NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

Cyclic lateral loading of the Whirokino Trestle pile foundations

P. Chigullapally, L.S. Hogan, & L. Wotherspoon
The University of Auckland, Auckland.

ABSTRACT

In New Zealand, over 60% of State Highway bridges were designed and built during eras with low seismic provisions and most of these bridges are supported by pile foundations. An improved understanding of the in-service performance of these bridge foundations is a key aspect of seismic resilience. However, there is limited data on the performance of these foundations in the field. A unique opportunity to study bridge foundations from this period has been presented during the replacement and demolition of the Whirokino Trestle in Foxton, New Zealand. A field-testing program involving static cyclic tests was developed in order to characterize the lateral behaviour of isolated pile foundations that are embedded in loose to medium dense sands. These results highlighted the significant influence of loading, plain round bars, and soil nonlinearity and gapping on the seismic response of older bridge piles. Even with the use of plain round bars and with widely spaced transverse reinforcement in these piles, there was no loss of strength observed at large displacements and through multiple cycles of loading, suggesting a better than expected performance of these older pile foundations.

1 INTRODUCTION

Bridges designed between 1930 and mid 1960`s in New Zealand were designed to resist only 0.1 times the superstructure weight and lacked any code provisions for seismic detailing of reinforcement (Hogan et al., 2013). Even with light reinforcement, many bridges from this period have performed well during previous earthquakes (Lew et al., 2020). The performance of pile foundations subjected to large deformations is an area where an improved understanding would be beneficial, especially based upon the observed damage following the 2010/2011 Canterbury earthquake sequence and 2016 Kaikōura earthquake (Giovinazzi et al., 2011; Palermo et al., 2010; Stringer et al., 2017).

During the demolition of the Whirokino Trestle Bridge, a unique opportunity to test the pile foundations of a bridge constructed in the 1930's under large displacements was presented. This paper presents some of the preliminary results from a series of static cyclic tests that were performed to characterize the response of the bridge pile foundations. The characteristics of the bridge and the field-testing methodology are first presented, followed by discussion of a subset of the test results and preliminary findings.

2 WHIROKINO TRESTLE BRIDGE

The Whirokino Trestle Bridge was located on State Highway 1 (SH1) in Foxton, New Zealand. The bridge was constructed by the New Zealand Ministry of Works in 1939, and was demolished and replaced by a new bridge in 2020. The bridge was a 1.2 km long reinforced concrete structure with 90 spans and a span length of 12.2 m (a view of two spans of the bridge is shown in Figure 1). The superstructure was supported by four 457 mm x 405 mm reinforced concrete columns connected to a pile cap. The foundation system consisted of five approximately 9 m long reinforced concrete piles that were 406 mm in diameter and spaced at 1.55 m centre-to-centre spacing. The pile reinforcement consisted of eight 22 mm diameter longitudinal bars and 4 mm diameter spiral transverse reinforcement at 50 mm spacing over the full length of the pile. A typical pier pile can be seen in Figure 2. Plain round bars were used in the construction of the entire bridge.

Figure 1: Side view of two spans of the Whirokino Trestle Bridge in Foxton, NZ.

2.1 Site characterisation

The soil profile and the pile test location was characterised using CPT testing and other existing geotechnical investigation data near the test pile. This showed that there were silty sands to sandy silts along the length of the pile, with increasing CPT tip resistance with depth. The top 2 m of soil had an average tip resistance of 4 MPa at which point it increased to 18 MPa with depth until 2.5 m and it further increased to 24 MPa until 4.5 m. The bearing layer of the pile foundations had a tip resistance in excess of 32 MPa, transitioning into very dense sands where the CPT met refusal.

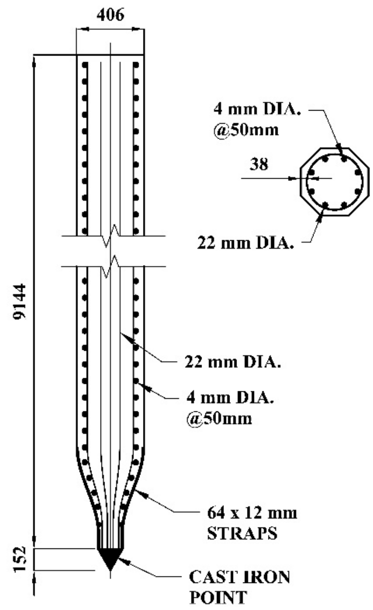


Figure 2: Whirokino Trestle Bridge pile details.

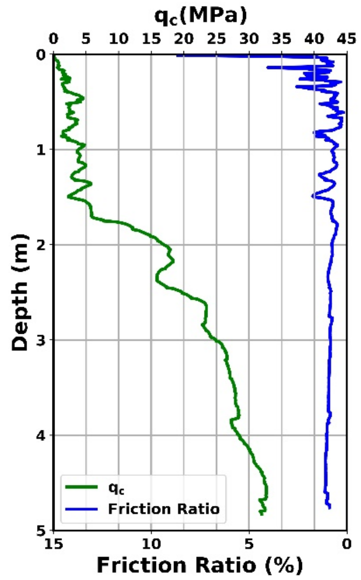
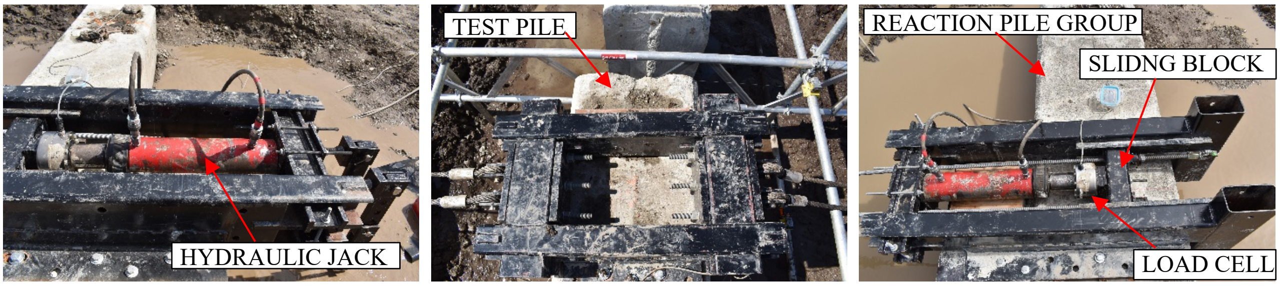


Figure 3: CPT log at Pile 1

3 TEST SETUP

To access each test pile, the superstructure was demolished down to the pile cap, and each pile was then isolated from one another by saw cutting through the pile cap to create a 400 mm gap. No axial load was applied during the testing due to the difficulty in applying this load in the field. As the axial load ratio of the pile was, less than 3% the impact of the axial load on the response of the pile was assumed to be negligible. A test pit, which was 1 m wide on each side of the pile, were created to access the test piles. These test pits were around 1200 mm deep for a typical test pile, which was around 200 mm below the bottom of pile cap. This was required to install the instrumentation on the top of the pile and to observe soil gapping. To control the soil conditions, prior to the start of every test and test setup installation the test pits were dewatered, with the soil saturated from the ground surface.

To load the test piles, the neighbouring intact pile groups were used as a reaction frame. Steel loading frames were attached to the test piles and the reaction piers, and connected by two 19 mm diameter wire ropes. The setup of the loading frames and wire ropes is presented in Figure 4. A 30-ton capacity hydraulic jack with 300 mm stroke was installed inside each of the loading frames at the reaction piers to apply horizontal loading to the top of the test piles. The hydraulic jack pushed a sliding block between the steel loading frames and this applied a tensile load through to the wire ropes (Figure 4). The slack in the wire rope was removed manually to minimise the loss of stroke, and the test pile and reaction pier were then pulled towards each other. The loading was applied until the target load or displacement was reached for the initial direction of loading. The wire ropes were then detached on the loading side of the test pile and were attached to the reaction pier on the other side of the test pile to continue the loading of next half-cycle. The wire ropes were attached and reattached after each half-cycle of loading until all the cycles were complete.

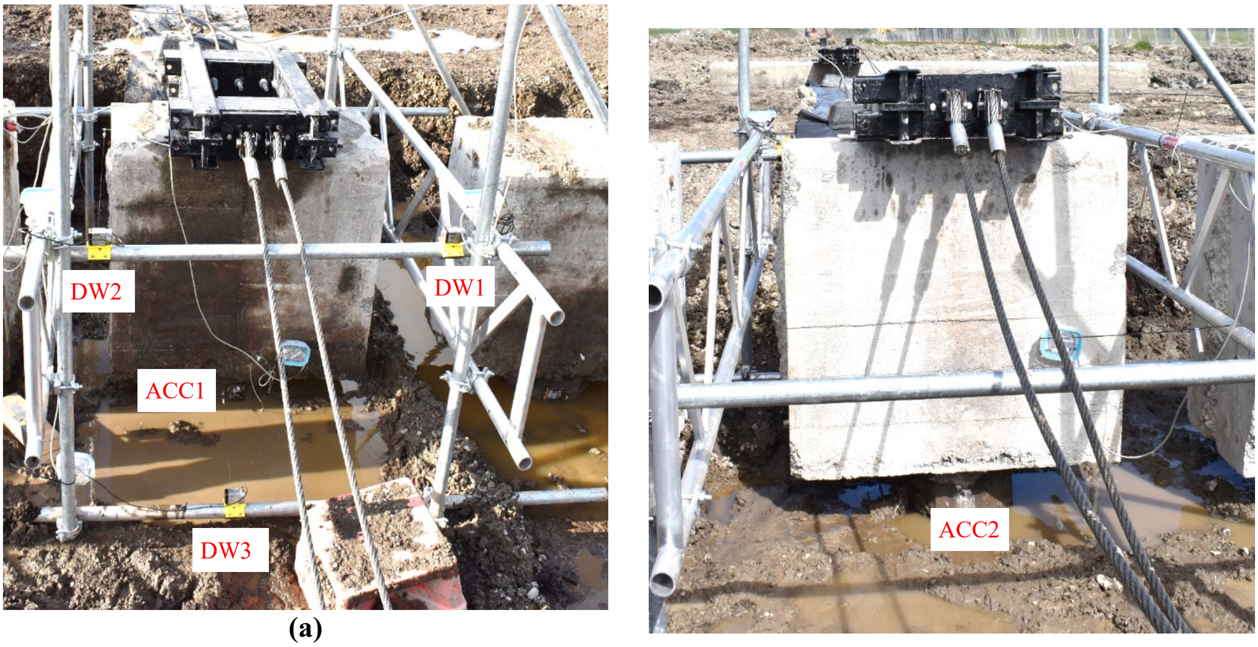


(a) **(b)** **(c)**

Figure 4: Top view of test setup (a) left reaction pile (b) test pile (c) right reaction pile

3.1 Instrumentation

To measure the horizontal displacement of the pile during testing, two draw wires were positioned at the top of the test pile, and one draw wire was positioned at the bottom of the test pile, just above the ground surface. A 25-tonne load cell was connected to each hydraulic jack inside the loading frame on the reaction pile to measure the applied load. All the draw wires were attached to a rigid reference frame that was anchored to the ground outside the zone of influence of soil deformation. Two unidirectional accelerometers were connected to both faces of the test pile in the direction of loading to measure the ground level rotation.



(a)

Figure 5: Locations of instruments on the test pile (a) South face (b) North face

3.2 Testing sequences

Testing was conducted to understand the effects of different loading protocols on the response of the piles. Pile 1 was tested using a single large cycle of lateral loading and to characterise response in the absence of any prior displacements. Pile 2 was tested under increasing cyclic loading to characterise the cyclic response, with initial cycles prior to pile yield force controlled, shifting to displacement controlled once yield had been reached. Table 1 summarises the testing sequences applied to Pile 1 and Pile 2.

Table 1: Target peak load/displacement values at the end of cycle at the top of pile

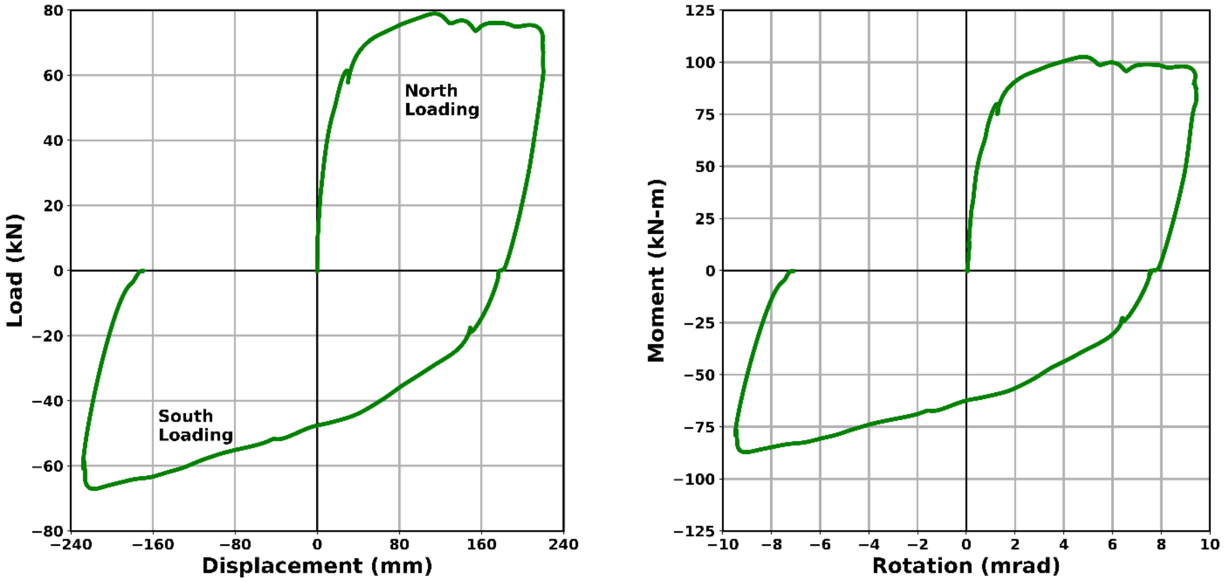
Cycle	Pile 1	Pile 2
1	250 mm	20 kN
2	-	40 kN
3	-	60 kN
4	-	50 mm
5	-	100 mm
6	-	150 mm

4 FIELD TESTING OBSERVATIONS

In these results, the average of the displacements from both the draw wires at the top of the pile was used to represent the pile top displacements. The average of the secant stiffness from the north and south half cycles were used to represent the cycle stiffness. The average of the rotation from the accelerometers on the north and south faces of the pile was used to represent the pile rotations. During testing there was no cracking observed at the interface between the pile cap and the pile, meaning the rotations calculated using the displacements at the top of pile cap and near the ground surface were very similar to the rotations calculated from the accelerometers. The amount of energy dissipated in a cycle was calculated as the area under the load-displacement hysteresis loop for the corresponding cycle.

4.1 Single cycle pushover testing

The load-displacement response at the top of pile cap and the moment-rotation response at the ground level from Pile 1 are presented in Figure 6. The load-displacement response suggests that Pile 1 remained elastic until a load of around 60 kN when loaded in the northern direction, corresponding to a pile head displacement of 28 mm. The change in slope of the load-displacement response during this range was a result of soil gapping and soil compressive nonlinearity. After this point there was a rapid reduction in stiffness observed, likely corresponding to the development of a plastic hinge below the ground level. The nominal capacity of the cross section occurred at approximately 78 kN, and after this point, the deformation of the pile-soil system increased without any strength degradation. The pile was pushed until a peak displacement of 220 mm in the northern direction and was released back due to lack of stroke in the hydraulic jack. When the pile was unloaded completely, there was a residual displacement of 175 mm in the north direction. The pile was then pushed in the southern direction to a peak displacement of 227 mm, and the load applied at this point was 67 kN. When the pile was unloaded completely, a residual displacement of 170 mm was observed in the southern direction. The peak pile rotations and peak moments observed were 9.4 mrad and 100 kN-m with a residual rotation of 7.6 mrad in the north direction and 9.3 mrad and 88 kN-m with a residual rotation of 7.1 milliradian in the south direction. There was a 5-10 % reduction in secant stiffness of the pile when it was loaded in the southern direction compared to the secant stiffness of the pile when loaded in the northern direction. The loading on the south direction for similar displacements in the northern direction was likely lower because of the loss of the stiffness due to incomplete closure of cracks in the pile foundation and the effects of soil gapping (Figure 7) around the pile after it was first loaded in the northern direction.



(a) (b)

Figure 6: Pile 1 (a) Load-displacement measured at the top of the pile cap, (b) Moment-rotation measured at ground level

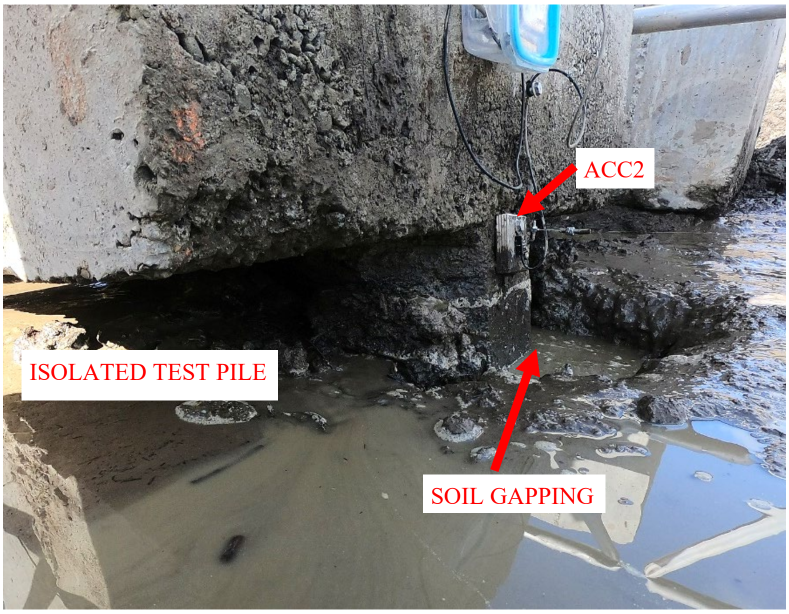
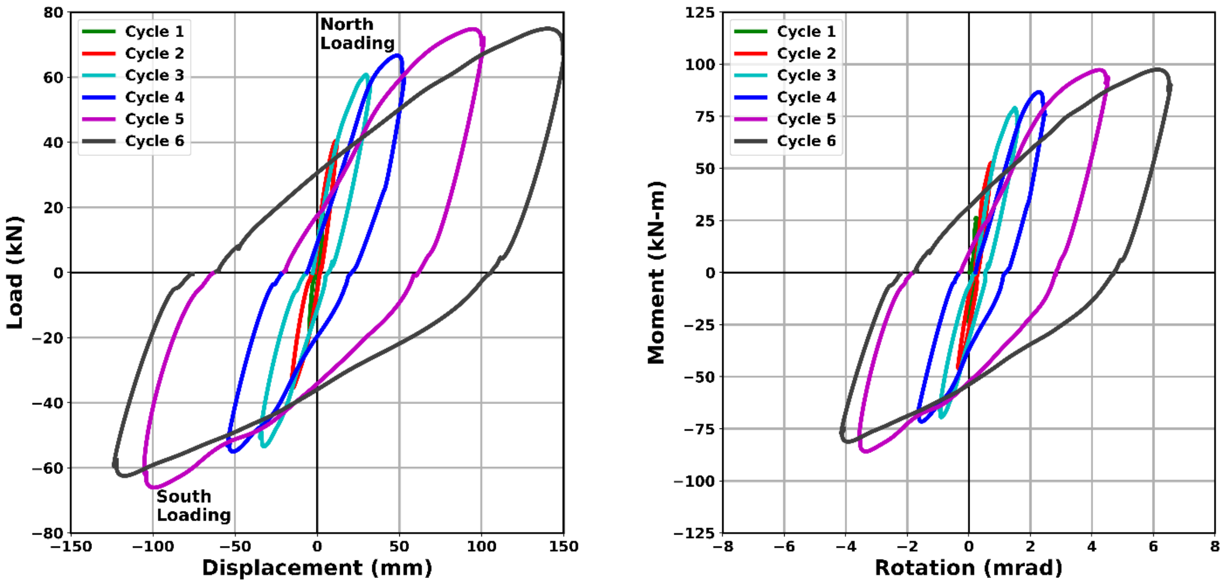


Figure 7: Soil gapping observed at ground level during testing

4.2 Multi-cycle pushover testing

The load-displacement response at the top of the pile cap and the moment-rotation response at the ground level from Pile 2 are presented in Figure 8. The load-displacement response suggests that Pile 2 remained elastic in the first three cycles when loaded in both the directions, with stiffness changes again from gapping and soil nonlinearity. The average peak load at end of third cycle was 57 kN, corresponding to an average peak displacement of 32 mm, representing a 60% loss in the average secant stiffness in third cycle compared to that of the first cycle. After this point, there was a rapid reduction in stiffness observed in the fourth cycle in both the directions, likely corresponding to the development of a plastic hinge below the ground level. At the end of fourth cycle, there was a 75% loss in cycle stiffness compared to that of the first cycle. In the fifth and sixth loading cycles, the average peak displacement at the top of the pile cap was 100 mm and 133 mm respectively, but the maximum loading reached was 75 kN in both the cycles. At this point, there was an 85-90% loss in cycle stiffness compared to the cycle stiffness in the first cycle. After the fifth cycle the deformation of the pile-soil, system increased without any strength degradation. Based on the observations from Pile 1, the pile-soil system could be reaching its peak loading capacity or would have seen a slight increase if the pile was pushed to greater displacements. The residual displacements at the end of each loading cycle in northern and southern directions were within 5-10% of each other, with higher values in the southern direction. An average cycle pile rotation of 5.15 mrad was observed and the corresponding average peak moment was 90 kN-m. There was an average residual rotation of 3.45 mrad when the load was completely removed.

In Pile 2, the rate of increase for the amount of energy dissipated was exponential before the pile yielded (Cycle 1 to Cycle 3). In this range energy, dissipation was controlled by increasing levels of soil nonlinearity. The rate of increase of energy dissipation was linear once the pile yielded (Cycle 4 to Cycle 6), with energy dissipations significantly higher than those prior to pile yield. The amount of energy dissipated in Cycle 3 was around 7 % of the energy dissipated in Cycle 6.At these higher loading levels this would be dominated by plastic hinge development and bar slipping in the critical areas. .



(a) **(b)**

Figure 8: Pile 2 (a) Load-displacement measured at the top of pile cap, (b) Moment-Rotation measured at ground level

4.3 Comparison of Pile Test Results

The load-displacement characteristics suggest that the piles in both tests behaved elastically until approximately 60 kN or up to a pile head displacement of approximately 30 mm. After this point, a plastic hinge develops, leading to the reduction in stiffness of the system. After a certain point both the piles were able to retain their strength through large displacements and multiple cycles of loading, suggesting a better than expected performance of these older pile foundations in spite of having less transverse reinforcement. .

There was no effect of cyclic loading on the piles, as both Pile 1 and Pile 2 reached similar peak loads. Pile 2 would have reached similar displacement as Pile 1 without losing any strength, if it were pushed to larger displacements in its last cycle. In both the piles, the secant stiffness of the pile in southern direction was relatively lower compared to the stiffness of the pile in northern direction.

The residual displacements for Pile 1 are higher than Pile 2 because Pile 1 was pushed to a larger peak displacement. The residual displacement at the ground level was less than 15 % for displacements below 30 mm at the top of the pile cap in both loading directions. The residual displacement at the ground level was 40 % to 80 % for displacements between 30 mm and 220 mm at the top of pile cap in both the loading directions. The residual displacements for loading in the southern direction were larger than those when loading in the northern direction The residual displacement increased linearly with increase in peak displacements at top of pile cap at all displacement levels. The rate of increase in displacements was higher for displacements above 30 mm compared to displacements below 30 mm.

5 CONCLUSIONS

Single and multi-cycle pushover tests were performed on isolated piles with plain round bars in sandy soils at two test locations of the Whirokino Bridge. The test site and loading protocols were chosen in order to capture the response of piles under different loading conditions and the effects of variation in stiffness due to gapping and cracking of the pile. Based on the field-testing results, the following conclusions can be drawn:

- The piles behaved elastically up until a load of 60 kN load or up to a pile head displacement of 30 mm, and beyond this point there was a significant reduction in stiffness.
- A lack of evidence of cracking at the interface between the pile cap and pile meant there was enough development length to transfer the forces between the pile cap and the pile.
- There was no loss in strength observed even when the piles were pushed to large lateral displacements.
- In spite of having a low transverse reinforcement these piles with plain round bars demonstrated a larger than expected deformation capacity.

6 ACKNOWLEDGEMENTS

Authors would like to acknowledge the financial support of the New Zealand Transport Agency, Earthquake Commission and QuakeCoRE, the New Zealand Centre of Research Excellence in Earthquake Resilience. We acknowledge the on-site logistical support of Fletchers construction, Jurgens Demolition and John Wood for his role in facilitating the start of this project. This is QuakeCoRE publication number 0652.

7 REFERENCES

Giovinazzi, S., Wilson, T. M., Davis, C., Bristow, D., Gallagher, M., Schofield, A., Villemure, M., Eidinger, J., and Tang, A., 2011. Lifelines performance and management following the 22 February 2011 Christchurch earthquake, New Zealand: Highlights of resilience, *Bulletin of the New Zealand Society of Earthquake Engineering* 44, 404–419

Hogan, L. S., Wotherspoon, L. M. and Ingham, J. M., 2013. Development of New Zealand seismic bridge standards, *Bulletin of the New Zealand Society for Earthquake Engineering*.

Lew, S. W., Wotherspoon, L., Hogan, L., Al-Ani, M., Chigullapally, P., & Sadashiva, V., 2020. Assessment of the historic seismic performance of the New Zealand highway bridge stock. *Structure and Infrastructure Engineering*, 1-13.

Palermo, A., Le Heux, M., Bruneau, M., Anagnostopoulou, M., Wotherspoon, L., & Hogan, L., 2010. Preliminary findings on performance of bridges in the 2010 Darfield earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, *43*(4), 412-420.

Stringer, M.E., Bastin, S., McGann, C.R., Cappellaro, C., El Kortbawi, M., McMahon, R., Wotherspoon, L.M., Green, R.A., Aricheta, J., Davis, R., McGlynn, L., Hargraves, S., van Ballegooy, S., Cubrinovski, M., Bradley, B.A., Bellagamba, X., Foster, K., Lai, C., Ashfield, D., Baki, A., Zekkos ,A., Lee, R., & Ntritsos, N., 2017. Geotechnical aspects of the 2016 Kaikoura earthquake on the South Island of New Zealand. *Bulletin of the New Zealand Society for Earthquake Engineering*, 50(2): 117-141.