Analyses of State Highway Bridges Damaged in the Darfield and Christchurch Earthquakes

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ABSTRACT:

Twelve State Highway (SH) bridges subjected to strong shaking in the 4 September 2010, M_w 7.1, Darfield and the 22 February 2011, M_w 6.2, Christchurch Earthquakes received significant structural damage. The damage varied from light to moderate but did not require closing the bridges to traffic. The type of damage experienced by several bridges indicated that longer duration shaking than occurred in the Christchurch Earthquake would have caused more serious damage and major traffic disruption. The paper describes the structural damage, and compares the results from detailed structural analyses of one of the damaged bridges with its observed performance.

1 INTRODUCTION

Fifty State Highway (SH) bridges were located within 50 km of the 4 September 2010, Darfield earthquake and 16 of these were within 10 km of the 22 February 2011, Christchurch Earthquake main shock and the 13 June 2011 aftershock. The strong motion records indicated that most of the bridges within 50 km of the first event were subjected to peak horizontal ground accelerations (PGA's) ranging from 0.12 to 0.51 g. The 16 bridges within 10 km of the second event were subjected to PGA's of 0.25 to 1.4 g. With one exception, the intensity of shaking at the bridge sites was estimated to be less in the 13 June 2011 aftershock than in the 22 February 2011 main shock. (Wood et al, 2012)

Many of the bridges subjected to strong shaking in the three main events were of reinforced concrete construction and designed prior to 1970 using working stress design (WSD) and earthquake loads based on a 0.1 g acceleration coefficient. Reinforced concrete flexural substructure members, such as circular piers and piles, would be expected to reach reinforcement yield at a response acceleration of about 0.18 g if they were sized and reinforced to WSD limits. In spite of the expected damage threshold level being much lower than the estimated bridge response accelerations in the earthquakes, only twelve bridges suffered significant visible structural damage. Liquefaction occurred at several sites and this was the principal cause of damage to at least two of the SH bridges. A number of bridges suffered non-structural damage such as slumping of abutment aprons and fracture of deck drainage pipes.

Following the main shocks, the highways were inspected by the network consultants to establish safety conditions and repairs that were required to enable traffic to flow. 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. Temporary or more permanent repairs were carried out on the pavements within the next few days but no immediate structural repairs were necessary to the bridges.

Of the 50 SH bridges subjected to strong shaking in the two earthquakes, 28 individual bridges were inspected as part of a preliminary study of their earthquake performance. Included in this total were four groups of twin bridges with each pair having similar but not identical details. A site walk-over was carried out at each of the inspected bridges with particular attention 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 faces of the piers and abutments and the surrounding ground gave some indication of the likelihood of foundation damage.

Approximate static analyses were used to assess the strength and performance of the critical bridge components. In the transverse direction, the analyses were based on a tributary mass assumption for the tallest or most critically loaded pier. In the longitudinal direction, the relative stiffness of the piers and abutments was considered. Where liquefaction was not considered to have a significant influence on the backfill strength, passive pressures on the abutments were assumed to provide longitudinal load resistance based on the estimated superstructure displacement.

Following these initial studies more detailed inspection and back-analysis of the damaged bridges is being carried out. This work is using probable material strengths, based on the best available information on the strength of the materials used at the time of construction, to assess the flexural and shear strengths of the critical components. Response accelerations in each of the bridge principal directions have been estimated using calculated periods of vibration and the average of the spectral accelerations for 5% damping calculated from the time-history acceleration records from the two nearest strong motion accelerograph (SMA) stations to each bridge.

2 STRUCTURAL DAMAGE

SH bridges that sustained significant structural damage are listed in Table 1 together with a description of the damage and the maximum PGA estimated from the records from the two closest SMA's to each site during the Darfield and Christchurch main events.

Table 1. Summary of Structural Bridge Damage

SH	Bridge Name	Estimated Maximum PGA at Site, g	Summary of Structural Damage Observed After Darfield, Christchurch Earthquakes and Main Aftershocks
1S	Kaiapoi Railway River	0.34 (Darfield)	Severe damage to abutment diaphragms from linkage loads. Cracking in some span diaphragms. Two linkage bolt failures.
1S	Chaneys Road Overpass	0.25 (Darfield)	Loose retrofitted linkage bolts at North Abutment. (Abutment A). Disturbance to abutment aprons.
MIS	Ohoka Road Underpass	0.25 (Darfield)	Spalling at column base in an area of poorly compacted concrete.
74	Styx Overpass No 2	0.19 (Darfield)	Flexural cracking and minor spalling in pier columns.
74	Anzac Drive	0.47 (Christchurch)	Rotation of abutments and cracking and spalling in portal piers.
74A	Rutherford Street	0.53 (Christchurch)	Closing of abutment joints, pounding at joints and displaced bearings.
74	Port Hills Road Overpasses	0.83 (Christchurch)	Spalling at the base of two piers and flexural cracking in others.
74	Horotane Valley Overpasses	0.83 (Christchurch)	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.
75	Halswell River (Landsdown)	0.33 (Darfield)	Severe flexural cracking in abutment walls.
77	Hawkins River	0.51 (Darfield)	Spalling and cracking of pier pile tops.

Details of four of the damaged bridges are shown in Figures 1 to 4. A detailed structural assessment of the performance of the Kaiapoi Railway River Bridge is described in the following section.

Figure 1. Kaiapoi Railway River Bridge. Photo on right shows abutment diaphragm damage.

Figure 2. Anzac Drive Bridge. Photo on right shows abutment rotation from lateral spreading

Figure 3. Port Hills Road Overpasses. Photo on right shows spalling (plastic hinge) at base of pier.

Figure 4. Hawkins River Bridge. Photo on right shows spalling at the top a pier pile.

3 KAIAPOI RAILWAY RIVER BRIDGE

3.1 Bridge Construction

The Kaiapoi Railway River Bridge, designed in 1968, carries four lanes of SH1S across both the Kaiapoi River and the South Island Main Trunk Railway. It is located about 1 km west of central Kaiapoi and has six simply supported 24.7 m long I beam spans with a reinforced concrete deck. The north and south bound lanes are on separated superstructures but are supported on common abutments and piers (see Figure 1). Piers are transverse portal frames with a total of six 914 mm diameter reinforced concrete circular columns. Abutments are conventional sill beams anchored into the backfill with friction slabs. Both the abutments and piers are founded on 508 mm square prestressed concrete piles. Spans are linked to the abutments and across the piers with twenty-four 38 mm diameter mild steel linkage bolts. Holding-down bolts are also provided at all support points.

3.2 Site Ground Motions

The bridge was located 43 km north-east of the Darfield Earthquake epicentre and about 31 km north-east of the closest point on the fault surface trace. It was located 22 km north of the Christchurch Earthquake epicentre and is 1.5 km from the KOPC SMA location where PGA's of 0.34 g and 0.2 g were recorded in the Darfield and Christchurch Earthquakes respectively. Elastic response spectra for 5% damping computed from the two horizontal ground motion components recorded during the Darfield Earthquake at KPOC are shown in Figure 5 and the time history of the N15E component, which is aligned in approximately the longitudinal direction of the bridge, is shown in Figure 6. In Figure 5 the spectra from the KPOC records are compared with the NZS 1170.5 spectra for return periods of 250 and 1000 years. In the short period range of interest for the bridge the spectra from the records lie between these two NZS 1170.5 spectra.

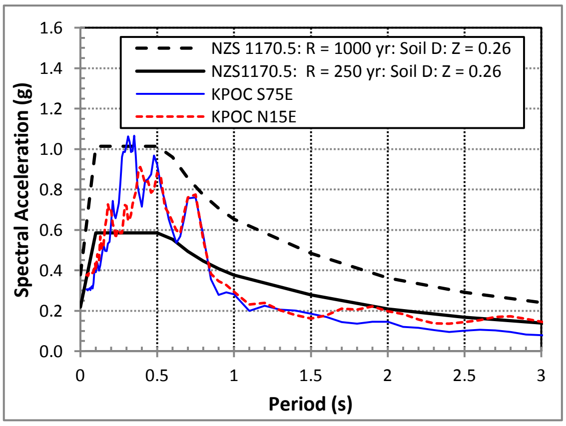

Figure 5. Acceleration spectra from KPOC.

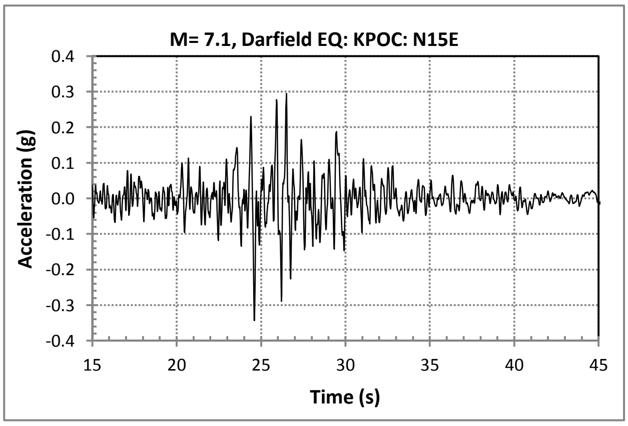

Figure 6. Acceleration time-history from KPOC N15E.

3.3 Damage

Structural damage occurred in the Darfield Earthquake. There was no visible increase in damage following the lower ground accelerations in the Christchurch Earthquake or subsequent aftershocks. The main damage was the failure of the diaphragms between the span beams at the abutments (see Figure 1). 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. This connection consists of two 32 mm diameter rods passing through the end blocks of the span beams (near the top and bottom) and across the complete diaphragm to lap with 20 mm diameter bars cast into the outer beams. Although the diaphragms have vertical stirrups, apart from the transverse connection bars they have very little horizontal reinforcement. They are loaded by three linkage bolts positioned about 380 mm from the bottom edge of the diaphragm sections.

Two of the twelve 38 mm diameter galvanised mild steel linkage bolts on the eastern side of the northern-most pier (Pier B) failed with fractures occurring through threads on the head end of the bolts. There was fine cracking across the bottom corners of some of the diaphragm sections on this pier.

There was spalling at ground level on one of the columns on the second nearest pier to the south end of the bridge (Pier E). 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 soil and gapping at several columns in the second to nearest pier to the south end suggested some lateral spreading of the soil towards the river bank. The pile tops are covered by either soil or water at the abutments and piers and have not yet been inspected.

3.4 Linkage Bolt Testing

The two linkage bolts that failed at Pier B were removed for tensile testing after the earthquake. Difficulty was encountered in removing the bars as they were apparently bent by transverse movements of the superstructure and had become locked in the holes cast through the span diaphragms. They were removed by jacking to loads close to the yield strength of the bars. The recovered lengths were 990 mm long with a 135 mm length of thread at one end. For testing, 40 mm was cut from the fractured end and this end threaded with a 150 mm length of 1.5 inch British Standard Whitworth (BSW) thread to suite the nuts recovered with the bars. The bolts were tensile tested using a special purpose loading frame in a universal testing machine. This frame loaded the bolts under the head and nuts in a similar manner to the load application on the bridge.

After the initial tensile tests a further two tensile tests were carried out to determine the strength of bar heads formed by installing nuts tight against a thread termination. Effectively a head of this type has no loaded length of thread except for the thread immediately under the nut. In the original installation the linkage bars were formed with a head of this type at one end, and a nut and lock nut on the threaded section at the other end. At the head end the bars were cut flush with the end of the nut and a light weld applied to the end of the bar to prevent the nut turning. Two 250 mm long test specimens were cut from the initially tested bar shanks for the bolt head tests. Short sections of 1.5 inch BSW threads were machined at each end to fit the nuts recovered from the bridge. Nuts were installed on both ends and wound tightly to the ends of the threads but were not welded to the bars.

In the initial tests the bars yielded in the original threaded length at a stress of 310 MPa and failed in this section at an ultimate stress of 490 MPa. The bars exhibited good ductility with overall elongation of 35 mm and elongations of about 20% in the original threaded sections. The failure loads in the bar head tests were about 15% higher than in the initial tests. One reason for this higher load is related to the thread run-out of the machine cut thread, which results in a higher stress area at the failure location close to the nut head. The total plastic elongation of the 155 mm long loaded length of the nut test specimens, including the sections of thread under the nuts, was about 6 mm. In the bridge installation this small elongation (less for a single head) may not be sufficient to allow the span inertia loads to become evenly distributed over a number of linkages positioned across the width of the bridge before failure occurred in the most heavily loaded linkage.

The testing showed that the bolts had at least their design strength. The failures at the head ends indicated that because of clamping by relative transverse movements between the span ends on the piers, the heads were loaded more heavily that the threaded ends. Although the overall elongation of a uniformly loaded bolt was good, the elongation in a non-uniformly loaded bolt might only be a few millimetres. Failures in the threaded and nut ends are shown in Figure 7.

Figure 7. Linkage bar failures.
Left – head failure on bridge.
Right – thread failure in tests.

3.5 **Bridge Response**

The friction slabs at the abutments make them very stiff for any direction of loading. However, the connections to both the abutments and piers are quite flexible as 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 s. 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 0.8 s. Although the portal piers are relatively stiff there is significant flexibility in the rubber bearing pads. Assuming simple single-degree-of-freedom (SDOF) response with 5% damping the response accelerations would have been about 0.6 g in both the transverse and longitudinal directions with a corresponding displacement response of about 50 mm. Displacement time-histories for elastic longitudinal response assuming damping values of 5% and 10% are shown in Figure 8.

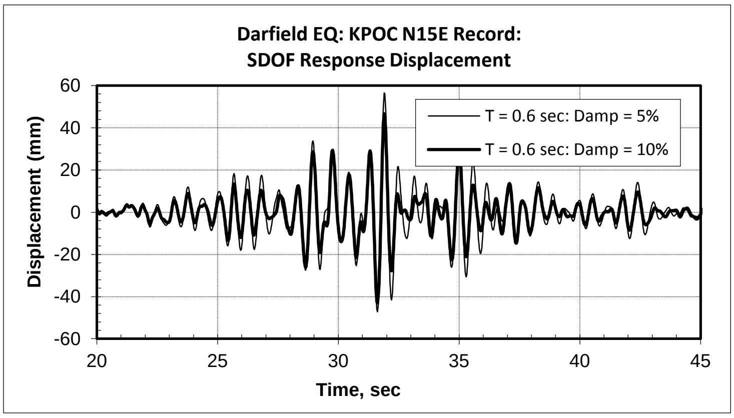

Figure 8. Longitudinal response from KPOC N15E record.

3.6 **Transverse Analysis**

A 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 and it was assumed that the pile tops were at river-bed level. For an assumed steel yield stress of 300 MPa the analysis indicated that the piles 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 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 static analysis was predicted to be a good approximation to the dynamic response of the two highest piers.

3.7 **Longitudinal Analysis**

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 “pushed” 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 “pulled” 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 “pull” inertia load from the superstructure to the abutment. Neither is very stiff so gaps on the pier joints need to allow sufficient movement for the axial and shear deformations in the linkages and bearings to develop before the complete load is transferred to the abutment friction slab and piles. 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 (6 mm Malthoid was used 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.

To investigate the performance of the damaged linkage bolts and span end diaphragms a detailed non-linear model was developed and subjected to incremented static loading. Non-linearity arises because of the closing of the joint gaps, tension only linkages between the spans and the passive soil pressures at the pushed abutment. Details of the stiffness elements used at the abutments and piers are shown in Figure 9. Stiffness parameters used in the model are summarised in Table 2.

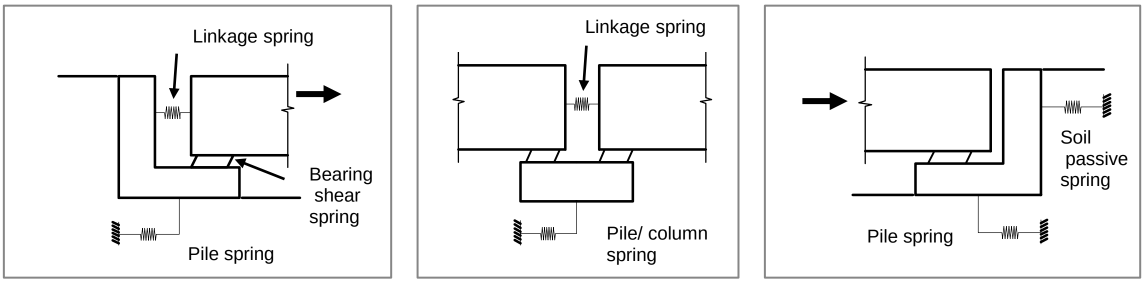

Figure 9. Stiffness elements used in longitudinal response model. Left – pull abutment. Centre – piers. Right – push abutment.

Table 2. Summary of Longitudinal Stiffness Parameters

Stiffness Item	Value MN/m	Comment
Linkage bolts	300	Total for 24 bolts at abutments and piers. Includes rubber washer.
Abutment piles	120	Total for 14/ 510 x 510 prestressed concrete piles.
Friction slab	1400	For pull loading. No sliding.
Combined slab & wall	1900	For push loading at 15 mm displacement.
Bearing pads	45	Total for ten 180 x 460 x19 mm thick pads at each span end. Shear Modulus = 1.05 MPa.
Maximum height pier (8.36 m)	4.7	At underside of deck including column and pile flexibility.
Minimum height pier (6.83 m)	7.0	At underside of deck including column and pile flexibility.

Stiffness of the abutment friction slabs under pull loads was estimated from experimental model study results reported by Yeo (1987). Abutment wall passive stiffness was estimated using the empirical expression given by Kahalili-Tehrani and Taciroglu (2010). Pile stiffnesses were calculated using the elastic continuum solutions presented by Pender (1993).

The total weight of the superstructure was estimated to be 39,000 kN. Pier weights varied between 2,000 to 2500 kN.

There is uncertainty in the joint gap values but by varying them and carrying out repeat analyses a range of possible values for the proportion of load carried on the pulled and pushed abutments was obtained. For the analysis results presented below the pier and abutment joint gaps were taken as 12 mm and 6 mm respectively.

Results from the analysis are summarised in Figure 10 which shows the linkage forces at the pulled abutment (Abutment A) and the closest pier (Pier B) plotted as a function of the spectral acceleration acting on the superstructure.

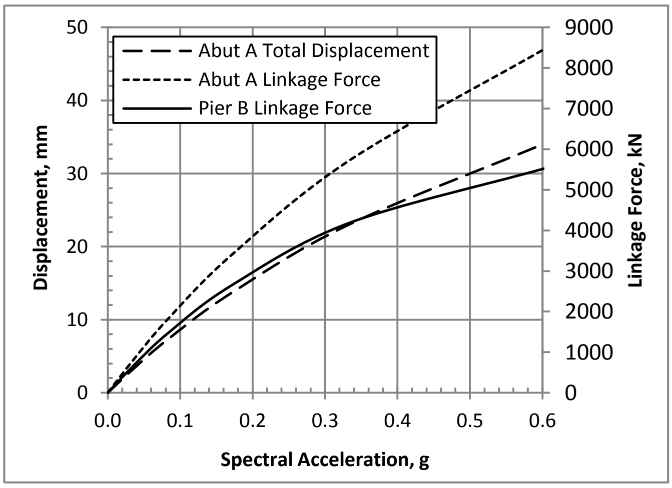

Figure 10. Longitudinal analysis results for linkages.

Also plotted in Figure 10 is the total displacement of the pulled abutment.

Based on the tensile test results the total yield and ultimate strength capacities of the linkage bolt systems at both the abutments and piers were 6,700 kN and 10,700 kN respectively (excluding the higher strength of the bolt head section).

At a response acceleration of 0.6 g the proportion of the superstructure inertia load resisted by the pulled and pushed abutments was 41% and 54% respectively (5% carried by the piers).

The strength of the abutment diaphragm sections was estimated using a two-dimensional plate model loaded by the three linkage bolts in each diaphragm section between the beams. This indicated that a shear friction type of failure would occur at the lower edge when the bolts were loaded to about 65% of the linkage bolt yield force.

3.8 Comparison of Analyses With Observed Performance

In the transverse direction the bridge performed somewhat better than expected as cracking and spalling damage would have been expected in the high piers at about the predicted response level of 0.6 g. The two separate superstructures, although nominally identical, will not vibrate in phase and this may have significantly reduced the peak dynamic response.

The longitudinal analysis results indicated that the abutment linkage bolts would yield at a response acceleration of about 0.45 g and that the abutment diaphragms would fail in shear at a response acceleration of about 0.3 g. Because of the failure of the diaphragms it was not possible to positively identify whether the linkage bolts at the abutments had reached their yield load. However, the analyses indicated that the load in the linkage bolts would have been sufficient to fail the diaphragms.

At a response acceleration of 0.6 g the linkage bolts at Pier B were estimated to be loaded to about 52% of their failure load. However, they are also loaded by rotation of the span ends under transverse response and there may have been a large variation in linkage bolt loads across the large total width of the bridge. The failure at the head end of the bolts was thought to be related to the bolt shanks locking-up under transverse response. There would have been insufficient tensile ductility for the loads in the bolts to be evened-up before they failed.

4 CONCLUSIONS

- Overall the earthquake performance of the SH bridges was good with only twelve receiving significant structural damage out of a total of about 50 bridges that received strong shaking in one or both of the two main earthquake events. This damage did not close any of the bridges to traffic and would be relatively straightforward to repair.
- Preliminary analyses carried out on all the damaged bridges showed that the damage was generally consistent with predictions.
- More detailed analyses are currently being carried out on the damaged bridges. The detailed analysis of the Kaiapoi Railway River Bridge has provided valuable information about the design and performance of linkage bar systems. Detailed assessment of the other damaged bridges is also expected to provide a better understanding of bridge response and performance under strong ground shaking.

5 ACKNOWLEDGEMENT

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