Quantifying the seismic resilience of bridges: a pathway towards a Resilience-Based Design

Quantifying the seismic resilience of bridges: a pathway towards a Resilience-Based Design

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ABSTRACT: The loss of functionality of road networks during the past Canterbury (2010-2011) and Kaikoura (2016) earthquakes has questioned New Zealand's established seismic resilience. The excessive direct and indirect costs due to downtime and non-structural damage highlighted the need to move towards new performance indicators. Resilience holds the key to describe the performance of modern structures by requiring low failure probabilities, reduced consequences from failures, and less recovery time. Combining the concepts of resilience directly with the structural design process can increase the confidence in implementing damage-resistant technologies to reduce the damage of bridges in an earthquake. Furthermore, translating resilience measures with concise and meaningful terms to decision makers will lead to a better understanding of the benefits of mitigation.

Resilience is not being considered in the seismic codes, as traditionally their main objective has been to prevent collapse and ensure life-safety. Performance-based design (PBD) is a supplement to code objectives and intends to demonstrate that pre-identified earthquake performance objectives for the structure are satisfied. Yet, it does not include explicit verification of the expected functionality of the structure after the earthquake. On the other hand, resilience-based design (RBD) appears as a holistic design process, which identifies and mitigates earthquake-induced risks to enable rapid recovery in the aftermath of major earthquakes.

This paper presents an overview of the structural performance of road bridges during the Kaikoura Earthquake and introduces the severely damaged ones as case studies. For each of the analyzed bridges, a seismic resilience curve has been developed based on the observations made during site inspections and data obtained on the functionality of the bridges over the time following the earthquake. A framework to incorporate the resilience concepts and measures, as key design criteria and indicators, into the structural design process is also introduced and exemplified using the case studies. Applying the proposed framework during the design phase will allow the estimation, by defining different recovery strategies, of final recovery times and preliminary recovery costs of the bridge after an earthquake.

1. INTRODUCTION

The Kaikōura earthquake, with a moment of magnitude equivalent to M_w=7.8, occurred two minutes after midnight on the 14th of November 2016 (NZST). The epicenter was about 60km south-west of the town of Kaikōura, New Zealand (NZ), while the hypocenter was at a depth of approximately 15km. The event was a summation of multiple fault ruptures over a large spatial domain, and its complexity has resulted in the reassessment of many assumptions about earthquake processes.

The transportation network of the entire northeast portion of the South Island was badly affected. In this area, there are over 268 State Highway (SH) bridge structures, most of which are made of reinforced concrete, and 636 local road bridge structures (Palermo et al., 2017).

The performance of bridges during major earthquakes is of interest for reviewing the assumptions that are made during the seismic design or assessing the structure and its approaches. Therefore, the overall aim of this paper is to present general observations for the performance and resilience shown by the most severely damaged road bridges in the Kaikōura Earthquake, and to open the discussion on how to improve seismic resilience of road bridges during future events.

The four bridges covered by the present paper are located within 5km radius of one another and within 11km of the township of Waiau, which is the closest town to the earthquake epicenter. Except for the Wandle River Bridge, which is on a shallow soil site, the bridges are situated on sites underlain with deep layers of dense gravels and sand, and it is likely that they were subjected to similar intensity of ground shaking (Wood & McHaffie, 2017). Table 1 shows the main general information of the four bridges analyzed, together with a map of their location in

Figure 1.

Table 1. Bridges assessed in the present report

Bridge Name	Year of design	Length	Number of spans	High-way	Distance of epicenter
		m			km
Lottery River	1984	123	6	Inland Route	11
Lower Mason River	1984	164	8	Inland Route	7
Mason River	1979	197	12	River Road	5
Wandle River	1987	51	3	Inland Route	13

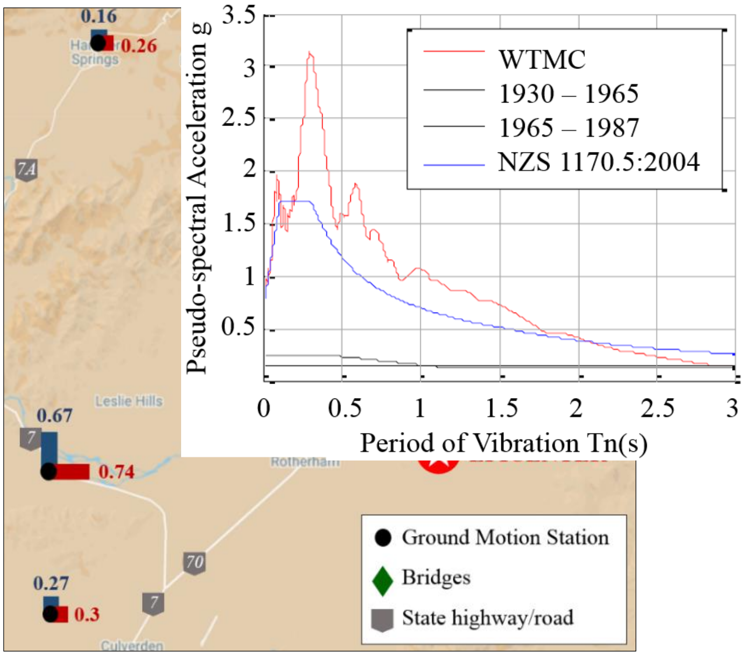
Figure 1. Map of bridges analyzed in the Hurunui District with the maximum measured PGA

1 EARTHQUAKE GROUND MOTIONS

During the Kaikōura earthquake, there was a large spatial variation in the shaking intensity, even within the moderate distances within the same district. When judging the level of damage to bridges, on top of the earthquake intensity, the variation in the design philosophy and the seismic detailing need to be taken into account as they can dictate either desirable or undesirable performance.

Figure 2 shows the geometric mean (of the two horizontal components) of the 5% damped elastic horizontal pseudo-spectral accelerations measured in Te Mara Farm Waiau station (WTMC), located close to Waiau (see

Figure 1). Overlaid are the NZ elastic design spectra used during three design eras. The present day elastic design spectra was taken from NZS1170.5:2004, it was obtained for a Bridge Manual Importance Level 2 (1/1000-year annual probability of exceedance, ULS) structure, soil class C, and having a Hazard factor Z = 0.45, with no near



fault effects. The two oldest design spectra, based on working stress design, were taken from Kaiser et al. (2017), and modified for compatibility with limit state design by Palermo et al (2017).

The bridges studied were designed in the period ranging from 1980 to 1987, hence, using the design spectra from the 1965-1987 era. The corresponding design spectra was greatly exceeded, as it can be seen in Figure 2. Most bridges had elastic periods of vibration within the range of 0.2 s to 1.0s, which is the period of vibration where the ground shaking at WTMC exceeded even the modern design spectra (Figure 2). The bridges had a transverse period of vibration in the range of 0.2s to 0.6s and the pseudo-spectral acceleration experienced by these bridges greatly exceeded 1.5g.

Figure 2. Geometric mean of the pseudo-spectral acceleration measured at WTMC strong motion station (Palermo et al., 2017)

2 STRUCTURAL PERFORMANCE AND DAMAGE

2.1 *Structural configuration of the bridges*

The Lottery River, Lower Mason River, Mason River, and Wandle River bridges (Figure 3) are structurally similar to one another. They are all multi-span precast concrete bridges supported on single column piers with hammer head pier caps. Further structural details are summarized in

Table 2.

Table 2. Main structural details and main damage summary for the four bridges under study (Wood & McHaffie, 2017)

Bridge Name	Superstructure	Pier type	Pier founda-tion	Main damage

Lottery River	Continuous RC deck, joint at middle pier, twin 20m prestressed I beams	Single column	Single cylinder	Severe hinging in columns & buckling of column rebar
Lower Mason River	Continuous RC deck, joint at middle pier, twin 20m prestressed I beams	Single column	Single cylinder	Severe hinging in columns & fracture/buckling of column rebar
Mason River	Five 16m pretensioned double core deck units	Single column	Steel H pile group	Hinging in columns, superstructure joint damage
Wandle River	Five 16m pretensioned double core deck units	Single column	Single cylinder	Severe hinging in columns - Bridge demolished

In the way they resist lateral loading, the Lottery River (Figure 3a) and Lower Mason (Figure 3b) bridges are identical. The transverse interaction has been isolated by the omission of transverse shear keys at the central pier (Palermo et al., 2017). In addition, they use a rudimentary form of seismic isolation under transverse loading such that they can displace an appreciable amount on the supporting elastomeric bearings before contacting the shear keys. As a result, the bridges make use of the low lateral stiffness of the bearings to elongate the period of the structure.

Figure 3. Bridges under study: (a) Lottery River Bridge; (b) Lower Mason River Bridge; (c) Mason River Bridge; (d) Wandle River Bridge (Palermo et al., 2017).

The longitudinal interaction between each half of the bridge has been isolated by a movement joint with knock off detail. The deck beams are tied together longitudinally except over the central pier. The beams rest on wide seats, thus, all piers resist longitudinal lateral loading, except the central pier, which is effectively much more flexible. Until the beams contact the shear keys, the overall transverse response can be thought of as rigid body translation of each half of the superstructure.

Similarly, the Mason River Bridge (Figure 3c) is split in two with a central movement joint. There are transverse shear keys at the abutments and internal longitudinal shear keys and linkage bars over all of the piers except the central pier. At the central pier, provision is made for longitudinal movement with loose linkage bars but steel plate lateral support stubs cast into the piers restrain the transverse movement. Therefore, the overall transverse response can be thought of as the deck having a curved displaced shape where the largest transverse displacement occurs at the central pier (Palermo et al., 2017).

The Wandle River Bridge (Figure 3d) is curved in plan and has a slope in the longitudinal direction. Linkages, shear keys and elastomeric bearings restrain the transverse and longitudinal movement over the piers, but only linkage bars resist both the transverse and longitudinal movement at the abutments. Longitudinal movement towards the backfill at the abutments is resisted by the abutment back-wall which is cast against the beams.

2.2 *Bridge performance and observed damage*

At the Lower Mason Bridge, all piers, except for the central, formed fully developed plastic hinges (PH) at the base. As mentioned, the central pier is located below a movement joint in the deck and has shear keys restraining the superstructure. Extensive cover spalling and a significant number of buckled and fractured longitudinal bars were seen. Pier damage at the Lottery River Bridge was similar but without an evidence of bar fracture. Figure 4 illustrates the damage patterns found on the column rebar of both bridges.

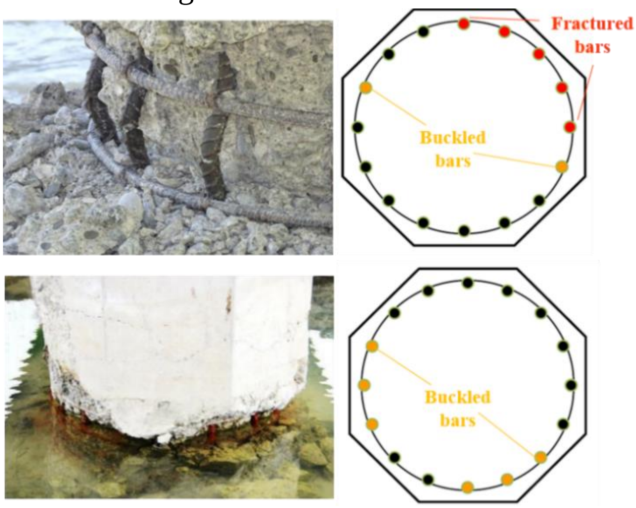
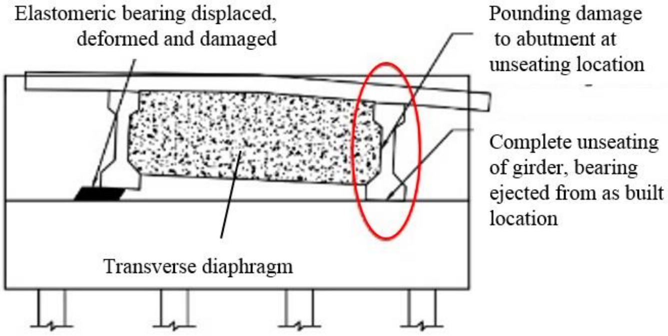


Figure 4. Pier damage sustained by Lower Mason River Bridge (top) and Lottery River Bridge (bottom).

Together with the PH zones, observed damage common to both bridges, Lower Mason and Lottery, included: damage to the knock-off block at the central pier movement joint, failure of the abutment knock-off blocks; residual displacement of most of the elastomeric bearing pads at the piers and abutments; and residual displacements at the central pier and abutment movement joints. Damage to the abutments is illustrated in Figure 5.

Figure 5. Abutment damage schematics for the Lower Mason



River Bridge (modified from: Palermo et al, 2017)

Similar damage observations were made for the Mason River Bridge, where the deck joints had opened or closed, and pier PH with bar buckling and concrete spalling were observed with increasing degree of damage towards the central pier. The pier damage pattern was interesting at this bridge as concrete spalling and rebar exposure occurred mainly at the face of the piers facing south, and tended to be more in the longitudinal loading direction.

The damage observed to the Wandle River Bridge was extensive, and included: separation between the hollow core deck units at the south end of the bridge; translation of deck units in the downstream direction; hinging and rotating of piers towards the upstream side (up to 7° rotation); opening of the deck joints at the abutments, along with significant approach damage (Figure 6).

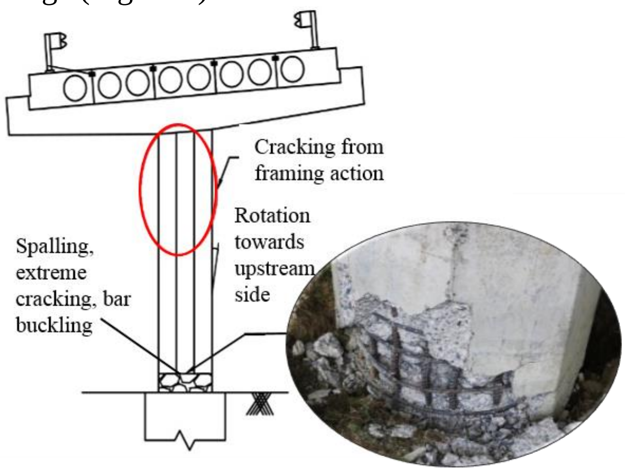
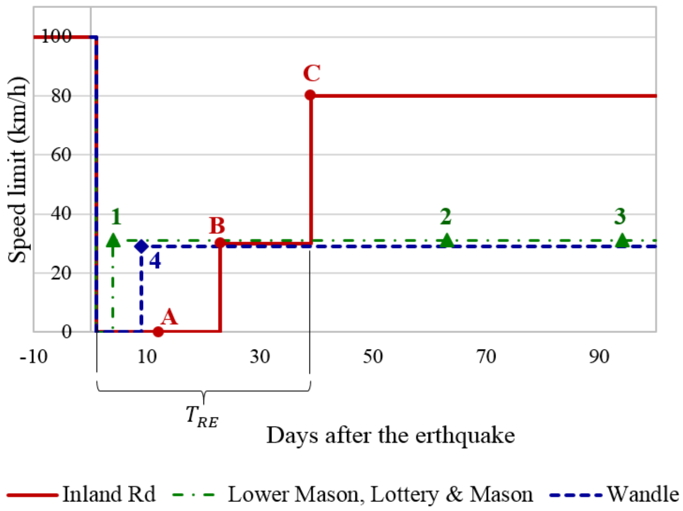


Figure 6. Damage schematics for the Wandle River Bridge (modified from: Palermo et al., 2017)

The displacement capacities for the bridges were estimated by Wood & McHaffie (2017) using a tributary mass type Displacement Based Design (DBD) analysis of a single pier, at the Damage Control Limit State (DCLS). They are compared in Table 3 with the displacement demand for transverse direction loading estimated from spectra computed from the WMTC records.

Table 3. Displacement capacity and WMTC demand

Bridge Name	DCLS Capacity	Demand from WMTC Records	Ratio Capacity/ Demand	Secant Period at WMTC Demand
	mm	mm		s
Lottery River	130	300	0.43	2.1
Lower Mason River	160	290	0.55	2.2
Mason River	180	130	1.38	1.1
Wandle River	180	350	0.51	1.8



Based on the assumption that the WMTC records are a reasonable representation of the bridge site ground motions, the displacement demands on the bridges were considerably greater than the DCLS capacity displacements, explaining the severe damage observed in the PH zones.

It can be concluded that the piers performed well, as they developed PH at the pier bases and went through the expected failure mechanism. However, significant instability was visible in the Wandle Bridge, where most likely all orthogonal motions were interacting with the bridge structure, causing large displacements and separation gaps between some of the deck beam units.

3 RECOVERY PROCESS

After the earthquake, the priority was to restore access to Kaikōura Township and surroundings, which were cut off by earthquake damage, and establish a safe and reliable alternative route. To achieve this, the Inland Route, via Waiau, was identified as the best option since the main route, through SH1, was completely blocked due to landslides.

It took 38 days after the earthquake to reopen the Inland Route. This was mainly due to earthquake induced damage on the road such as significant rock fall and slips that needed to be cleared, in addition to the associated construction activity which also contributed to the hazard. The bridges were able to be reopened, to some level of service, within a few days. However, it took up to three months to complete their repairs. It is important to highlight that repairs to the bridges so far are considered temporary in some extent and permanent restoration of the bridges is to be planned

In general terms, the bridge structures withstood the earthquake sequence well, demonstrating considerably robustness. However, many bridge approaches were damaged due to settlement and horizontal movement, thereby making some bridges impassable. Approach reconstruction through the placement of fill and resealing quickly made the majority of these road bridges usable again.

Figure 7 illustrates the recovery rate of the Inland Route and the bridges under study during the first one hundred days following the earthquake using the maximum speed allowance as a functionality indicator for normal, Class 1, vehicles. The earthquake occurred at 12:02 am on the 14th of November, causing the Inland Route to be immediately closed by the Civil Defense Emergency Management. The route remained closed until it was given access for the NZ Defense Force convoys to get through with emergency supplies to Kaikōura, on November 25th, 12 days after the earthquake (point A in Figure 7).

Figure 7. Functionality based on the maximum speed limit for the Inland Route and the four bridges under study.

The route remained fragile and hazardous, closed to the general public until the 6th of December, day 23 after the earthquake (point B in Figure 7), when controlled emergency and resident access was allowed with a speed limitation of 30km/h, 70% less than the pre-earthquake speed limit of 100km/h. Works undertaken during these days to make the severely damaged road safe enough included clearing the road of more than fifty slips, stabilizing slopes, and assessing the damage to the bridges.

It was only until the 21st (point C in Figure 7) of December when the public were granted unrestricted access to the Inland Road, bringing the first unrestricted road connection back to Kaikōura, for a total recovery time T_{RE} of 38 days after the earthquake.

The Lower Mason River Bridge was opened to traffic on November 17th, day 4 after the earthquake, (point 1 on the graph) with a speed restriction of 30km/h and a sign “one heavy vehicle at a time”. Overweight vehicles were to use the adjacent ford that was installed on the day after the earthquake. Prior to opening the bridge, the following works had been completed: detailed inspection of the bridge, removal and appropriate filling of the knock off device at the central expansion joint, and approach repairs and re-levelling. To reinstate the original seismic capacity, a pile jacketing was issued for all pier columns, which was finished around mid-January (point 2 in Figure 7). After the pile jacketing was completed, the one-heavy-vehicle-at-a-time restriction was removed but the 30km/h restriction remained due to the uneven road surface.

The recovery process for the Lottery Bridge was very similar to the Lower Mason, since they share similar structure type and damage. Pier columns were unable to be inspected on the first days after the earthquake due to the bed and water level, yet, damage was anticipated to be less than at Lower Mason due to the shorter bridge columns. Excavation of pier piles occurred on November 25th confirming that damage was of a lesser extent, and pier jacketing was issued and completed around mid-February (point 3 in Figure 7).

Likewise, the Mason River Bridge shared the same timeline, however no security works have been carried out on this bridge as the earthquake damage was considered not to affect the serviceability of the structure. Investigation is currently underway to expose bridge piers and replace buckled bars.

For the Wandle River Bridge, decision was made on November 15th to install a Bailey bridge upstream with a 33m span. Installation was completed by November 21st and opened to traffic on the 22nd, nine days after the earthquake (point 4 in Figure 7). The Bailey bridge was able to be erected so quickly, within eight days, because New Zealand Transport Agency (NZTA) maintains an emergency stock of Bailey bridges with a contractor also in place to respond in the event of emergencies. The Bailey bridge provided largely unrestricted access across the Wandle River, however, a 30km/h speed restriction was installed due to the alignment and to provide impact protection to the bridge. The Wandle Bridge was demolished in May 2017 and a new bridge design is programmed with a low priority for 2018.

Permanent recovery works on the bridges have been given a low priority. The initial response works intended to restore an acceptable level of service for the Inland Route that would enable access into and out of Kaikōura following the November earthquake. Once this was achieved, the priority of works was switched to recovering SH1. There is also reluctance to undertake further significant work on the Inland Route, as this route acts as a bypass to SH1 to access Kaikōura from the south.

It is important to note that while the route may have been at open, pre-earthquake speed limits of 100km/h, heavy vehicles would still have to comply with lower speed requirements. Additionally, all bridges are single lane, therefore, it is unlikely that even normal vehicles were travelling across the bridges at maximum speed.

4 HOW TO PREDICT STRUCTURAL RESILIENCE?

The NZ South Island transportation network had a resilient response, as it counted with alternative routes connecting locations, allowing to restore access to isolated communities within a relatively reasonable time. On the other hand, access to Kaikōura was closed during the first 23 days for the general public, what makes us wonder, resilience for whom?

Furthermore, in terms of bridge performance, the bridge structures withstood the earthquake sequence well. Bridges demonstrated considerably robustness and hence resilience as, even if damaged, they allowed the transit of vehicles and therefore to reinstate functionality in the route. But, as they are still awaiting for permanent repairs to be undertaken, what is the current damage resistance capacity or probability of failure? Can we expect them to show resilience for future events? What leads us to the question, how can we assess and predict structural resilience?

4.1 *Resilience assessment*

In studies dealing with seismic hazard, resilience can be considered as a performance indicator that quantifies the residual functionality along with the effort in responding to the seismic event (Deco et al, 2013). In this conceptual approach, performance can range between 0 and 100%. In case of an earthquake, usually serviceability of a structure drops to a lower level, being 0% a complete loss of service, followed by a recovery process after the disaster, and then finishes when there is a restoration of the serviceability to its “normal” performance level. This characterization of system performance leads to a broader conceptualization of resilience, where a resilient structure can be understood as one with the ability to present reduced chances of failure, reduced consequences from failures and reduced recovery time (Bruneau et al, 2003).

Within the *Technical Guidance for Engineering Assessments* (2017) in NZ, structural resilience is defined as the ability of the structure to continue to perform in earthquake shaking beyond an Ultimate Limit State (ULS) shaking demand level. Therefore, the bigger the space between the point of onset of nonlinear behavior and the point of brittle behavior of the structure, which would lead to collapse, the more resilient is a structure. Structural resilience can be thought as an inherent characteristic of structures designed to the new codes or even older well designed structures, as their performance will follow the desirable hierarchy of element failures, which was the case of the bridges under study. There can be the case where a structural system has a low resilience and is susceptible to sudden reduction in their performance and functionality as the earthquake shaking increases beyond a particular value.

Measuring resilience has been an exploding field of inquiry in the past decade. Disaster resilience assessment approaches fall into three primary categories: indicators, scorecards, and tools (Cutter, 2015). In the following section the assessment of resilience through the use of indicators will be further discussed.

4.2 *Resilience indicators for road bridges*

Indicators are quantifiable variables that represent selected characteristics of resilience (Cutter, 2015). Generally, individual indicators can be combined to create a resilience index (Kammouh et al. 2017b). The index illustrates multi-dimensional nature of resilience by aggregating multiple indicators, but also condenses its complexity into a single numeric value.

Recent bridge design codes consider performance uncertainty by including specific factors in the computation of structural resistance and load. However, this does not comply the uncertainties in loading and deterioration processes which make the prediction of lifecycle performance of bridges so complex. Prediction of time-dependent bridge performance under uncertainty may require the use of several indicators, thus, it is important to select adequate indicators to evaluate and predict the structural performance and functionality of bridges.

Significant research has been done on quantifying structural performance with deterministic and probabilistic indicators (Frangopol & Saydam, 2014). For example, system reliability measures, such as the probability of failure or the reliability index, are adequate for quantifying the safety of a structure with respect to ultimate limit states, but the system redundancy index is required to evaluate the availability of warning before a system's failure (Frangopol & Saydam, 2014). Additionally, performance indicators related to damage tolerance of structures, such as vulnerability and robustness are essential to consider for bridges under deterioration together with the indicators related to system safety.

Subsequently, it is required to consider multiple indicators when evaluating the seismic resilience of a bridge. Indicators must be weighted according to their contribution towards the resilience of the system, so that resilience is quantified as a normalized value of the area inside the enclosed shape made by linking the adjacent indicators weights or scores. Kammouh et al. (2017a) introduced and exemplified three weighting methods for resilience indicators based either on the dependence tree analysis or on the spider plot analysis (Kammouh et al. 2017a, Kammouh et al., in press).

Table 4 shows some examples of resilience indicators for bridges. They have been divided into two categories, 'soft' and 'hard' indicators, according to the difficulty involved in their measurement. Soft indicators, in general, can be determined qualitatively through surveys, data compilation or specific sample procedures, whereas hard indicators would require a higher effort, whichever computational, numerical or probabilistic. They can be calculated using mathematical formulas and/or matrices in order to approximate and understand the characteristic of the system under study.

Table 4. Examples of 'soft' and 'hard' type of resilience indicators for bridges

Soft Indicators	Hard Indicators
Speed Limit	Safety factor
Travel time	Reliability index
Physical access	Probability of failure
Weight allowance	Redundancy index
Operative utility systems	Vulnerability
Condition Ratings	Robustness

Indicators can be measured at a member or a component level, an overall structure or a system level, and a group of structures or a network level. For example, when evaluating stability problems, the performance is normally quantified at a member level, and normally, this is enough to ensure an adequate level of safety. However, when evaluating resilience, a member level approach might not provide the necessary information about the overall performance of the bridge or the bridge network.

5 LESSONS LEARNED

Since all bridges were subjected to similar ground motions, it is informative to compare the damage to them and the design features that influenced their structural performance and resilience. The following sections summarize the main issues raised and lessons learned from study of these bridges.

5.1 *Bridge intervention and route recovery*

An acceptable level of service was able to be restored within a short period of time. This can be attributed to a combination of a number of factors:

- Both NZTA and the Hurunui District Council, entities in charge of the bridges, had a Bridge Consultant (Opus International Consultants) in place to respond in the event of an emergency.
- The Bridge Consultant was familiar with the affected bridge stock and had information readily available through an established and maintained Bridge Information System.
- Network contractors (Downer and Sicon) were in place to respond in the event of an emergency.
- Emergency procedures in place to coordinate a response, inspections and works.
- Emergency bridge stock and contractor in place to erect Wandle Bailey Bridge.

- Clients, Consultant and Contractors experienced from Canterbury Earthquake Sequence on 2010-2011.
- Very knowledgeable client able to make informed decisions.

5.2 *Bridge response and behavior*

In modern design, there is a tendency to design more efficient structures, with longer spans and reduced foundations with a reliance on acceptable damage and dissipation through the use of PH. While this was largely observed on the bridges on the Inland Route, a few things that eventuated were undesirable such as the buckling and the fracture of the reinforcement and the large residual deformations for Wandle Bridge.

In this event, older bridges that had not been designed to modern standards performed better than expected. The likely reason for this is that these structures were designed with a greater level of redundancy, thanks to their shorter spans, more piers and more piles for the foundations.

For the Inland Route around the Waiau area, the earthquake demands from the Kaikōura Earthquake were severe, in excess of a 1000 year return period. As such, it is considered a significant earthquake with a low probability of occurring in the bridges' typical design life. On more critical routes, such as SH1, the design philosophy tends to shift towards designing to meet higher importance levels. For bridges on critical routes, the design would typically be for a return period of 2500 years at the DCLS and a return period in the order of 10 000 years at the Collapse Limit State.

5.3 *Reparability and influence of the structural form*

The structural scheme can play a part in the repair strategy, costs and hence resilience. In the case of monolithic bridges, repair of PH should be considered at the design stage. On the Lottery and Lower Mason bridges, the 1.5m diameter cylinder foundations located directly beneath the 1.0m diameter columns of the piers simplified the repair (Figure 8a). New reinforcement could be doweled into the existing pile and the pier re-cast, restoring the moment capacity of the column (Figure 8b). In contrast, for the Mason Bridge, repair is more difficult due to the shape and size of the pier cap (Figure 8c), as the geometry will make it very difficult to dowel in additional bars in and re-cast concrete. The result will be a significant repair cost and potential downtime for the structure.

Designing and detailing monolithic connections taking into account the repair strategy for different design level events would optimize the recovery after an earthquake event. Furthermore, if the bridges were designed for damage avoidance, incorporating energy dissipation and reparability, performance and recovery of the bridges could be improved. Hence, full restoration on the functionality of the route could be accomplished immediately after the earthquake.

Figure 8. Plastic hinge repair: (a) PH in Lower Mason Bridge; (b) repair of PH by epoxy grout bars; (c) geometry at Mason River Bridge PH zones.

6 CONCLUSIONS

Resilience indicators for bridges are important tools for predicting and evaluating the structural performance and functionality of bridges given a seismic event. They can help to characterize the basic elements of the structure under analysis and thus help to raise the awareness on the benefits of mitigation technologies and strategies, and making the business case for understanding the investments and results of enhancing resilience at different scales. This paper provides a general conceptualization of resilience indicators as a starting point for future research to be undertaken on the prediction of structural performance and resilience of bridges.

A brief overview of the structural performance and resilience shown by the most severely damaged bridges during the Kaikōura Earthquake has been given. At the time of initial inspection, the bridges were rapidly assessed and were generally only open to emergency traffic. Since then, temporary repair works allowed public access through these routes; however, long term repair or replacement strategies are still being considered by the managing authorities. From the perspective of life safety we can consider the overall performance as satisfactory.

The bridges, even if damaged, behaved as expected from design and allowed to reinstate transit of vehicles and restore the functionality in the route. This made possible to restore access to isolated communities within a relatively reasonable time of 23 days, thanks to the availability of an alternative route which has worked as a bypass. However, was this a satisfactory performance in a seismically active developed country or do we need to improve our current design philosophy so that we can warrant enhanced resilience during future events?

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