PRELIMINARY FINDINGS ON PERFORMANCE OF BRIDGES IN THE 2010 DARFIELD EARTHQUAKE

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SUMMARY

On September 4, 2010 a M 7.1 earthquake occurred with an epicentre near the town of Darfield 30-40 km west of the Christchurch CBD. In the days following the earthquake inspections were carried out on highway, road City Council and pedestrian bridges in the Canterbury area. This paper details the preliminary findings based on visual inspection of about fifty five bridges. The paper comprises information supplied by consulting engineering firms which were also directly involved in the inspections soon after the earthquake.

INTRODUCTION

The immediate districts surrounding the fault of the Darfield earthquake contain more than 800 road, rail and pedestrian bridges. Overall, bridges have suffered little structural damage, and this damage was mostly limited to the areas which suffered extensive liquefaction and lateral spreading. A number of factors contributed to this overall good performance. Firstly most bridges in the Canterbury area are small to moderate spans; such spans are recognized to generally exhibit a more sturdy seismic response, due largely to their symmetry and limited reactive mass. Secondly, like many buildings and other infrastructure in the areas subjected to the earthquake excitations, bridges were generally designed to resist forces substantially larger than the demands imparted by this particular earthquake.

Based on visual inspections by the Natural Hazard Platform (NHP) Bridge Research Group of about 55 bridges, and on information provided by external institutions who undertook parallel inspections, the following summarizing comments can be made:

- Eight road bridges were closed in the days following the earthquake. Five of these road bridges were damaged to the extent that they remained closed for at least five days after the earthquake while temporary repairs were made. With the exception of one of the eight closed bridges, none suffered major damage to the superstructure, and the reason of closure was generally due to induced damage to the area surrounding the bridge, i.e. ‘approach damage’.
- Six pedestrian bridges suffered severe structural damage. These bridges were not designed with the same strengths and stiffness of road bridges, and as such they could not resist the high demands induced by lateral spreading of the riverbanks due to soil liquefaction. The six bridges were damaged to the extent that most will need replacement.
- The survey undertaken only witnessed one railway bridge which required major repairs.
- Highways bridges were generally unaffected, aside from one case where the approach spans subsided and cracked, reducing access to the bridge.
- Bridges close to the fault did not suffer any damage in places where soil liquefaction did not occur. For example, the State Highway 1 bridge across the Selwyn River, less than 5 km from the fault, and the railroad bridge adjacent to it, remained operational.

Bridges were graded similarly to buildings. A green label/tag meant no visible damage, a yellow label stands for safe for use with visible damage, while red label/tags were given to bridges prone to collapse (unsafe for use). Location of red and yellow bridges can be seen in Figure 1.

Initial response of roading authorities

Five governmental authorities own and operate the majority of the bridges within the area where the Modified Mercalli Intensity Level was in excess of MM6 [1]. The five authorities are: New Zealand Transport Agency (NZTA), ONTRACK (ex KiwiRail), Christchurch City Council, Selwyn District Council, and the Waimakariri District Council. The initial responses of the different authorities were to delegate preliminary visual inspections to the consulting engineering companies, in particular:

- NZTA: Contracted Opus International Ltd to perform an initial and more detailed reconnaissance. By midday, of the day of the earthquake, all the critical state-highway links within the suspected at risk areas had been assessed.

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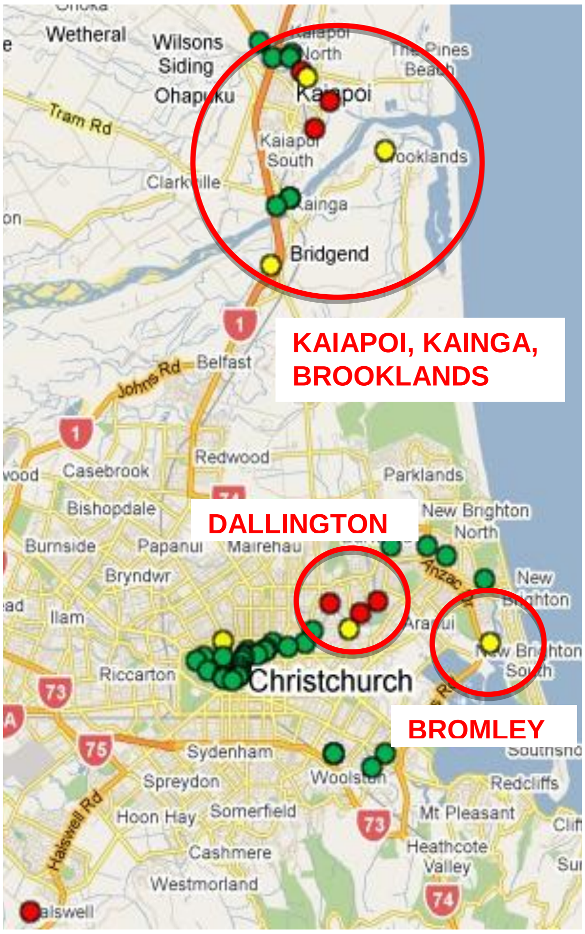
No damage was observed warranting the complete closure of any bridge.

- ONTRACK: Contracted Novare Design to undertake the assessment of the railway network bridges. There were some bridges damaged to the extent of being an additional reason for closure of railway lines.
- Christchurch City Council: Contracted Opus International to undertake the reconnaissance of the city's bridges. Opus inspected approximately 300 bridges within the city, and 180 bridges on the Banks Peninsula (Figure 2). Less than 3% of the bridges had damage that lead to closure.
- Selwyn District Council: Undertook the inspection of the 150 bridges in their jurisdiction in house. Two bridges in total were closed due to earthquake damage.
- Waimakariri District Council: Employed MWH to undertake the inspection of the 200 bridges in the district. Much of the bridge damage was within the township of Kaiapoi.

Following some considerations on seismicity and soil properties, a detailed overview of the most common damage detected during inspections is reported. Damage will be classified in deck and superstructure, piers and abutments, approach, and lifelines.



No visible Damage (green)

Safe for use, with visible damage (yellow)

Bridge unsafe for use (red)

Figure 1: Map showing inspected bridges approximate areas of high levels of liquefaction.

SEISMIC DEMAND AND SOIL CONDITIONS

Seismicity

The overall lack of damage to the structural systems of the short to mid-span bridges can be explained by closely inspecting the acceleration response spectra of typical sites in Christchurch and Kaiapoi areas. Figure 3 compares the NZS1170 Design Spectra for site soil classes D and E [2] against the horizontal acceleration response spectra of six strong motion stations within the Christchurch and Kaiapoi areas. For periods in the range of 1.50 seconds and less, the response spectral acceleration is on in average below the design spectral acceleration; whereas for periods above 1.5-2.0 seconds the spectral acceleration exceeds the design spectrum level. The approximate period range of a Christchurch short to mid-span road bridge is indicated by a circle in Figure 3.

Within this period range, it is quickly seen that the actual spectral acceleration is considerably less than the values that a bridge should have been be designed for.

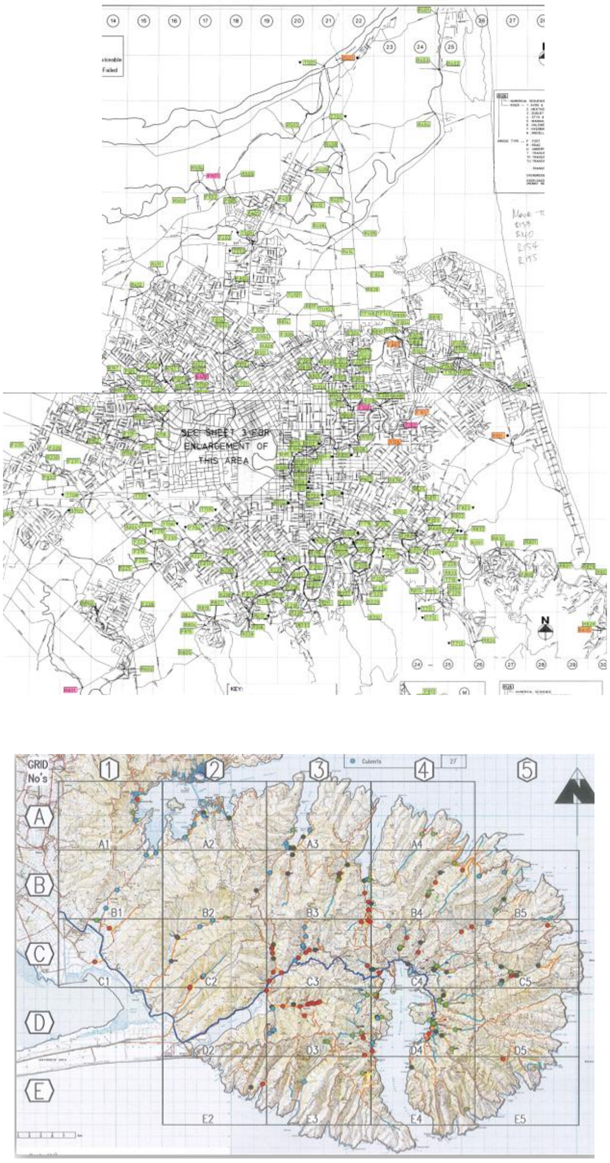


Figure 2: Christchurch City Council and Banks Peninsula bridges inspected by Opus [Courtesy of M. Cowan, Opus International].

Correlation between structural damage and liquefaction

A strong correlation was observed between areas where bridge damage was observed, and areas where liquefaction occurred. This correlation is shown in Figure 1. The areas enclosed in circles are the areas where extensive liquefaction occurred.

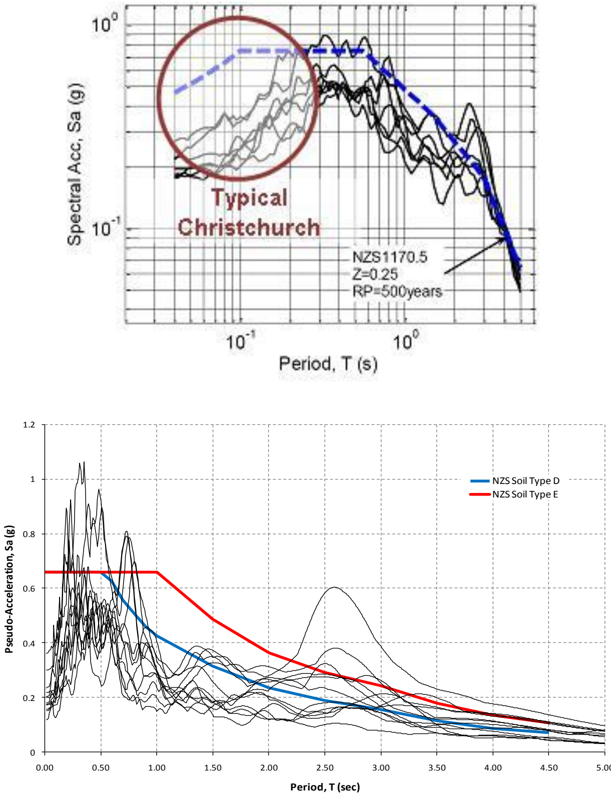


Figure 3: Horizontal Acceleration Response Spectra (ARS) of six seismometer stations in the Christchurch and Kaiapoi areas against the NZS1170 Design Response Spectra (DRS) for Soil Classes D and E [2] (Generated by M. Anagnostopoulou based on Geonet data).

Fourteen of the fifteen damaged bridges were inside the areas that experienced extensive liquefaction. The detailed liquefaction assessment map developed by Environment Canterbury [3] prior to the earthquake confirms that the major bridge damage occurred in the high sensitive liquefaction areas.

This strong geographical link between location of damaged bridges, and location of extensive liquefaction, lead to the immediate realization that lateral spreading, which is closely associated with liquefaction, would be the primary action on bridges leading to damage. The damage witnessed, as the following sections report, confirmed this immediate hypothesis.

DAMAGE TO ROAD BRIDGES

Bridge Deck and Super-Structure

Overall the road bridges exhibited little damage to the beam and deck elements. The one case of severe super-structure damage to a road bridge was to a short-span masonry arch bridge. The bridge was on Saby's Road, approximately 31 km from the epicentre. The bridge was subjected to a large longitudinal compression force due to lateral spreading of the river bank. The bridge essentially acted as a compressive brace between the two river banks, and unable to resist the force of the riverbank, formed a three hinge mechanism as shown in Figure 4. Two culverts were installed to temporarily replace the damaged masonry arch.

Figure 4: Saby's Road Bridge arch damage [Photo by A. Palermo].

Bearing to abutment connection

A different type of observed bearing damage was the deformation of the rubber isolation bearing pads due to large relative lateral movement between the deck and the abutment/pier. The most dramatic example of this occurred at South Brighton Bridge (Bridge Street, South Brighton, Christchurch). At this bridge the bearing pads deformed to a strain of approximately 30%, and also slipped approximately 100-150 mm (Figure 5a).

The third bearing reaction to the earthquake witnessed in road bridges was the complete crushing of the bearing. This occurred in the Old Waimakariri Motorway Bridge, where the mortar bearing pads were extensively fractured due to excessive movement of the abutment (Figure 5b).

(a)

(b)

Figure 5: (a) Deformed and displaced bearing on South Brighton Bridge [Photo by T.Cristini]; (b) Crumbled mortar bearing pad on the Old Waimakariri Motorway Bridge [Photo by A. Scott].

(a)

(b)

Figure 6: ***(a) Horizontal crack on pier of South Brighton Bridge [Photo by Lucas Hogan]; (b) Evidence of transverse movement on the Old Waimakariri Motorway Bridge [Photo by A. Kivell].***

Piers

A small number of road bridges exhibited flexural cracking of the piers (Figure 6a). These cracks are part of the expected structural failure mechanism of the bridge [4] in such an event, and due to the limited damage they do not immediately compromise the structural integrity of the bridge.

However unrepaired cracks might become a potential threat to the design life of the bridges close to marine environment. The cracking pattern provides an accelerated pathway for chloride ingress and hence compromises the overall durability of the structure. Recent studies confirmed a potential loss of the seismic performance, i.e. strength and ductility under aggressive environments [5], [6].

Some pier damage was observed on the twenty seven-span Old Waimakariri Motorway Bridge. This was the only bridge in which notable transverse seismic response was observed; as shown by the 20 mm gap between the pier and the ground in the transverse direction (Figure 6b). Cracking due to longitudinal movement was also observed on some piers.

Interestingly, external piers on the flood zone suffered more cracks than those in the current normal river flow path. This was caused by river scouring increasing the height and therefore reducing the stiffness of central piers, thus inducing more load on the shorter and stiffer external piers.

A complimentary way of explaining the phenomenon is that due to the high stiffness of the deck, all the piers displace at the top approximately of the same amount, the shorter piers need to rotate more than the longer piers to reach an equal top displacement. This increased rotation results in an increased section curvature, which in turn leads to increased level of cracking. The crack pattern consisted of one unique crack running though the pier section.

Abutments

Abutment damage observed included rotation and translation of the entire abutment, abutment cracking as well as, flexural cracks on the exposed abutment piles. This was caused by the soil structure interaction phenomenon activated by laterally spreading approach slopes in areas suffering from liquefaction. This effect was seen only on bridges with deep abutments or closely spaced abutment piles (Figure 7a and Figure 7d). These types of abutments are in contrast to the current design philosophy for lateral spreading banks that uses a shallow beam type abutment, with a small number of deep piles. This allows a laterally spreading slope to spill past the abutment.

Many new bridges have tie-rods inserted between the deck and the abutment. There were two observed situations where the action of these tie-rods was not very effective. The first was observed on the Christchurch Northern Motorway Kaiapoi River Bridge (Figure 7b). This bridge experienced large longitudinal tension forces which fully activated the tie-rods. These forces were transmitted from the superstructure’s I-beams, and through a coupling beam and the tie-rods, to the abutment. The coupling beam was the weakest link in this load path, and experienced significant concrete spalling, and cone type pull out.

Tie-rods were also installed in a similar fashion at the South Brighton Bridge (Bridge Street, South Brighton, Christchurch).

In contrast to the Kaiapoi River Bridge, these rods were not tension activated by seismic mass of the bridge but by the longitudinal push action of the soil. This caused a 5° rotation of the abutment and a gap to form between the tie-rod bearing plate and the coupling beam of approximately 100 mm (Figure 7c).

(a) **(b)**

(c) **(d)**

Figure 7: ***(a) Vertical cracks on Gayhurst Road Bridge Abutment [Photo by F. Sarti]; (b) Tie-rods into abutment on the SH1 Kaiapoi River Bridge [Photo by A. Scott]; (c) Abutment at South Brighton Bridge [Photo by G. Paganotti]; (d) Flexural crack on the abutment pile of Pages Road Bridge near Brooklands [Photo by M. Bruneau].***

Approach

Seven out of eight road bridges were closed because of the damage to the approaches. In all cases this was caused by lateral spreading, which caused the slope to move across and downwards towards the river. As the abutments were generally supported on piles, the bridges themselves remained relatively unmoved.

This differential movement led to the formation of a very large vertical settlement between the bridge deck and the approach at the River Road Bridge (River Road, South Lincoln). On this bridge the alignment was level before the earthquake; afterwards an evident change in elevation of 500 mm between the unmoved bridge deck, and the approach (Figure 8a) has been registered. As elaborated in the "Soil Structure Interaction" section of this report, cracks in the approaches parallel to the axis of the bridge provide evidence of the resistance against lateral spreading induced by this short monolithic span (Figure 8b).

(a)

(b)

Figure 8: ***Relative vertical movement between approach and deck at: (a) River Road Bridge, Lincoln [Photo by L. Wotherspoon]; (b) Gayhurst Road Bridge [Photo by L. Hogan].***

In some instances, while the bridges remained essentially intact, the approach spans partially or totally failed, making access to the bridge either more difficult or impossible. For example, the twin continuous bridges at the Chaney's overpass on State Highway 1 north of Christchurch were found to be structurally sound, and tied to their abutment walls to prevent unseating there. However due to liquefaction of the site surrounding the bridge (Figure 9a and 9b); the approach to the southbound lanes of State Highway 1 settled by a few inches (Figure 9c). After a brief closure for inspection, this busy route was reopened with signage reducing the speed to 30 km/h (down from 100 km/h) for the safety of motorists.

(a)

(b)

(c)

Figure 9: ***Chaney's Overpass: (a) Extensive liquefaction around piers [Photo courtesy of Anthony Rooke, OPUS]; (b) cracking at approach [Photo courtesy of Anthony Rooke, OPUS]; (c) Deck settlement due to surrounding liquefaction [Photo by M. Anagnostopoulou].***

DAMAGE TO PEDESTRIAN BRIDGES

Bridge Deck and Super-Structure

Almost all pedestrian bridges which were closed due to the earthquake suffered extensive damage to the superstructure. The bridge failures occurred in regions that exhibited extensive liquefaction, and had associated lateral spreading of the riverbanks. Lateral spreading played a crucial role as the horizontal inward movement of riverbanks induced an additional and unexpected longitudinal compressive force through the super-structure. This phenomenon led to several different failure modes which are related to the structural typology of the bridge:

- Global lateral torsional buckling, as seen in the steel truss Avonside Drive pedestrian bridge (Figure 10a).

- Plastic hinging at mid-span, as seen at the Dallington reinforced concrete arch pedestrian bridge (Figure 10b).
- Longitudinally rocking of towers, at the Mandeville pedestrian suspension bridge in Kaiapoi (Figure 10c), and consequent opening-up of existing hinges that consisted of in-contact horizontal wood splices (and that were not effectively restrained from opening up during the rotation of the towers).
- Plastic hinging near abutments, at the concrete prestressed tee-beam pedestrian bridge at Porritt Park.

(a)

(b)

(c)

***Figure 10:** (a) Global lateral torsional buckling at Avonside Drive Pedestrian bridge [Photo by M. Anagnostopoulou]; (b) Plastic hinge at apex of arch of Dallington Pedestrian bridge [Photo by M. Bruneau]; (c) Failure mechanism of the Mandeville Pedestrian Bridge [Photo by M. Anagnostopoulou].*

Bearings

The pedestrian bridges which were near to collapse did not have any form of lateral isolation between the super-structure and the abutments and/or piers. This resulted in major damage in the connections between the bridge deck and the abutments and piers. Observed modes of failure included fracturing and yielding of connection steel rods (Figure 11a); spalling of concrete around the connection; uplifting of the deck off the pier; rotation at the interface between the deck and the pier (Figure 11b); and total horizontal translation of the bearing connector off several piers.

a)

b)

c)

d)

***Figure 11:** (a) Yielding lug, with concrete spalling on the Avonside Drive Pedestrian Bridge abutment; (b) Bearing lifted off pier on Avonside Drive Pedestrian Bridge; (c) Bearing translated off pier on Courtenay Stream Pedestrian Bridge, Kaiapoi [Photos by F. Sarti]; (d) Pile split along the vertical plane on Courtenay Stream Pedestrian Bridge in Kaiapoi [Photo by M. Anagnostopoulou].*

Piers

Almost all pedestrian bridges did not suffer any damage to their piers with the exception of a disused historic railway bridge. In fact this bridge crossing Courtenay stream in Kaiapoi lost support over part of its length (Figure 11c). Many of those piers split along the vertical plane in which bolts were used to connect the spans to the pier (Figure 11d). Evidence of the lateral pressure on the piers was provided by the large gaps between the soil and piers on one side of the piers.

Abutments

Two pedestrian bridges showed visible damage to the abutments. In both cases, significant lateral spreading had occurred on at least one river bank. It is worth noting that of all the damaged pedestrian bridges observed, these two were the only bridges with a concrete main superstructure element. At the Dallington Pedestrian Bridge, a horizontal crack split the entire abutment at each end of the bridge (Figure 12a). The same bridge also exhibited severe damage to the pile caps of the abutment on the side of the river that was subjected to extensive lateral spreading. Figure 12b shows this damage while the repair is in progress. The other case, Porritt Park Pedestrian Bridge, experienced large rotation and translation of the south abutment (Figure 12c). This abutment sits on six small piles approximately 200 mm diameter. It is likely that the piles do not extend very deep. As such, they provided minimal lateral stiffness to the system, and only served to increase the surface area the soil wedge interacted with, thus exacerbating the problem. This rotation resulted in extensive cracking at the abutment face, as well as hinging at the deck-abutment connection.

(a)

(b)

(c)

***Figure 12:** (a) Horizontal shear crack seen on entire abutment of Dallington Pedestrian Bridge [Photo by M. Le Heux]; (b) Wing of pile cap of Dallington Pedestrian Bridge, showing exposed crack, and repairs [Photo by M. Le Heux]; (c) Rotation and plastic hinge formation at Porritt Park Pedestrian Bridge [Photo by L. Hogan].*

EFFECTS ON LIFELINES

The areas which the closed bridges gave access to all had alternative access routes. Even so, the local authorities had restored access to all the damaged road bridges within a week of the earthquake. One interesting case is the Kianga Road Bridge in Brooklands, where the bridge and approach road were repaired within five days of the earthquake, but the police requested the bridge remain shut to help keeping looters away from abandoned houses in the area.

In general over Christchurch, there were several minor leaks in pipes on bridges caused by the earthquake. However the Kianga Road Bridge showed the only major failure of a pipe on a bridge when a sewer pipe fractured, contaminating the river with raw sewage (Figure 13a). Incidentally, flexural cracks were observed at this bridge on the abutment piles that were exposed as a consequence of soil lateral spreading (Figure 7d).

a) **b)**

***Figure 13:** (a) Sewage leak at Kianga Road Bridge [Photo by A. Palermo]; (b) Crack in approach pavement running in the longitudinal direction of the Dallington Pedestrian Bridge [Photo by L. Hogan].*

LONGITUDINAL SOIL STRUCTURE INTERACTION

The damage caused by the interaction between the bridge and the surrounding ground led to some of the more interesting observations of this fieldwork. In situations where the riverbanks suffered from lateral spreading, the bridge structure itself acted as a prop across the river. This compressive strut in effect provided resistance to the lateral spreading and altered the soil crack pattern. This resistance was aided by the abutments.

The effect of this phenomenon on the structure was that in some cases this longitudinal compressive force transferring through the bridge was greater than the axial capacity of the deck or other structural components and lead to failure of the bridge structure. This was more evident in the pedestrian bridges.

Conversely, the effect of this phenomenon on the soil was that spreading of the slope around the abutment lead to soil gapping from differential movement, passive soil wedge failure, and large cracks forming on the approach running parallel to the longitudinal bridge axis. The cracks in the approach running longitudinally with the bridge were unique, in that all other cracks in the surrounding area ran parallel with the river (i.e. transversely to the bridge). These cracks were potentially the cause of damage to the services in the road running transversely to some bridges as highlighted by the Dallington Pedestrian Bridge (Figure 13b).

SUCCESS OF RETROFITTING AND LIFELINE PROGRAMS

Several bridges in the Canterbury region have undergone some form of seismic retrofitting during the past 15 years in particular.

One of the most widespread retrofit programs was undertaken by Transfund New Zealand (now incorporated into NZTA) [7], which saw the installation of tie-rods and devices such as the transverse shear key shown in Figure 14. Similar devices were installed on bridges on most of the regions highways.

The post-earthquake investigations have not been in depth enough to ascertain if these devices were activated during the seismic shaking. However, it is positive that no bridges that were retrofitted under this program were structurally damaged. Lateral spreading had been identified as an issue on some key life-line bridges. The Ferrymead Road Bridge is one such case, where the concerns due to liquefaction were so great that the entire bridge was replaced. The abutments of the new bridge were specifically designed to accommodate lateral spreading. The bridge was not damaged during the earthquake, and although this was not a design level event, the strong performance of the bridge, helped to validate New Zealand governing bodies' commitment to life-line projects.

***Figure 14:** Transverse shear key and tie-rod retrofit of SH74 port hills overpass [Photo courtesy of Anthony Rooke, OPUS].*

The greatest success story for retrofitting and life-line programs to come out of this earthquake is the 1960s built Dallington Pedestrian Bridge (Figure 15). Although pedestrian bridges are not usually identified as key life-line assets, this bridge was designated as a key-link due to the 66 kV electricity cables that pass over the Avon River. Orion New Zealand, the owners of the electricity cable and the bridge (at the time of the retrofit the bridge and cables were actually owned by Southpower), undertook an in-depth geotechnical and structural investigation into the expected performance during the earthquake. This investigation and subsequent retrofit took place circa 2000 (Figure 15).

***Figure 15:** Pedestrian Bridge in Dallington undergoing earthquake damage repairs [Photo by M. Le Heux].*

Although out of the scope of this paper, to the credit of the geotechnical engineer commissioned with the investigation, the lateral spreading predictions he made matched well with the immediate visual observations made after the earthquake event. The investigation concluded that the passive earth pressure that would be placed on the bridge abutments under the predicted lateral spreading would exceed the longitudinal compressive capacity of the bridge. This excessive lateral pressure was partly due to the numerous but shallow abutment piles acting as effectively a retaining wall against lateral spreading during the earthquake event. The retrofit solution used, was the placement of raked wing piles on the abutment driven down into to the stronger soil. This retrofit solution is depicted in Figure 16. Retrofitting was also carried out on the cable cradles on the approach to the bridge to reduce the level of deformation that the 66 kV cables would be placed under during an earthquake.

The retrofit scheme was considered a success. Although plastic hinging occurred at the apex of the bridge (Figure 10b), and considerable abutment and pile cap cracking also occurred, the bridge did not collapse, and the electricity cables remained operational. The bridge would have most likely collapsed and caused disruption to the power supply if no retrofitting had been carried out.

a) **b)**

***Figure 16:** (a) Dallington pedestrian bridge with retrofit work exposed, showing 66 kV cable cradle entering bridge [photo by M. Le Heux]; (b) close up of the new raked pile detail [Courtesy of John Mackenzie, ELMAC Consulting Engineers].*

CONCLUSIONS

Overall bridge structures performed very well in the 2010 Darfield Earthquake and subsequent aftershocks, confirming the design expectations [8].

The good performance is attributed to two facts: a) many of the bridges in the region did not experience design level shaking, b) most Christchurch City Council road bridges built in 40s, 50s were overdesigned. However, despite the lower earthquake demand, compared to design level, significant damage occurred on the approaches due to liquefaction and lateral spreading at the riverbanks. This certainly caused unexpected disruption which might be mitigated through damage-free approach solutions.

Pedestrian bridges performed badly compared to road bridges due to less attention to seismic issues during the design and lack of knowledge of geotechnical problems. However, as witnessed for the Avon River Bridge, if proper retrofit interventions are planned structural failure can be avoided despite extensive damage to approaches. The authors believe that similarly to buildings where damage-free solution are targeted for primary [9] and secondary structural members [10], future bridge structures should target advanced damage-free solutions [11] which allow to preserve the integrity of the structure and surroundings after the earthquake.

FURTHER DEVELOPMENTS

The report presented here is representative of the first stage of the investigation into the performance of bridges to the Darfield Earthquake, namely the initial data gathering stage. The planned sequence of investigation from here is to:

1. Condense, combine, and collate the data received from all the different sources into one transferrable and complete dataset that will be presented as an available as a resource in to the NZSEE clearing house. The dataset will also include life-lines and network data. This will be done through the collection of data such as damage to critical services, and transport disruptions.
2. Analyse the data-set and choose case-study bridges for which a more detailed investigation will be carried out on. The chosen case studies will be the bridges from which it is evaluated significant lessons of behaviour of bridges under lateral spreading may be gleaned. Detailed field and analytical investigations will be carried out on the case-study bridges.
3. Utilize the case-studies to evaluate current guidelines for liquefaction and lateral spreading analysis of bridges [12].
4. Network modelling of post-earthquake traffic disruption.
5. Conceptual and analytical review of potential post-earthquake damage-free mitigation strategies to bridges and networks.
6. Investigate alternative economical solutions to damage-free pedestrian bridges which are built in zones prone to lateral spreading and liquefaction.

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