LESSONS LEARNT FROM 2011 CHRISTCHURCH EARTHQUAKES: ANALYSIS AND ASSESSMENT OF BRIDGES

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SUMMARY

On 22 February 2011 the M_w6.2 Christchurch earthquake occurred with an epicentre less than 10 km from the Christchurch Central Business District (CBD) on an unknown buried fault at the edge of the city. The majority of damage was a result of lateral spreading along the Avon and Heathcote Rivers, with few bridges damaged due to ground shaking only. The most significant damage was to bridges along the Avon River, coinciding with the areas of the most severe liquefaction, with less severe liquefaction damage developing along the Heathcote River. Most affected were bridge approaches, abutments and piers, with a range of damage levels identified across the bridge stock. In the days following the earthquake, teams from various organizations performed inspections on over 800 bridges throughout the affected Canterbury region. This paper details the preliminary findings based on visual inspections and some preliminary analyses of highway and road bridges. The paper comprises information supplied by consulting engineering firms which were also directly involved in the inspections soon after the earthquake.

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INTRODUCTION

The surroundings around Christchurch contain more than 800 road, rail and pedestrian bridges. Over half of the bridges are integral reinforced concrete or hybrid, i.e. precast concrete beams and cast in place substructure (piers, foundations), with the remainder of timber and steel construction. Many reinforced concrete bridges subjected to strong shaking were designed and constructed prior to 1970 using working stress design (WSD) and earthquake loads based on a 0.1g 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 0.18g if sized and reinforced to WSD limits.

The 22 February 2011 moment magnitude (M_w) 6.2 resulted in strong ground shaking in the central and eastern regions of Christchurch, with the majority of significant bridge damage focused in this region. Most of the damage was a result of liquefaction and lateral spreading of the river banks, with very few examples of significant bridge damage on non-liquefiable sites. A number of bridges suffered non-structural damage such as slumping of abutment aprons and fracture of deck drainage pipes. Overall, bridges suffered little structural damage compared to other structures such as residential houses and commercial buildings.

In spite of the expected damage threshold level being much lower than the estimated bridge response accelerations in the earthquakes, only a few bridges suffered significant visible structural damage as a result of ground shaking. Their typical monolithic construction and axial strength also meant older designs were able to resist the axial demands placed on the structure due to lateral spreading, even though they were not specifically designed for these loads.

Following the earthquake, the bridge stock was inspected by the network consultants and researchers to establish safety conditions, repairs that were required to enable traffic to flow and document damage. 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.

This paper presents a summary of observations from the field and some initial analyses on a selection of the most severely damaged bridges in central and eastern Christchurch.

SEISMIC DEMAND AND SOIL CONDITIONS

The M_w6.2 2011 Christchurch earthquake had an epicentre less than 10km from the Christchurch CBD between Lyttelton and the south-eastern edge of the city. The close proximity and shallow depth of this event resulted in higher intensity shaking in Christchurch relative to the 2010 Darfield earthquake in September 2010. Further aftershocks occurred during the following months, with one of the strongest, the M_w6.0 on 13 June 2011, with an epicentre again on the south-eastern edge of the city [3].

Horizontal Peak Ground Accelerations (PGA) (geometric mean of the horizontal components) during the 2011 Christchurch earthquake at the strong motion stations across the city are summarized in Figure 1. Horizontal PGAs were 0.37g-0.51g in the Christchurch CBD. Significant vertical accelerations were also registered in this earthquake. In the Port Hills area a horizontal PGA of 1.41g was recorded near the epicentre at the Heathcote Valley Primary School (HVPS) strong motion station. Strong motion records indicated that most of the bridges within 10 km of the Christchurch earthquake were subjected to horizontal PGA's of 0.25g to 1.4g [3]. 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 earthquake [7].

Ground motions near the source have different characteristics than those far from the epicentre. Significant vertical accelerations were registered in this earthquake, and can have larger amplitude than those of the horizontal components. Vertical ground motions that are associated with propagating compressive waves usually have higher frequency content than horizontal ground accelerations. Especially, in liquefied soil close to the surface, the amplitude of vertical component will likely be amplified while horizontal components will be reduced, as was observed in Port Island during the 1995 Kobe earthquake [2]. A large part of the man-made Port Island sank more than 40 cm due to liquefaction. Horizontal and vertical ground accelerations recorded at depths of -83 m, -32 m, and -16 m and at the ground surface clearly showed that the liquefied soil had amplified the vertical component while the horizontal components which are associated with shear waves were reduced at shallow depths. In the Christchurch earthquake only ground surface accelerations were measured. However, a similar development of ground motions in liquefied soil to those of Port Island can be expected.

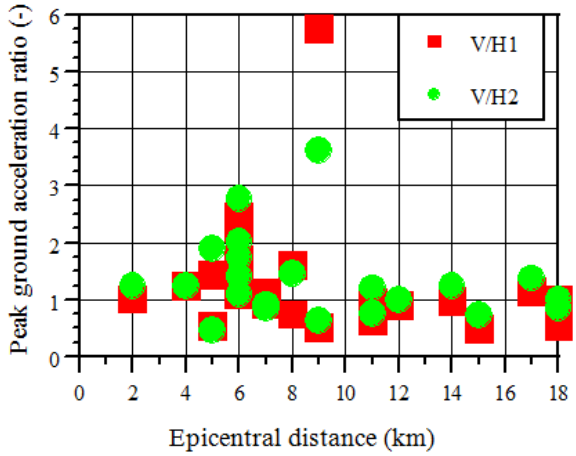


Figure 1: ***Influence of the epicentral distance on the peak ground acceleration development in the 2011 Christchurch earthquake.***

Figure 1 shows the ratios V/H1 and V/H2 of the peak vertical ground acceleration V to the largest ground acceleration in the two horizontal directions, H1, H2 respectively. The development of these ratios with the epicentral distance indicates that within the zone considered (18 km) V can still be larger than H. Larger V/H ratio does not automatically mean that a structure will suffer damage due to the vertical ground motions, because H can be small. In the 2011 Christchurch earthquake, however, in the case of V/H1 = 2.45 and V/H2 = 2.77 at the epicentral distance of 6 km the peak vertical ground acceleration has indeed a large value of 1.6 g. At a distance of 9 km V/H1 even reached the value of 5.75 (Figure 1).

Acceleration response spectra of typical sites from the 2011 Christchurch earthquake are compared with the New Zealand Design Spectra [6] for site soil class D in Figure 2. Horizontal acceleration response spectra for five strong motion stations (CCCC, HPSC, HVSC, PRPC, SHLC) close to bridges damaged during the Canterbury earthquakes are considered for the comparison. Within the period range of 0-0.8 seconds, a representative period range of many New Zealand road bridges, spectral acceleration values in eastern Christchurch were much higher than the 500 year return period level often used for building design. At a number of bridge sites the short period spectral accelerations exceed the 1,000 year return period level currently used as the minimum design level for most city and State Highway bridges. (State Highway Bridges on important routes are designed for a 2,500 year return period level of earthquake shaking [4]).

Figure 3 indicates the locations of the road bridges most severely damaged in the 2011 Christchurch earthquake, along with the zone of moderate to severe liquefaction damage. Much of central and eastern Christchurch area was identified as having high liquefaction susceptibility, with most of this area affected by some level of liquefaction following the Christchurch earthquake. Most of the damaged bridges were located along the Avon River, coinciding with the zone of moderate-severe liquefaction. Compared to the Avon River, bridges crossing the Heathcote River suffered much less damage, with much smaller regions of moderate-severe liquefaction damage even though these areas were closer to the fault rupture than most of the Avon River bridges. Apart from the Ferrymead Bridge (-43.5584, 172.7086) at the mouth of the Heathcote River, all bridges were either undamaged or suffered only minor damage. Figure 2 confirms the above mentioned damage trend; the short period spectral accelerations were very high at several of the strong motions (PRPC, HVSC) in the south-east of the city.

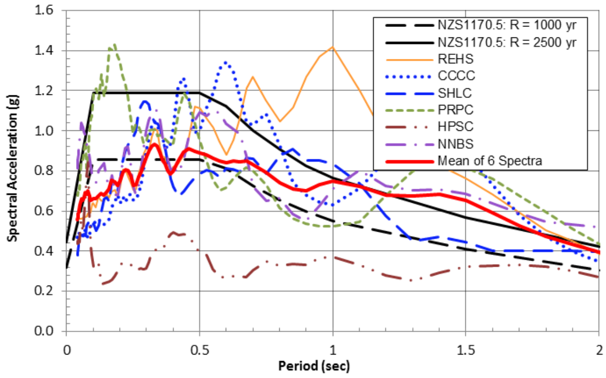


Figure 2: ***Response spectra of the geometric mean of the horizontal accelerations at strong motion stations in central and eastern Christchurch compared to NZS1170.5 design response spectrum for Christchurch, site subsoil class D. Four letter symbols represent different strong motion stations (see Figure 3 for locations).***

Figure 3: ***Overview of Christchurch and the surrounding region, indicating locations of a selection of damaged bridges, strong motion stations, and regions of moderate-severe liquefaction following the 2011 Christchurch earthquake.***

Figure 4: ***Selection of damaged bridges in the CBD area. From left to right: Colombo Street Bridge; Bridge of Remembrance; Antigua Street Footbridge; Moorhouse Avenue Overpass.***

CBD BRIDGES

Most of bridges located in the Christchurch central business district (CBD) are historic structures that marked an important stage in the economic and social development of an effective transport network laid out in 1850 by the Canterbury Association survey of Christchurch. Initially simple structures and originally made of timber, many had been replaced by the 1880s with massive single span concrete bridges. The good performance of the main spans of these bridges was mainly due to their monolithic structure, acting as a stiff strut between river banks affected by lateral spreading. No collapses were registered and bridges maintained their vertical load carrying capacity after the earthquake. Nevertheless, for this same reason the majority of damage was registered at the abutments (flexural cracking) and at the approaches (settlement and spreading). Some of the most severely damaged bridges in the CBD are shown in Figure 4, with details of the damage suffered by each bridge summarized in the following sections.

Colombo Street Bridge

Description

The Colombo Street Bridge (-43.5271, 172.6366), built in the 1930s, crosses the Avon River between Kilmore Street and Oxford Terrace in a north-south alignment. The single span bridge has a nominal clear span of approximately 16.8 m comprising arched riveted steel girders on the outside and constant depth riveted steel “I” girders over the internal portions. The arched edge girders are built into the reinforced concrete pilasters, supported by shallow foundations. They have been constructed in a second stage when the bridge was widened. The girders sit within a recess in the abutment wall. The deck slab extends beyond the ends of the beams to meet the abutment head wall leaving a nominal expansion gap of 20 mm.

Earthquake Damage

After the 2011 Christchurch earthquake there was evidence of severe liquefaction in the bridge vicinity, with lateral spreading of both approaches. Cracks along the river bank leading up to both abutments were observed. Additional less severe lateral spreading developed as a result of the 13 June 2011 aftershocks.

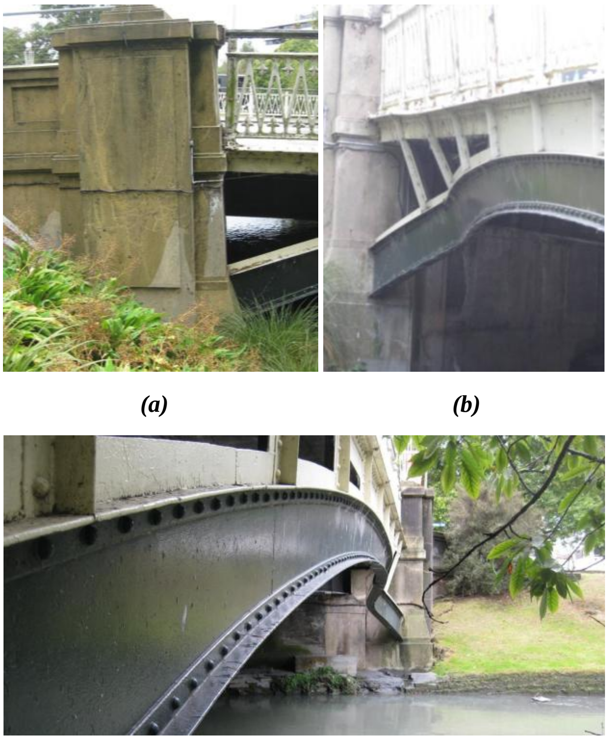
The lateral displacement of the banks behind the abutment contributed to the large cracks observed in the abutments and their back-rotation. Abutment wall rotation was observed on both sides of the bridge, but was more pronounced at the north abutment (Figure 5a). It appears that the abutment wall is restrained at the top by the bridge superstructure and may have rotated outwards at the bottom due to earthquake earth pressures or lateral displacement towards the river. The north abutment pilasters also rotated outwards as a result of the lateral spreading pressures on the wingwalls, with vertical cracks in the abutment wall.

Significant damage identified following the Christchurch earthquake involved severe buckling of the arched edge girders both on the upstream and downstream sides of the bridge (Figure 5b-c). Moreover, the cast iron handrail attached to the top of the arched edge girders became unstable. Connection of the pilasters to the rest of the abutment may be questioned, but irrespectively, even if the pilasters were well connected to the rest of the abutments, abutment rotations produced a larger horizontal displacement at their base than at their top, resulting in buckling of the arches. Because of additional movements and damage resulting from the June 2011 earthquakes, the bridge was still closed at the time of writing (September 2011).

Bridge of Remembrance

Description

The Bridge of Remembrance (-43.5332, 172.6334), constructed in 1924, is a single span, stone faced reinforced concrete arch, of variable thickness and skewed in plan. The Bridge of Remembrance crosses the Avon River between Durham St and Oxford Terrace on Cashel Street. Horizontal earth thrusts are resisted by massive arch thickenings, which extend over the full width of the bridge. The bridge rests upon shallow foundations.



(a) *(b)*

(c)

Figure 5: ***Colombo Street Bridge (a) Rotation of the abutment [Courtesy of Opus]; (b) Buckling of steel arch [Photo by M. Yashinsky]; (c) Close up view of the steel arch [Courtesy of Opus].***

(a) *(b)*

Figure 6: ***Bridge of Remembrance (a) Damage to steps on western approach [Courtesy of Opus]; (b) Damage to north-eastern wingwall [Courtesy of Opus].***

Earthquake Damage

This bridge suffered moderate superficial damage as a result of the 2011 Christchurch earthquake. However, the Triumphal Arch over the bridge sustained more significant structural damage. In terms of the bridge, reported damage included paving damage at the approaches (Figure 6a), large vertical cracks to all wingwalls (Figure 6b) and widespread cracks near the base of arch, along the full length at base of both parapets and through the buttress of the arch above the east abutment. The width of some of these cracks increased after the 13 June 2011 aftershocks.

The majority of the damage has resulted from settlement of the approach fill, lateral spreading soil pressures resulting in deflection and rotation of the wingwalls and possibly the abutments. Minor rotation of all four wingwalls caused cracking of the stone facades. The concrete sections behind the cracked stonework are also likely to have been cracked. This is not considered a major concern at present, as the wingwalls are independent from the main bridge structure and are founded on concrete pads.

Cracking of the parapets may indicate some deflection and hogging of the main arch of the Bridge of Remembrance. Hogging damage to the crown of the arch is unlikely to have a significant effect on the live load capacity of the bridge, given that it only carries pedestrian loads. Hogging of the bridge may result in a slight reduction in the residual seismic capacity of the structure. However, collapse of this robust structure under earthquake induce soil loading/lateral spreading is highly unlikely.

Antigua Street Footbridge

Description

The Antigua Street Footbridge (-43.5340, 172.6277) comprises riveted steel arched trusses on the outsides of the bridge, connected by cross-bracings and rolled steel channel transoms, which support longitudinal timber stringers, and transverse timber decking. The girders are built into solid mass concrete abutment walls, and are supported by shallow foundations. The single span bridge has not been designed to accommodate seismic loading.

Earthquake Damage

Signs of liquefaction were visible in the vicinity of the bridge, with ejected sand and lateral spreading evident approximately 100 m south of the footbridge on Antigua Street. Cracks along the river bank leading up to the southern abutment were observed. The movement of the soil led to the shearing off of the abutments where the top chord of the truss connects. Significant damage was evident at the approaches where asphalt has pushed timber decking units out of place. Moreover, the bottom chord of truss has buckled, while there was no buckling in the top chord possibly due to restraint action of timber deck.

The bridge was closed at the time of writing (September 2011), and additional damage was observed after the 13 June aftershocks. The hog in the longitudinal profile of the bridge was approximately 450 mm at the centre of the span. Both abutments have back-rotated between 1-1.5°, (Figure 7a), and displaced towards the river, with the truss arch compressing. The vertical and diagonal truss members adjacent to the north-eastern abutment have disconnected due to rotation of a low level concrete retaining wall, causing the riveted connection to fail.

The majority of the observed damage resulted from localized liquefaction and/lateral spreading pressures. The lack of piles and lack of reinforcement in the abutments can be considered a critical structural weakness for this bridge. These have resulted in shear failure of the concrete abutments and wingwalls, with the top of the abutment wall being restrained by the bridge superstructure. While this has reduced the capacity of the abutments, they are likely to have some residual shear capacity due to the aggregate interlock of the concrete at the cracked sections.

(a)

(b)

Figure 7: ***Antigua Street Footbridge (a) View underneath showing hogging [Courtesy of Opus]; (b) Damaged north-eastern abutment [Courtesy of Opus].***

(a) *(b)*

Figure 8: ***Antigua Street Footbridge (a) Typical failure of end support of timber footway beams [Courtesy of Opus]; (b) Typical failure of cross bracing member adjacent to corroded joint [Courtesy of Opus]***

The abutment displacement and rotation caused the vertical deflection and hogging of the steel arch truss (Figure 7a). This hogging meant some cross bracing members failed adjacent to more rigid, corroded joints (Figure 8b). The shear failure of the concrete wingwalls has caused the supports to the timber footbridge beams to fail (Figure 8a). These supports had very little anchorage into the concrete wingwalls, and would not have required much movement to cause them to fail. The failure of these supports is not a major concern, as they only support a very small section of the footway, which is currently supported by the rigid handrail.

Moorhouse Avenue Overpass

Description

Moorhouse Avenue Overbridge (-43.5399, 172.6367) is an eleven span reinforced concrete structure providing grade separation between Moorhouse Avenue and Colombo Street. Built in 1964, the reinforced concrete T-beam superstructure is supported by two column bents. The bridge is founded on 406 mm diameter octagonal reinforced concrete piles. Moorhouse Avenue itself is one of the four avenues that encase the CBD of Christchurch City, allowing traffic to flow around the CBD. The structure was constructed in three separate sections, linked with expansion joints (Figure 9).

Earthquake Damage

This bridge was designed prior to current design philosophies, which has resulted in regions where the seismic performance of the structure is deficient through the presence of brittle failure mechanisms. Significant pier damage developed during the 2011 Christchurch earthquake due to transverse ground shaking. The overall performance of the structure was unsatisfactory, with significant shear cracking and buckling of the piers. This damage affected the vertical load carrying capacity of the structure along with the lateral capacity. As a result of this damage, the bridge was closed for over five weeks following the February 2011 earthquake while strengthening works were undertaken.

The bridge sustained damage to one column near the north-east approach where a deck expansion joint was located. The insertion of steel rod linkages in the deck at the expansion joint at only one side of the bridge induced irregularity in the structures transverse response. In fact, with the west and central part of the bridge linked, the bridge pier at the eastern expansion joint suffered extensive displacement demand. The slenderness of the pier affected the vertical load carrying capacity of the structure along with its lateral capacity. The columns also had widely spaced transverse reinforcement, making the structure susceptible to shear failure, a brittle failure mechanism (Figure 10a). Observations after the 2011 Christchurch earthquake indicated that the damaged columns had started to buckle (Figure 10b) putting the central span at risk of collapse. In this instance, damage was induced by extensive ground shaking; large transverse horizontal accelerations may have caused a flexural-bucking failure mechanism in the columns.

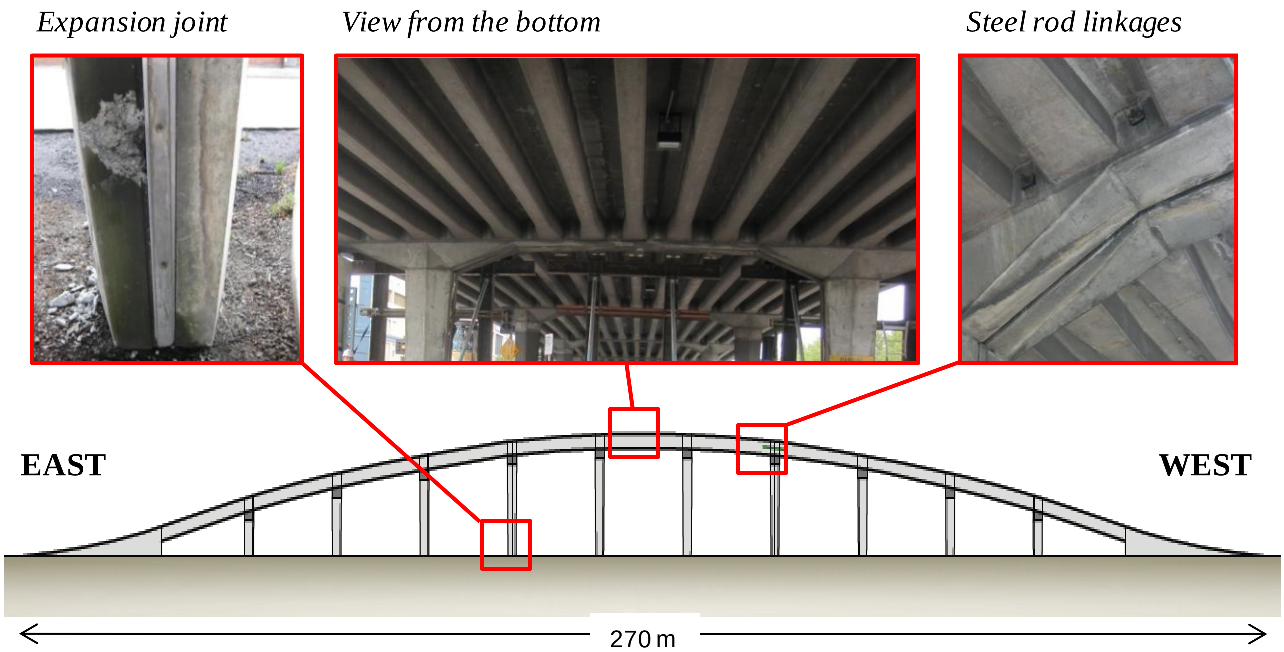


Figure 9: ***Moorhouse Avenue Overpass. Sketch of the bridge elevation with locations of the expansion joints and steel rod linkages (Taken from [5]).***

Due to the higher displacement demand in the west-central part of the bridge, the deck pounded against the south-west abutment of the bridge causing extensive spalling and bar bucking. Ground shaking in the longitudinal direction caused the deck to pull away from the abutment, developing vertical cracks and yielding the reinforcing steel. When put back into compression, the concrete kerb spalled and the reinforcing bars buckled. This has occurred as the abutment is also taking some of the longitudinal lateral load of the centre section due to the placement of steel ties.

Initially temporary strengthening works were erected around the failed pier. These consisted of two columns built-up from square concrete blocks with timber blocking between to prop the bridge while further work was undertaken. A multi-span structural steel frame was designed and constructed soon after the earthquake spanning between the two failed columns. This lead to the bearing of the tapered deck beams on the steel portal to be on a smaller cross-section and was meant only as a temporary solution to prevent collapse of the damaged section. This solution did not provide any additional lateral stability and so the over bridge was not reopened to vehicle traffic until the final temporary solution was implemented.

To improve the transverse capacity of the damaged structure Opus (consulting engineers) elected to construct dual cross-bracing units at each of the piers. These straddled between the piers and used doubled up steel sections to act as a diagonal bracing system as shown in Figure 11.

(a) *(b)*

Figure 10: ***Moorhouse Avenue Overpass. (a) Shear failure mechanism of the pier at the expansion joint [Photo by A. Palermo]; (b) Concrete spalling and bar buckling at south-west side abutment [Photo by A. Kivell] (Taken from [5]).***

Figure 11: ***Moorhouse Avenue Overpass. Temporary propping [Photo by G. Whitla].***

EAST AND SOUTHERN SUBURBS BRIDGES

Damage to bridges outside of the CBD focussed on structures spanning the Avon River, which flows from the CBD through the eastern suburbs of Christchurch. Soil conditions and shaking in this region resulted in large settlements and lateral spreading due to liquefaction. Liquefaction was not observed to the same extent along the other major waterway passing through southern Christchurch, the Heathcote River, contributing to less damage observed to bridges along this river.

A vast majority of road bridges in the heavily affected areas were constructed prior to the 1970s when structural robustness was a key consideration in bridge design. As a result, damage to road bridges was concentrated to the substructure and approaches, with the superstructure, sustaining mainly pounding damage and minor cracking.

The restraint provided by the superstructure prevented the horizontal displacement of the top of the abutments, resulting in large back-rotations of bridge abutments due to lateral spreading loads. Piers were also susceptible to rotation due to liquefaction and lateral spreading of the underlying soil layers. Translational and rotational movement of abutments placed large curvature demands on many piles supporting abutments, resulting in cracking and spalling, with piles tops undergoing large plastic rotations. It is expected that this will also have resulted in pile hinging below ground level.

While structural damage did not restrict vehicle use following the 2011 Christchurch earthquake, many bridges were out of service for a number of hours due to settlement and lateral spreading approach damage. Temporary repairs were quickly in place in most cases, but interruption to emergency services at such a critical time raises concerns. Similarly, services carried on bridges (water, electricity) were often incapacitated with services not being returned for several weeks.

The following sections summarise a selection of Christchurch City Council road bridges (Figure 12) in the east and southern suburbs that sustained the most significant damage.

Ferrymead Bridge

Description

The Ferrymead Bridge (-43.5584, 172.7086) is located at the mouth of the Heathcote River, on the edge of the Avon Heathcote Estuary. This bridge is the primary link to Sumner, an outer suburb of Christchurch, and carries thirty thousand vehicles per day. Construction of the bridge was originally planned in 1862, highlighting the strategic importance of the connection between Christchurch and the suburb of Sumner. The existing three-span prestressed concrete bridge was constructed in 1967 replacing the previous bascule bridge. The west abutment and bent are supported by floating pile foundations, while the eastern bent and abutment have foundations founded on bedrock. This structure was undergoing a major upgrade (deck widening) when the February 2011 earthquake occurred.

Earthquake Damage

The Ferrymead Bridge was the only bridge along the Heathcote River to suffer significant damage. Lateral spreading caused significant damage to the new abutment components and temporary construction platforms. The existing structure also sustained damage due to lateral spreading, with permanent rotation and cracking of a number of the piers situated in the estuary (Figure 13a).

The bridge was temporarily repaired with prestressed rods connecting the bottom of the piers to the abutments (Figure 13b). This limited further movements of piers inwards

Figure 12: Selection of damaged road bridges. Clockwise from top left: Ferrymead Bridge; Bridge Street Bridge; Gayhurst Road Bridge; Avondale Road Bridge; Pages Road Bridge; Fitzgerald Avenue Bridge.

towards the river/estuary which could compromise the overall stability of the bridge. Due to the earthquake damage, heavy vehicle restrictions had been placed on the bridge following the 2011 Christchurch earthquake.

Bridge Street Bridge

Description

The Bridge Street Bridge (-43.5252, 172.7241) crosses the Avon River near the Avon-Heathcote Estuary, servicing the suburbs of South New Brighton and South Beach and providing their primary link to the centre of Christchurch. The three span bridge was constructed in 1980 using an in-situ concrete deck supported by precast, post-tensioned I-beams. The superstructure is supported on two octagonal reinforced concrete piers, with ‘hammerhead’ pier caps. Both the piers and the abutments are supported on raked reinforced concrete octagonal piles. Elastomeric bearing pads have been used to isolate the superstructure in both the longitudinal and transverse directions. Steel shear keys on the pier caps link the superstructure rigidly to the piers.

Earthquake Damage

The primary cause of damage was from lateral spreading of the reclaimed embankment soil on which the abutments are located. Situated in an estuary, the underlying soils were heavily prone to liquefaction and after the onset of shaking, loss of soil stability caused failure and spreading throughout the approach to the bridge (Figure 14a). Flow of soil against the wingwall, abutment and through the piles resulted in abutment back-rotation due to the restraint provided at the top of the abutments by the superstructure. Transverse translation of the skewed abutments relative to the bridge deck was a result of lateral spreading perpendicular to the bridge axis, and some pounding between the deck and abutments (Figure 14b).

Flexural hinging, cracking and spalling was observed in the abutment piles at their connection to the pile cap caused by both the rotation and the translation of the abutments (Figure 14a). Slumping of soil around the abutments due to lateral spreading exposed the tops of the abutment piles. Hinging at the top of the piles was accompanied by flexural cracking and spalling in the exposed sections of the piles under both abutments. The crack patterns observed on the piles indicate that the piles were subjected to bi-directional bending during the earthquake due to the rotation of the abutments and the transverse translation (Figure 14c). Crack widths on the west piles were greater than on the east piles suggesting larger deformations. Uncertainty remains with regards to potential plastic hinging below the ground surface.

(a) *(b)*

(c)

Figure 13: Ferrymead Bridge (a) Tilted piers [Courtesy of Opus]; (b) Close-up view of flexural cracks at the top of the piers [Courtesy of Opus]; (c) Securing works of Ferrymead Bridge [Courtesy of Opus].

(a) *(b)*

(c) *(d)*

Figure 14: ***Bridge Street Bridge (a) Back-rotated abutments [Photo by M. Bruneau]; (b) Pounding of the deck on the abutment [Photo by A. Kivell]; (c) Plastic hinging in abutment piles of Bridge Street Bridge [Photo by M. Bruneau]; (d) West approach settlement and temporary repair [Photo by S. White].***

Major cracking and settlement of the approach pavement and embankments impeded vehicle access to the bridge (Figure 14d). Non-structural components such as guard rails and handrails founded in slumping soil were heavily damaged. The amount of approach settlement at the west end of the bridge was significantly more than observed at the east end, with large amounts of aggregate required to build up the approach road level to that of the bridge before it could return to service. The deck was largely undamaged with damage limited to spalling at the ends due to pounding against the abutments.

Much of the damage described above was initiated in the 2010 Darfield earthquake, with damage levels increasing as a result of the 2011 Christchurch earthquake.

Gayhurst Road Bridge

Description

The Gayhurst Road Bridge (-43.5216, 172.6728) crosses the Avon River, connecting Gloucester Avenue in Avonside to Gayhurst Road in Dallington. Constructed in 1954, the continuous three span deck is supported on wall piers and concrete abutments with wingwalls. Both piers and abutments are supported by reinforced concrete piles. Prior to the Darfield earthquake, the bridge deck and approaches were approximately level, constructed at the natural level of the river banks. The main concern for the site was liquefaction, with previous investigations indicating significant liquefaction in a 100-150 year return period seismic event.

Earthquake Damage

The most significant damage was caused by lateral spreading, resulting in movement and loss of restraint at the abutments, slumping of banks and damage to adjacent roads. The combined effect of the 2010 Darfield, 2011 Christchurch and 13 June 2011 earthquakes was over one metre of settlement of the northern approach to the bridge. There was much less movement to the south, with only a few centimetres of settlement identified.

Lateral spreading resulted in rotation and displacement of the northern abutment backwall and wingwalls (Figure 15b). Wingwalls displaced towards the river and rotated in plan with their outer, more poorly restrained, edges moving towards the river causing the development of flexural cracks in both abutment-to-wingwalls and abutment-to-deck connections. The top of the wingwalls on both sides of the northern abutment displaced 900 mm towards the river relative to the edge of the bridge deck, with lateral movement of 100-150 mm at their base. This rotation/translation of the structural element resulted in high flexural stresses in the abutment and eventually cracking, as shown in Figure 15b. The differential settlement of the soil beneath the abutment and the ground shaking may also have contributed to the formation of cracks. A worsening of the cracking was evident after the 13 June aftershocks, resulting in a total displacement of the top of the wingwalls of 1,300 mm towards the river relative to the backwall. The rotation and translation of the wingwalls meant they completely detached and moved in front of the backwall (Figure 15c). Conversely, there was little indication of damage to the southern abutment, with no significant displacement or rotation accompanying minor cracking.

Lateral spreading exerted a lateral force on the pier base, causing a large moment at the stiff pier-deck interface, inducing cracking of the pier. A single horizontal crack along one face of a pier, approximately one metre from the deck soffit, developed during the 2010 Darfield earthquake and increased after 2011 Christchurch earthquake (Figure 15a). Liquefaction would have reduced the lateral stiffness of the pier foundation system, allowing rotation of the bottom of the pier towards the centre of the river.

(a)

(b) *(c)*

Figure 15: ***Gayhurst Road Bridge (a) Damaged pier with flexural cracking [Photo by L. Hogan]; (b) Flexural cracks at the north-western wingwall [Photo by L. Wotherspoon]; (c) Displacement of north-western wingwall [Photo by L. Wotherspoon].***

(a)

(b) *(c)*

Figure 16: Fitzgerald Avenue Bridge (a) Damage at the approach of the eastern bridge [Photo by A. Kivell]; (b) Back-rotated abutments [Courtesy of Opus]; (c) Flexural cracks on the piles [Photo by L. Hogan].

Bridge inspections indicate that the north abutment and piles are vulnerable to a further rotation, settlement and movement of the wingwalls during a future earthquake. Analysis of the structures has revealed the most likely collapse mechanism is the loss of superstructure support though further rotation and damage of the north abutment. Flexural and shear cracks that have propagated in the rotated bridge pier and abutments could trigger future instability, as they have already sustained damage which compromised their nominal strength.

Fitzgerald Avenue Bridges

Description

The Fitzgerald Avenue Bridges (-43.5262, 172.6506) are located on one of four primary arterial corridors that encase the Christchurch central business district, and are classified as National Strategic bridges in terms of roading and services network importance. The twin bridges constructed in 1964 are located on Fitzgerald Avenue, spanning the Avon River. Each has a two span reinforced concrete superstructure, supported by wall piers and pile foundations. A recent retrofit of the bridges tied the piers and abutments to the deck using steel brackets. As well as providing a critical lifeline to the city for emergency access, the bridges also carries a number of service lines which are situated within its deck, or run beneath it.

Earthquake Damage

Following the 2011 Christchurch earthquake the Fitzgerald Avenue Bridges were closed to traffic for several weeks. Closure was necessary because of severe spreading of the road north of the bridges which developed large fissures (Figure 16a), and Christchurch City Council’s decision to place an emergency cordon around the ‘four avenues’ of the Christchurch CBD, of which Fitzgerald Avenue is one. Large amounts of settlement occurred at the bridge approaches primarily on the northern bank; this was accompanied by two directional lateral spreading. The combination of these factors resulted in the back-rotation of the northern bridge abutments (Figure 16b). Moderate pounding damage was observed at the ends of the simply supported deck beams, exposing reinforcing steel in some regions. Services housed within the bridge were damaged due to lateral spreading.

Rotation of abutments placed high demands on the northern abutment piles. Figure 16c shows the damage to the eastern-most pile of the eastern bridge. Failure occurred in tension with a 30 mm crack opening up across the entire section. In some cases, heavy cracking and spalling due to hinging of the piles at their connection with the pile cap occurred resulting in exposure of prestressing tendons. Cracks ranging from 0.1 mm to 1.5 mm were present on the river side of all the piles. Below ground pile performance was unable to be determined but it would be expected that differential movements between liquefied and non-liquefied layers caused damage in addition to that at the pile to pile cap connection.

Pages Road Bridge

Description

The Pages Road Bridge (-43.5092, 172.7214), constructed in 1931 crosses the Avon River, connecting the city centre to New Brighton, an eastern suburb of Christchurch. The bridge is a cast-in situ monolithic structure with its superstructure supported on two concrete wall piers located in the river. Both piers and abutments are supported by concrete piles, each with fourteen 7.3 m long octagonal piles (350 mm diameter). The pier caps and abutments are both skewed 6° with respect to the bridge axis. Eight principal beams follow an arched profile in the longitudinal direction and are assisted by one transverse beam for each end span, while there are two for the central span. Reinforced concrete wingwalls at each end of the bridge have different design characteristics. At the western abutment they are straight, with tie beams connecting the north and south wingwall for lateral support. At the eastern abutment, the wingwalls are curved, with tie beams connected to anchor blocks to provide support. Expansion joints are located between the wingwalls and the abutment.

Earthquake Damage

Following the 2011 Christchurch earthquake, damage to the area surrounding the bridge as a result of liquefaction intensified from that observed following the 2010 Darfield earthquake. Moderate lateral spreading was evident to the north-west and south-east of the bridge, with flooding in the

(a) *(b)*

Figure 17: Pages Road Bridge.(a) Lateral spreading to the south of the eastern abutment following the June 13 aftershocks; (b) Rotation and cracking of the south-east wingwall (top) and cracking of rock facing and lateral spread crack at north-west corner following June 13 aftershocks (bottom) [Photos by L. Wotherspoon].

latter region. Cracking and settlement of the approaches developed at both abutments as a result of lateral spreading, with movements perpendicular to the river on the west approach, and movements towards the south-west on the east approach. There was a large volume of ejecta and flooding around the western abutment. The region to the north-east of the bridge suffered less liquefaction induced damage compared to the other areas. Following the 13 June aftershocks, there was further minor settlement of the approaches and additional lateral spreading. A large lateral spread immediately to the south of the eastern abutment is shown in Figure 17a.

Because of its robust design, the overall performance of the bridge was good. The bridge deck and beam elements experienced little damage. Throughout the length of the bridge, no significant pavement cracking was observed, as well as any flexural cracking of the beam elements. However, moderate damage was recorded to the services (water and power cables) running along the bridge. A water pipe at the west abutment was broken and cables underneath the deck were exposed.

Following the 2011 Christchurch earthquake all wingwalls hinged and rotated at the interface with the abutment backwall, but there was no visible rotation of the abutment structure. Rock facing supporting the approaches near the wingwalls also moved and cracked, as indicated in Figure 17b. Damage to the wingwalls was increased following the 13 June 2011 aftershocks, with further rotation. Additional cracking and collapse of a section of the rock facing to the south-east of the bridge occurred as a result of lateral spreading.

STATE HIGHWAY BRIDGES

Thirty State Highway (SH) bridges were located within 20 km of the 2011 Christchurch earthquake and the 13 June 2011 aftershocks. Eighteen of these 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 in three of the sets having similar but not identical details. The fourth pair of twin bridges (Styx Overbridges) were constructed at different times and differed significantly in structural form and detail. Most of these bridges had also been inspected after the 2010 Darfield Earthquake. (A further nine bridges more distant from the 2011 epicentres were inspected following the 2010 Darfield Earthquake). Inspection and analysis details of the three bridges shown in Figure 18 are summarised here.

Approximate static analyses were used to assess the strength and performance of the critical components of the inspected SH bridges. 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.

Probable material strengths were used to assess the flexural and shear strengths of the critical components. These strengths were based on the best available information on the strength of the materials used at the time of construction. Response accelerations in each of the bridge principal directions were estimated using calculated periods of vibration and the average of the spectral accelerations for 5% damping (usual design assumption) calculated from the time-history acceleration records from the two nearest strong motion stations to each bridge.

SH 74 ANZAC Drive Bridge

Description

The Anzac Drive bridge (-43.5009, 172.7011), constructed in 2000, carries four lanes of SH 74 across the Avon River about 6 km north-east of central Christchurch. It has three simply supported 18 m long spans constructed of prestressed concrete hollow core deck units. The piers are transverse portal frames with four reinforced concrete rectangular columns founded on 1.5 m diameter bored/driven reinforced concrete piles. The abutments are conventional sill beams supported on 16 or 17 vertical 310UC piles. They have 3 m long settlement slabs tied to them with closely spaced reinforcement. The foundation soils are loose to medium sands or silty sands down to very dense sands at about 16 m depth.

Site Ground Motions and Bridge Response

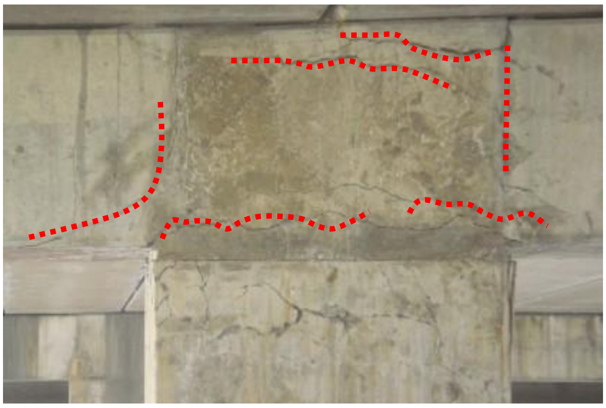
The bridge was located about 9 km north of the epicentre of the 2011 Christchurch earthquake. Passive resistance from the 2.1 m high abutment backwalls, settlement slabs and closely spaced abutment piles stiffened the bridge in the longitudinal direction producing a short period of vibration estimated to be in the range of 0.25 to 0.3 seconds. The portal frame piers founded on the large diameter piles are very stiff in the transverse direction resulting in an estimated first transverse mode period of between 0.13 to 0.2 seconds. Assuming simple single degree of freedom (SDOF) response with 5% damping the response accelerations in the Christchurch earthquake would have been about 0.7g and 0.5g in the longitudinal and transverse directions respectively.

Earthquake Damage

Significant structural damage was observed following the strong shaking in the 2011 Christchurch earthquake. Lateral spreading of the approaches caused the abutment piles and beam seatings to rotate about 8^{o} (Figure 19a). Cracking and spalling was observed at the beam-column joints in the piers and some of the pier columns rotated up to 2º (Figure 19b). Damage was more pronounced at the outer of the three columns in each pier. There was also significant lateral spreading and settlement of the approach pavements and embankments. Concrete walkway structures under each end of the bridge moved towards the river about 300 mm and 500 mm at the north and south ends respectively. No significant additional structural damage was reported following the 13 June 2011 aftershocks.

Figure 18: ***Selection of damaged State Highway bridges after February 22, 2011. From left to right: Anzac Drive Bridge [Photo by E. Camnasio]; Horotane Overbridge [Courtesy of Opus]; Port Hills Overbridge [Courtesy of Opus].***

(a) *(b)*



(c)

Figure 19: ***Anzac Drive Bridge (a) Back-rotation of southern abutment [Photo by A. Palermo]; (b) Flexural cracks at the central pier [Photo by N. Chouw]; (c) Plastic hinging at pier cap [Photo by N. Chouw].***

Performance Assessment

The static analysis for transverse loading indicated that the tops of the centre columns, which were the most critically loaded sections in the portal frame piers, would reach their ultimate flexural strength capacities at a response acceleration of about 0.55g. The observed cracking and spalling in the piers under an estimated response acceleration of 0.5g was reasonably consistent with this prediction.

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 and the piers, the longitudinal static 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 0.55g. 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 1.1g 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 pier columns are detailed with confinement reinforcing and would perform satisfactorily if subjected to much larger longitudinal displacements than yield level.

Structural damage observations following the earthquakes were reasonably consistent with analysis predictions. Predictions of the lateral spreading displacements have not been carried out but the site is of particular interest because of the difference in spreading-induced damage that occurred in the 2010 Darfield and 2011 Christchurch earthquakes. In the Christchurch earthquake lateral spreading forces placed large demands on the abutment steel piles and probably led to plastic hinging below ground level. After the 13 June 2011 aftershocks settlement and spreading had exposed 1300 mm of the top of the piles at the south abutment.

SH 74 Horotane Valley Overpass

Description

The Horotane Valley Overpass (-43.5725, 172.6947), constructed in 1963, consists of twin bridges each carrying two lanes of SH 74 across Horotane Valley Road. The Overpass is located about 2 km west of the Christchurch portal to the Lyttelton Road tunnel. Both bridges are similar except that the No. 2 Bridge carrying the west bound lanes widens towards its west end to provide additional width for an off-ramp. Each bridge has three simply supported spans with prestressed concrete beams supporting a reinforced concrete deck. The spans are 13.9 m and 12.5 m for the end and central spans respectively. The bridge abutments and the single stem rectangular piers are founded on spread footings. The spans are well linked to the abutments and piers by both holding down dowels and linkage rods.

The Overpass bridges have recently been strengthened by fitting fabricated steel shear keys at the abutments, primarily to resist transverse loads. Each of the nine brackets at each abutment (single abutment structure for both bridges) is fitted with a 30 mm diameter bolt into the bottom of the beams. This provides additional longitudinal restraint in addition to that provided by the original linkage. Additional linkage rods were added between the outer beams at each pier. These were designed to improve the deck diaphragm action under transverse loading and avoid unseating.

Site Ground Motions and Bridge Response

The Overpass was located about 1.3 km from the epicentre of the 2011 Christchurch earthquake. Records from the closest strong motion station (HVSC at 1.6 km from the bridges) are likely to have been significantly influenced by topographic effects and the estimated ground motions were based on the second closest strong motion station (LPCC). The mean PGA recorded at this station was 0.83g in the Christchurch earthquake.

The large spread footings at the abutments result in the bridges being relatively stiff in the longitudinal direction with periods expected to be in the 0.3 to 0.5 second 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 second assuming no rocking of foundation pads. The best estimates of response accelerations in the longitudinal and transverse directions for 5% damping were about 1.0g and 0.5g respectively. Because of the wide variation in the short period ordinates of the response spectra computed from the two nearest SMA's (HVSC and LPCC) the estimate of response accelerations contains a large degree of uncertainty and may have been greater than these best estimate values.

Earthquake Damage

Following the 2011 Christchurch earthquake, fine horizontal cracking was observed on the lower half of all four piers. Crack widths were up to 0.2 mm and the cracks were located between about 500 to 2,600 mm above ground level. The cracks were more pronounced on the column faces nearest to the roadway under the bridge. Measured distances between the bases of the piers indicated that these spacings had shortened by 200 mm on the No 1 Bridge and 260 mm on the No 2 Bridge.

The east end of the No. 2 Bridge displaced horizontally about 100 mm in a southwards direction. This resulted in severe vertical cracking at the junction between the abutment seating and the abutment wall between the two bridges. There was minor spalling at the ends of beams where they were seated on the abutments.

All four abutments appeared to have moved forward by up to 20 mm. This movement caused severe shear cracking in the backwall and shearing of two of the bolts on the new linkage brackets (loaded in shear) at the west abutment of the No 1 Bridge. Bolts on the new linkage brackets also sheared on both abutments of the No. 2 Bridge. The west abutments had settled by about 60 mm. This was particularly visible on the south side of the No. 2 Bridge (Figure 20a). Surface sliding of soil was evident under the west abutments and wide cracks and separation gaps between the soil and abutments were evident at the east abutments indicating significant down-slope movements (Figure 20b).

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

Minor structural damage to the bridges had been observed following the 2010 Darfield Earthquake. The most significant damage was the loosening of all the abutment linkage rods on both bridges with the slackness under the washers varying between 5 and 10 mm.

(a) ***(b)***

Figure 20: Horotane Overbridge (a) Sheared bolt at the abutment retrofit [Photo by J. Allen]; (b) Slope failure at the abutments [Photo by J. Wood].

Performance Assessment

The slackness of the abutment linkage rods 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 widening 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 appeared to slide 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 rods adding to the slackness in the linkage rod 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 rods (11 at each abutment structure) yielded as they have sufficient strength to resist the total longitudinal inertia force from a response acceleration of about 0.3g on the superstructure.

An analysis of the sliding stability of the abutment structures, loaded by the bridge, inertia force from the abutment mass and backfill static and earthquake pressures indicated that they would slide forward at a ground acceleration of about 0.25g. 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 0.12g. The shallow slope failures observed following the 2011 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 move towards the centre of the bridge resulting in 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 some of the cracking observed in the walls.

The simplified static analysis for transverse loading indicated that the bases of the pier stems would reach their ultimate flexural strengths at a response acceleration of about 0.25g. Although the bases of the piers were not visible there was no evidence of soil gapping or concrete cracking at ground level and it therefore seems unlikely that piers were loaded in the transverse direction above the yield level of the flexural reinforcement. 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.

SH 74 Port Hills Overpass

Description

The No. 1 Overpass Bridge (-43.5711, 172.6934), constructed in 1963, carries the south bound lane of SH 74 and an on-ramp lane across Port Hills Road. The No. 2 Bridge, constructed at the same time, carries the single north bound lane of SH 74 across Port Hills Road. The Overpass is located about 7 km south-east of central Christchurch and about 2.2 km north-west of the Christchurch Portal entrance to the Lyttelton Road tunnel. Both bridges are of similar construction with six simply supported spans of prestressed concrete log type beams supporting a reinforced concrete topping. The spans vary in length between 9.4 to 12.6 m and the overall length of the wider No. 1 Bridge is 72.4 m. The bridge abutments and the reinforced concrete single stem rectangular piers are founded on spread footings.

The Overpass bridges have recently been strengthened by fixing fabricated steel shear keys to the underside of the beams at both the abutments and piers to resist longitudinal earthquake loads. Linkage rods were 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 was provided by rods extending between the brackets and the soil face of the abutment seating beams. New shear keys fixed to the faces of the abutments and piers provide resistance to transverse loads. 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 linked originally by longitudinal 12 mm diameter bars also 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.

Circular steel shrouds were added at one pier column on each bridge where they passed through a significant depth of soil on the abutment slopes. The shrouds provide a clear annulus to prevent the piers from being stiffened by the soil which would have resulted in them carrying a disproportionate share of the earthquake loads through loss of symmetry.

Site Ground Motions and Bridge Response

The Overpass was located about 1.3 km north of the epicentre of the 2011 Christchurch earthquake. It is about 150 m north-west of the Horotane Valley Overpass and would have experienced similar ground motions (see previous description).

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 second 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. 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. For 5% damping the response accelerations in the Christchurch earthquake would have been about 1.0g and 0.5g in the longitudinal and transverse directions respectively.

Earthquake Damage

In the 2011 Christchurch earthquake flexural cracking developed in the lower halves of all pier stems of both bridges except those adjacent to the abutments. (The lower halves of two of the piers adjacent to the abutments were not visible because of the retrofitted shrouds and soil surrounding them). Soil gapping at ground level occurred at the faces of most of the pier stems with separation cracks up to 15 mm wide. Spalling damage occurred at ground level on the centre pier of the No. 1 Bridge (eastern bridge), and buckling of reinforcing bars (Figure 21). Spalling at a similar location on one other pier was reported following the 13 June 2011 aftershock. The nominal 10 mm gaps between the new shear keys and the abutment face at the south-east abutment of the No. 1 Bridge closed up with no clearance on two of the four keys.

Down slope soil movement on the abutment slopes under the bridge resulted in wide cracks in the soil and at the contact between the soil and abutment face under the south-east abutments. Wide cracks running parallel to the roadway at the top of the approach embankment developed in the soil on the east side at the south-east end of the bridges. There was also minor settlement and displacement of the approach pavement and back-fill at the abutments.

Figure 21: ***Port Hills Overbridge. Buckling of reinforcing steel at the base of the piers [Courtesy of Opus].***

Performance Assessment

The simplified static transverse loading analysis indicated that the pier stems would reach their ultimate flexural strengths at a response acceleration of about 0.3g. This is significantly lower than the estimated response acceleration in the 2011 Christchurch earthquake of 0.5g. The damage in the Christchurch earthquake indicated some inelastic behaviour of the central piers but the overall performance was 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.

The 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 0.5g. 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 0.6g. Only small longitudinal movements occurred in the estimated 1.0g response acceleration in 2011 Christchurch earthquake so overall the longitudinal performance appeared to be rather better than predicted.

Without the new shear keys and linkage rods at the abutments failure of the original hold-down dowels would have occurred at a response acceleration of about 0.25g. The retrofitted shear keys at the abutments probably prevented significant damage as the loading on the original dowels in the Christchurch earthquake would have been significantly greater than their strength capacity.

LIFELINES CARRIED BY BRIDGES

Ground shaking and liquefaction/lateral spreading were damaging not only to the bridge substructures but also to pipeline systems crossing bridges. Many services are located along the longitudinal bridge axis under or within the bridge deck. Lifeline networks were severely damaged along the Avon River due to extensive liquefaction and lateral spreading. Quite consistently, areas where soil-bridge interaction occurred in lateral spreading, pipelines were also damaged. Several pipes were damaged due to a differential settlement between the bridge and the surrounding soil. This indicated that pipe connections were not appropriately designed to accommodate deck-to-pipe, or abutment-to-pipe relative displacements. Figure 22 shows some of the typical damage reported.

An example of important services carried on a bridge is the Dallington Pedestrian Bridge, where lateral spreading created extensive cracking parallel to the bridge and perpendicular to the alignment of the road and buried pipes, potentially leading to damage to sewer pipes in the road. This bridge has two 66 kW power cables placed under the bridge deck, providing electricity to 20,000 inhabitants, making this modest pedestrian bridge a structure with strategic importance. Some moderate to extensive damage was observed at many road bridges where pipes were distorted and/or leaking in the proximity of the connections from deck-to-abutment, and abutment-to-approaches. The main issues arose with stiff pipes, such as sewage and water pipes, as they are fully fixed to the deck and usually run through the abutments. On the other hand, the flexibility of power and/or telephone cables were able to accommodate larger displacement demands.

(a) *(b)*

(c) *(d)*

Figure 22: ***(a) Gayhurst Road Bridge, damage to lifelines [Photo by A. Palermo]; (b) Dallington Pedestrian Footbridge, damage to electrical services to the Dallington area [Photo by M. Le Heux]; (c) Repair to water pipe at Bridge Street Bridge [Photos by A. Campbell]; (d) Repairing works at Bridge Street Bridge [Photo by A. Palermo].***

CONCLUSIONS

With some noteworthy exceptions highlighted in this paper, the overall bridge performance was satisfactory in the 2011 Christchurch earthquakes. Old road bridges in the city were mainly robust integral bridges with stiff superstructures which responded quite well to ground shaking and liquefaction/lateral spreading.

The city precast concrete bridges built after the 1960s performed satisfactorily but residual rotations/displacements were more evident due to the lack of effectiveness of deck to abutment linkages.

State Highway bridges performed well with no bridges closed to traffic because of structural damage. At several bridges moderate traffic disruption resulted from lateral spreading and differential settlement of the approach fills. The seismic strengthening work started in 2000 [1] was effective in reducing the damage to most of the seven bridges that had been strengthened.

Ground shaking and liquefaction/lateral spreading were very damaging at connections of services to bridges. Many pipeline connections or connecting members fractured causing extensive water leakage and/or pollution of the river crossed.

Design guidelines for abutment/pile liquefaction/lateral spreading should be more widely implemented in order to reduce this type of damage.

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