of about $0.15 \times 1.1^2 = 0.18$.

Four of the bridges inspected were designed between 1931 and 1956 and these were assumed to have been designed for earthquake using the provisions of the 1956 Bridge Manual. Six of the bridges inspected were designed between 1956 and 1962 and these would have undoubtedly been designed to the provisions of the 1956 Bridge Manual.

4.2 NZSS 1900, Chapter 8, Basic Design Loads (1964)

This loading standard was first published in 1964. Provisions in this document for seismic design were a major advance of previous design requirements and included the division of the country into earthquake zones with seismic coefficients varying between zones and also dependent on the period of vibration of the structure. At this time most seismic design was still based on working stress limits.

Between 1964 and 1971 bridges may have been designed to the Public Building seismic load provisions of NZSS 1900 rather than the Bridge Manual. For short period bridges located in the Christchurch area the working stress horizontal seismic coefficient specified in NZSS 1900 was 0.12, i.e. 20% higher than the 1956 Bridge Manual, giving an equivalent ultimate strength coefficient of about 0.22.

4.3 Highway Bridge Design Brief, Rev A (1971) (HBDB)

In 1971 the Ministry of Works and Development published the first version of Highway Bridge Design Brief (Rev A), which adopted many of the provisions of NZSS 1900 Chapter 8 and its subsequent amendments, used until about 1973. The HBDB included, for the first time, provisions for the ultimate limit state strength design of bridges and for structures to be resilient by possessing ductility.

4.4 Bridge Manual (1994) (BM)

A revised version of the HBDB, named the Bridge Manual, was published in 1994. Seismic design provisions in the BM were developed from the NZS 4203 loading code first published in 1992. The seismic provisions in both the BM and NZS 4203 included contour maps for zone factors and response spectra that varied with both the period of vibration and a structure ductility factor. A structural performance factor that was related to the site soils, and a risk factor related to the highway importance were incorporated in the BM design provisions for the first time. Apart from the changes to the zone map and response spectra there were not large changes between the seismic design provisions of the 1978 version of the HBDB (Rev D) and the 1994 BM. Both documents related the seismic design coefficient to the structure ductility with significant reductions in the coefficient for structures specifically detailed to provide good ductility in their lateral load resisting components.

Most bridge structures designed from 1971 to the present time have been detailed to provide ductility factors of at least four. If a ductility of this value is assumed then the ultimate strength seismic design coefficient for short period bridges on deep soil sites in Christchurch is 0.2 and 0.24 for bridges designed in accordance with the provisions of the HBDB and the 1994 BM respectively.

4.5 Bridge Manual – Second Edition 2003

A revised version of the Bridge Manual was published in 2003. The earthquake design provisions were based on NZS 4203: 1992 and were essentially the same as in the 1994 version of the Bridge Manual. A provisional amendment to the Bridge Manual was issued in December 2004 to bring it into line with a draft version of NZ 1170.5: 2004 which was under review and adopted soon after the Bridge Manual amendment was published. Importance level categories for bridges were revised with bridges on the highest importance level routes required to be designed for a return period level of 2500 years. This increased the risk factor (or return period factor), R, from 1.3 to 1.8 for bridges on major highways. NZ 1170.5; 2004 included a significant revision of the seismic hazard throughout the country with increases in some regions and reductions in others. The spectral shape factor was also revised.

Of the bridges inspected only the Styx Overbridge No 2, designed in 2006, would have been designed to the December 2004 amendment of the Bridge Manual. If it was assumed to have a ductility factor of 4 (a maximum of 6 is permitted) it would have been designed for a horizontal design action coefficient of about 0.25. This coefficient is dependent on both the ductility factor and the period of vibration.