A MULTI-LEVEL COMPARISON BETWEEN PLASTIC HINGES AND DISSIPATIVE CONTROLLED ROCKING FOR BRIDGES – THE AWATERE RIVER BRIDGE CASE STUDY

Brandon MCHAFFIE1, Ana SARKIS2, Alessandro PALERMO3

ABSTRACT

In New Zealand, the 14th of November 2016 Kaikōura Earthquake sequence heavily damaged the bridge structures located in proximity of the epicenter. Among the affected bridges, the Awatere River Bridge is the newest structure, being constructed in 2007. This bridge performed as expected during the Kaikōura Earthquake, presenting only flexural cracking and spalling in the columns, however, still it suffered damage. Loss of functionality of road networks and the excessive direct and indirect costs due to downtime and non-structural damage have led to significant advancement in damage-resistant technologies. Dissipative Controlled Rocking (DCR) is a damage resistant technology that provides re-centering and energy dissipation with post-tensioned tendons and mild steel dissipaters. This combination provides strength, energy dissipation, displacement capacity/control and limits structural element damage. However, since codes do not capture the benefits of these technologies, from a competitive design view point there is little or no advantage of designing a low damage solution as the plastic hinge alternative is inherently cheaper.

This paper presents a detailed performance assessment of the Awatere River Bridge during the Kaikōura Earthquake through site inspections and subsequent analysis. Furthermore, a comparison of the bridge structure, which was seismically designed to modern codes, to an equivalent DCR structure will be carried out. The key outcomes are to illustrate the improved performance and resilience of DCR and the increased cost associated with the technology. In addition, costs and repair strategies can be compared for the structures after they are exposed to design level seismic events.

Keywords: Kaikōura Earthquake; Low damage; Dissipative controlled rocking; Bridge design; Case study

1 INTRODUCTION

The Kaikōura earthquake, of moment of magnitude Mw=7.8, occurred in New Zealand (NZ) early on November 14th, 2016. From the epicenter, about 60km south-west of the town of Kaikōura, the quake propagated along the east coast of the South Island and towards Wellington, rupturing several faults in the process. The transportation network of the entire north-east 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). Given the event is a summation of multiple fault ruptures over a large spatial domain, its complexity has resulted in the reassessment of many assumptions about earthquake processes. Furthermore, studying the performance of bridges during major earthquakes is of interest for reviewing the assumptions that are made during the seismic design or assessment of structures.

The Awatere River Bridge, located in SH1, is the most recently constructed of the bridges that received significant damage in the Kaikōura Earthquake. The bridge is of special interest because of its modern design and moderately long length. For this reason, the Awatere River Bridge is herein introduced as a case study. General observations on the performance are presented, based on observations made during

1PhD Candidate, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, brandon.mchaffie@pg.canterbury.ac.nz
2PhD Candidate, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, ana.sarkisfernandez@pg.canterbury.ac.nz
3Professor, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand, alessandro.palermo@canterbury.ac.nz
site inspections and on subsequent computational analyses performed.

Following, the bridge structure is compared to an equivalent structure designed implementing Dissipative Controlled Rocking (DCR). The final aim is to illustrate the improved structural performance and resilience of bridges designed with repairable energy-dissipating systems, when compared to the traditional monolithic design. Greater resilience capacity generally requires higher initial investment compared to prior practices. Therefore, the paper also addresses the issue of affordability of resilient design, as from a competitive design viewpoint there is little or no advantage of designing a low damage solution as the plastic hinge alternative is inherently cheaper.

2 AWATERE RIVER BRIDGE

2.1 Bridge description

The Awatere River Bridge was designed in 2004 and opened in 2007. It replaced a historic structure that combined both road and rail via a one-way through truss. The historic structure remains as a rail bridge only with the road deck removed. The bridge is made up of eight 27.4m internal spans and two end spans of 27.2m, for a total length of approximately 274m. The carriageway width is 10m, consisting of two 3.5m lanes with 1.5m wide shoulders. The superstructure consists of continuous precast pre-stressed concrete U beams and deck slab units (Figure 1a). Cast in-situ diaphragms and slab topping provide continuity over the piers. The nine piers are reinforced concrete twin-column portal frames (Figure 1b). The reinforced concrete columns of one meter in diameter are founded on 1.2m drilled piles. The clear distance between the pile and bottom of the cap beam is 5.5m. At the abutments, the superstructure U-beams are seated on elastomeric bearing pads which allow rotation and longitudinal movement. Transverse movement is restricted by concrete shear keys cast integral with the abutment seating beams.

![Figure 1. Awatere River Bridge: (a) longitudinal view of super-structure (b) sub-structure](image)

The bridge is located 114 km from the epicenter of the Kaikōura Earthquake. The peak horizontal ground acceleration (PGA) recorded in Seddon, at the SEDS strong motion station, 1.5 km south of the bridge, was 0.76g. As shown in Figure 2 below, the Kaikōura earthquake approximately represented a 1/500 year event for this structure which has a period in the transverse direction of 0.8 seconds.

![Figure 2. Spectral acceleration and design spectra, SEDS strong motion station](image)
2.2 **Observed Structural Damage**

The Awatere River Bridge sustained flexural cracking at the piers, linkage bar anchorage and shear keys and residual displacement of the bearings. The bridge was also subjected to strong ground shaking in the 2013 Seddon earthquake which resulted in column hinging and linkage bar damage (Divers, 2013). Therefore, some of the spalling was attributed to spall repairs undertaken following 2013 Earthquake. Figure 3 below shows schematically the elevation view of the bridge with the main damage observations. Cracking in the cover concrete within 1.5m of the top of the pier columns was notable and about half of the columns also presented spalling at the top of the columns in the pier and head-stock interface (Figure 4). In addition, a few pier columns, also had cracking and spalling at the base of the piers, just above the top of the pile (Figure 4). The cracking and spalling in the pier columns mainly occurred at locations where the concrete had been previously repaired with mortar after the 2013 Seddon earthquake.

Damage to the columns appears to be from hinging within the plastic hinge zone of the portal frame (pier and head-stock substructure) which resulted from the strong horizontal shaking in a transverse direction (Palermo et al., 2017). The hinging appears to have initiated yielding of the longitudinal reinforcement in the columns. The longitudinal reinforcement also appears to have locally lost its adhesion to the concrete mortar indicating a slight bend in the bar. While the confinement in this zone of the columns appears to be just within code requirements it may be advisable to improve the confinement considering this observation of bent bars. After the earthquake this was a common observation, with several other structures illustrating severe buckling of longitudinal bars highlighting a problem in anti-buckling requirements for bridge piers not captured in the current NZ standards.

The bridge deck at the abutment A appeared to have a residual displacement in the southeast direction based on the residual displacement of the elastomeric bearings (Figure 5). Pounding damage occurred at the corners of the northern abutment end diaphragm (Figure 5). It appears the bridge moved transversely and the diaphragm made contact with the abutment shear keys causing a section of previously repaired concrete mortar to break away (Palermo et al., 2017).

**Figure 3. Damage observations Awatere River Bridge elevation (modified from Palermo et al., 2017)**

**Figure 4. Damage observations at the piers**
2.3 **Effects of damage**

Following the earthquake, from site inspections and subsequent analysis (Opus, 2017), the next conclusions were made:

- Damage on the bridge did not affect the live load capacity.
- The seismic capacity of the bridge in the longitudinal direction appeared to be unaffected as the seating and overlap at the abutments remained unchanged and the superstructure remained continuous across the piers.
- The damage sustained during the earthquake may have resulted in a minor deterioration in the transverse seismic capacity of the pier columns as some of the ductility of the longitudinal reinforcing bars has been drawn upon as they begin to yield. The subsequent response of the bridge to transverse shaking may be as the hinges have softened with the concrete transitioning from un-cracked to cracked condition. This could be fixed by improved confinement of areas with spalled cover concrete. Nevertheless, the bridge has returned to its neutral position following the shaking and the deterioration is probably negligible in terms of collapse limit state.

The decision to repair the structure will remain up to the client and the risk they are willing to take with the structure. Due to the cost of repair it is likely that the client will choose to accept a reduced seismic capacity for the structure. Meanwhile, accelerometer instrumentation has been installed on the bridge and on the ground nearby to investigate the seismic response of the bridge as a function of the ground shaking (Wood, 2017).

3 **AWATERE RIVER BRIDGE RESPONSE- PLASTIC HINGE ANALYSIS**

3.1 **Pushover Response**

To investigate the column damage static pushover analysis in the transverse direction of the piers was performed using a finite element model of the section. The tributary mass assumption was used to estimate the transverse response of the central piers. The dynamic weight of 4,100kN was taken as the tributary mass. This includes the U beams, topping, pier cap, diaphragm and one third of the column weight. Table 1 summarizes the inputs used for the analysis of the column and Figure 6 shows the pushover response of the top and bottom sections of the pier.
Table 1. Inputs for section analysis (top section of pier)

<table>
<thead>
<tr>
<th>COLUMN PROPERTIES</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>1000 mm</td>
</tr>
<tr>
<td>Diameter of long bars</td>
<td>25 mm</td>
</tr>
<tr>
<td>Diameter of trans steel</td>
<td>20 mm</td>
</tr>
<tr>
<td>Spacing of trans steel</td>
<td>150 mm</td>
</tr>
<tr>
<td>Axial load (single col)</td>
<td>2050 kN</td>
</tr>
<tr>
<td>Concrete comp strength</td>
<td>52 MPa</td>
</tr>
<tr>
<td>Long steel yielding stress</td>
<td>550 MPa</td>
</tr>
</tbody>
</table>

Approximate displacement demands on the piers as a result of the Kaikōura earthquake were obtained using a displacement-based method. Recorded accelerations in Seddon were used to derive the displacement spectra. This can now be done at the site with the accelerometers installed at the bridge; however, at the time of the Kaikōura earthquake the accelerometers were not yet installed on the structure. The ground accelerations recorded at Seddon were rotated to give the transverse acceleration input at the Awatere River Bridge.

The damped displacement response spectrum is estimated below (see Figure 7a). For the approximate transverse period of 0.8 seconds the bridge pier displaced approximately 90mm. In addition, the earthquake was in the order of a 1/500 year event and so a comparison was made to the ADRS demand curve which further verified an estimated response of around 90mm (see Figure 7b). The resulting ductility demand for this displacement is 2.5 for the bottom hinge.

3.2 Time History analysis

To further investigate the performance of the monolithic connection, Non Linear Time History (NLTH) analysis was performed. A lumped plasticity model was adopted for the simplified model. The plastic hinge was modelled with a rotational spring using a modified Takeda hysteresis rule with properties to match the moment rotation relation obtained through sections analysis.

3.2.1 Estimating Modal Damping Ratios

Estimating damping ratios for real structures subject to moderate ground shaking is very difficult and hence there is relatively sparse information on this important topic. Typically, recommended damping ratios are based largely on the few detailed case histories as well as expert judgment. The analysis presented here is somewhat unique in that the structure is monitored by accelerometers. In addition, the ground motion input is recorded by an accelerometer on the ground at the site (Wood, 2017). Given the
input ground motion and the response of the structure the model can be calibrated by altering the damping ratio to represent the recorded response of the structure. Unfortunately, thus far the recorded ground motions have been relatively small so the damping that represents the response could be slightly lower than that expected during a design level event. However, it does provide a very good estimate and verifies that the modelling assumptions correctly capture the response of the structure. In this case the response to the recorded motion represented the response with a damping ratio of 3%.

Figure 8 shows very good agreement between the calculated and recorded responses indicating that the input motion from the south end was representative of the motion at the base of the central pier. This would indicate that travelling wave effects were probably not very significant. A complete model could be set up and analyzed to investigate travelling wave effects. Moreover, it may be possible to estimate the apparent wave speed from the two sets of records measured by the ground instruments. However, the data recording frequency of 50Hz was too low to allow this; hence, it has been increased to 200Hz and the time lag effect can be investigated in future aftershocks (Wood, 2017).

![Figure 8. Comparison between recorded displacement history and output from Ruaumoko with a 3% damping ratio](image)

### 3.2.2 Kaikōura ground motion

Knowing the damping ratios for the structure the monolithic model was subjected to the Kaikōura ground motion recorded by the SEDS accelerometer located in Seddon.

![Figure 9. (a) Accelerations recorded by SEDS rotated to the transverse direction for the structure; (b) displacement time history response of the model subjected to rotated SEDS motion](image)

The analysis indicates that the pier has displaced approximately 102mm at the peak response and this displacement can be used to estimate the maximum strain that the steel has reached. However, the residual strain capacity is still difficult to predict, particularly when considering effects such as low cycle
fatigue and strain ageing. NLTH shows the strain level the steel reached during the Kaikōura earthquake is in the order of 0.025 or 2.5%. This is relatively low given the ultimate strain capacity is usually in the order of 12% for G500 seismic reinforcement. On the other hand, this is not the first seismic event the structure has undergone and the same repaired cracks appear to be opening, indicating that the strains could be concentrated at these locations rather than spreading uniformly along the length of the plastic hinge. Therefore, the strains in these bars could be even higher than 2.5%.

Is important to remark that, for most existing structures in NZ, strain ageing of G300 bars would have a significant effect on the residual strain capacity of the bars. Loporcaro, Pampanin, & Kral (2014) indicate that strain ageing may reduce the limit of uniform extension of G300 bars from 20% to 10%. This effect is not present in G500 bars although these bars have a lower ultimate strain capacity than G300bars.

The Damage Control Limit State (DCLS) strain for steel is 6%. If the Awatere River Bridge was designed to this standard it would be expected that the bars could reach this strain in a design level event. Knowing that the strain capacity of the bar is likely to be at least 10% it is accepted that even with some residual strain this structure would safely survive another design level earthquake. The question then becomes how will the structure will perform in a Collapse Avoidance Limit State (CALS) event now that some strain capacity has been utilized in addition to the combination of fatigue issues? To determine this with any certainty is very difficult and would require complex analysis and invasive material testing (such as removal of the rebar) that would affect the performance of the structure. In theory, once a bar has strain hardened plastic deformation should shift meaning that the structure could still perform adequately in a design level event. However, this adds another level of uncertainty since there is no easy way to tell over what length plastic deformation has occurred. Some assessment can be done at a cost which again will give an indication of the maximum strain but no information on residual capacity. The uncertainty surrounding these connections is where Dissipative Controlled Rocking (DCR) has a huge advantage.

4 DESIGN ALTERNATIVE: DISSIPATIVE CONTROLLED ROCKING

Research on the seismic design of reinforced concrete bridges, has focused on improving performance in order to reduce physical damage and residual drift associated with plastic hinging. Developed damage-resistant technologies intend to minimize post-earthquake damage in the bridge structure providing continued functionality for the transportation network. Dissipative Controlled Rocking (DCR) is a connection type developed to be used in place of traditional plastic hinges. It incorporates Post Tensioning (PT) to provide re-entering, mild steel dissipaters to provide energy dissipation and steel armoring at the interface to prevent concrete degradation (Figure 10).

![Figure 10. General layout of a pier with a low damage DCR connection tested at the University of Canterbury](image)

This combination of PT and dissipation leads to a flag-type hysteresis as illustrated in Figure 11 below. The behavior limits residual displacement after an earthquake provided the PT and axial load moment contribution (re-centering) is larger than the moment contribution of the mild steel. The ideal thing about
these connections is that after a design level event the connections can be repaired to 100% of the original capacity by replacing the external dissipaters.

![Figure 11. Idealized hysteresis behavior of DCR](image)

The main advantages of rocking and hybrid connections, detailed appropriately, are the elimination of residual drifts and the minimization of structural damage, which are desirable features for resilient structures. Low damage technologies on bridges facilitate repair and inspection by incorporating external replaceable dissipaters that can be unbolted and reinserted without any need for temporary supports or restraints. Since the extent of damage is significantly limited, no significant cracking away from the main rocking interfaces is expected even after a collapse avoidance limit state and no significant spalling is expected at or near the rocking interfaces. Additionally, unlike plastic hinge design, low damage systems prevent residual drifts on the bridge due to its self-centering nature. Finally, control over damage leads to a minimized traffic disruption after an earthquake reducing indirect costs due to downtime all around the transportation network.

The main drawback of low damage technologies is the novelty of the technology and slight increment of construction cost, which could be offset by the minimized traffic disruption and safer repair methods as well as the other advantages enlisted before. However, those benefits are not easily measurable nor are they easily understood at a design level by owners and decision makers. Therefore, when pricing an innovative mitigation technology it is important to look at the long term profitability by comparing the cost of vulnerable infrastructure with the additional investment of making it more resilient.

### 4.1 Low damage design of the Awatere River Bridge

To compare the performance of a DCR connection and monolithic connection, the DCR connection was designed to have approximately the same design displacement at the design level and the same pier diameter as the monolithic pier. However, the DCR connections have a bigger design force due to a reduced damping associated with this type of connections and increased stiffness due to PT. The initial analysis was done using an ADRS pushover as shown in Figure 12. The damping used for the ADRS curve was 11% in accordance the PRESSS design handbook (Pamapanin et al, 2010). The monolithic connection was compared to a ADRS with 18% damping in accordance with the Displacement-Based Seismic Design of Structures (Priestley, Calvi, & Kowalsky, 2007). Figure 12 shows that in a DCLS event (1/2500 years) both connections will reach an ultimate displacement of approximately 160mm.
Table 2. Inputs for DCR section analysis

<table>
<thead>
<tr>
<th>COLUMN PROPERTIES</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>1000 mm</td>
</tr>
<tr>
<td>Fuse Diameter</td>
<td>24 mm</td>
</tr>
<tr>
<td>Num Fuses</td>
<td>16</td>
</tr>
<tr>
<td>Axial load</td>
<td>2050 kN</td>
</tr>
<tr>
<td>Concrete comp strength</td>
<td>52 MPa</td>
</tr>
<tr>
<td>Long steel yielding stress</td>
<td>500 MPa</td>
</tr>
<tr>
<td>PT diameter</td>
<td>75 mm</td>
</tr>
<tr>
<td>Initial PT force</td>
<td>400 kN</td>
</tr>
</tbody>
</table>

Figure 12. ADRS pushover curve for the DCR and monolithic connections

4.2 Time History analysis

To further investigate the performance of the DCR connection, and to allow a comparison to the monolithic connection, NLTH analysis was performed. As illustrated in Figure 13, a lumped plasticity model was adopted for the simplified model. This type of modelling has been shown to accurately predict the behavior of both monolithic and DCR connections (Palermo, Pampanin, & Marriott, 2007). The DCR connection uses two rotational springs in parallel. One rotational spring is assigned a Non Linear Elastic rule to represent the self-centering contribution from the PT and axial load. The other rotational spring is assigned a Ramberg-Osgood hysteresis rule to represent the mild steel dissipaters (Carr, 2004). The springs were then calibrated to represent the moment-rotation behavior provided by each contribution.

As shown in Figure 14 the results from the NLTH indicate a maximum displacement of 106mm which is very similar to the 102mm expected by the monolithic connection so as designed the two systems have very similar performance at the level of shaking in the Kaikōura earthquake.
5 COMPARISON BETWEEN PLASTIC HINGE AND DCR DESIGN

5.1 Structural performance and expected damage

As shown in the Non-Linear Time History analysis the response to the Kaikōura earthquake resulted in a very similar maximum displacements of approximately 100-110mm for both types of connection. However, in larger earthquakes the DCR connection will undergo lower displacements given the significant stiffness increase due to the engagement of PT. In addition, due to the unbonded length of the external dissipation devices these connections can also sustain larger displacements before rupture. The result is that from a response perspective, the DCR connection has better performance than the equivalent monolithic connection.

Furthermore, when damage is considered the DCR connection is superior. As shown in Figure 15a,b the damage to plastic hinges can be severe and determining residual capacity and repair strategies is very difficult. With the DCR connections all damage is confined to dissipaters (Figure 15c) which can, in some cases, be replaced for less cost than testing and analyzing the equivalent plastic hinge. Moreover, the damage and repair strategy is the same irrespective of the size of the earthquake and type of structure which will greatly improve recovery times. Even in a Collapse avoidance limit state (CALS) DCR connections can be designed to have damage that is limited to rupture of the dissipaters (Figure 15c). The equivalent monolithic connection would undergo concrete degradation, yield/fracture of rebar leaving the structure in a state that could require a full replacement (Figure 15a,b) and if not costly repairs. This would be an acceptable outcome by current practice. However, having to rebuild a structure that has sustained only a DCLS event may be less acceptable and currently there is significant cost associated with testing and analysis to determine the level of residual capacity in the structure. This is where the DCR connection becomes affordable, as the cost associated with returning a DCR structure to full functionality is in the same order as the cost associated only to the assessment of a monolithic structure.

5.2 Effect of reparable on the design framework

In conventional forced-based seismic design, the displacement ductility factor ($\mu$) provides a means by which, a structure can be designed for lower seismic loads on the basis of accepting more damage in a particular DCLS event. For this reason, the NZTA Bridge Manual (NZTA, 2016) places a limit on the acceptable displacement ductility factor. As shown in Figure 16, the acceptable level of ductility is largely based on the accessibility and hence reparableity of the plastic hinge regions. This provision strongly influences the ease of access for inspection and repair.
Building on the notion that reparability influences ductility, an alternative seismic design philosophy is proposed. This is based on the premise that as long as the appropriate CALS (which ensures life safety) is satisfied for a particular structure, it would be justifiable to allow higher ductility, based on the economic and social impacts of the expected damage and speed of repair. This would allow structures which have been specifically designed to achieve limited damage that can be easily repaired, to be designed to a higher ductility provided there is no compromise on life safety. For this reason DCR or similar connections can be designed for CALS and the DCLS can be altered based on the economic effects of doing so. In this way the client can decide at which design level replacement of dissipaters is acceptable. This is atypical when compared with traditional design methods where structures are designed for DCLS and CALS is checked. Reducing the DCLS demands will decrease base shear demand on the structure which, for example, could reduce the required moment capacity of the piers. The increase in cost associated with this technology can then be potentially offset by reducing over-strength demands elsewhere in the structure. Equivalently, a more frequent return period for the DCLS can be introduced which increases the probability of a DCLS event occurring over the life of the structure on the basis that the damage is limited and repair is economically viable. Furthermore, 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. 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 instead of a traditional monolithic design the bridge was designed with a technology such as DCR, functionality of the bridges could be restored to its full value immediately after the earthquake.

5.3 Costs

Currently it is hard to make a direct cost comparison between the two solutions for the Awatere River Bridge. However, on 2016, the first bridge with low damage connections, the Wigram-Magdala Bridge, was constructed in Christchurch. The structure incorporated DCR along with steel incased concrete. The estimated increase in cost for the structure when compared to a traditional design was $200,000 which over a total project cost of approximately $30 million represents less than a 1% increase. This amount is recoverable in operating costs during the life of the structure. Therefore, when pricing a resilient design is important to look at the long term profitability by comparing the cost of vulnerable infrastructure with the additional investment of making it resilient.

5.4 Importance of investing in more resilient infrastructure

Within the Technical Guidance for Engineering Assessments in NZ (2017), 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 bridge 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.
Resilient infrastructure is fundamental in three different levels. First, from the perspective of economy where increasing risks and disasters combined with non-quality infrastructure results in a constant reconstruction. Second, from the human perspective where failure of infrastructure will ultimately loss of wellbeing or loss of lives, increasing the human costs after the disaster. And finally, from a systemic point of view, above all communities want to keep themselves into a scenario of having insurance systems to cover them, however, lack of resilient infrastructure could lead into a scenario where they are systematically unsustainable.

6 CONCLUSIONS

Initially, this paper investigated the damping ratio for the Awatere River Bridge by comparing the recorded response with outputs from a NLTH analysis. This showed that in the transverse direction a damping ratio of 3% accurately predicted the response of the structure. For simplicity, longitudinal response was not included in this paper, but a damping ratio of around 7% accurately captured the response in this direction. The increase in damping is likely associated with the additional soil interaction at the abutments. Given that the input motion from the abutment accurately predicted the response at the top of the central pier it is unlikely that travelling wave effects were significant. The post-earthquake benefits of applying DCR were then investigated. NLTH analysis showed that as expected the displacement response was similar. Both connections reached a maximum displacement of around 105mm. The key differences benefits of DCR over the monolithic connection are the zero residual displacement after a design level earthquake, reduced structural downtime, reduced post-earthquake analysis, reduced cost to repair and a common repair strategy across all structures. This leads to a far more resilient network, particularly after seismic events which have caused structural damage which is accepted. Another design philosophy was introduced where by the reparability of the DCR connections allows it to have a more frequent DCLS return period provided CALS and life safety is still maintained. This idea would allow asset owners to make decisions on the level of risk accepted for a structure and consequently the frequency of required dissipater replacement based on the economics of doing so.

7 REFERENCES


