Full scale snapback testing of reinforced concrete bridge piers

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ABSTRACT: An outline of a field testing program to characterise the dynamic response of a number of full scale reinforced concrete bridge piles is presented. The two piles were part of a pier from a recently demolished highway bridge on State Highway 16 in Auckland, New Zealand and extended 6.5 m above ground. The aim of this research was to quantify the non-linear stiffness and damping behaviour of each soil-foundation system as there is currently a lack of in situ test data of this nature. A monotonic load was applied to characterise the force-displacement response of each pile, using load increments up to 150 kN. At the end of each monotonic loading step, a snapback test was performed to allow for free vibration of the pile and to assess the damping characteristics of the soil-foundation system. The test piles were found to have limited degradation of their initial stiffness, however secant stiffness was reduced by up 25% for tests performed to the same load levels before and after the maximum pile loading. Equivalent viscous damping ratios from snap back testing ranged from 4-15%, with a damping values increasing as snapbacks were released from higher initial loads.

1 INTRODUCTION

The importance of accounting for soil-structure interaction in the seismic response of bridges has been widely acknowledged for many years (Gazetas and Mylonakis 1998). One of the inherent difficulties in accounting for this behaviour, particularly in a design setting, is the lack of information about the expected nonlinear stiffness and damping behaviour of pile foundations, as few large scale experimental tests have been performed to date. An opportunity to conduct such testing on extended pile-shafts (referred to as piles in the remainder of the report) from the bridge piers arose with the deconstruction of the Henderson Creek Bridge No. 2. The characteristics of the test bridge, the testing methodology and the results from the lateral loading and free vibration snap back testing of two reinforced concrete piles from the bridge are presented herein, along with an interpretation of the force-displacement properties, the dynamic response, and damping characteristics of each pile.

2 HENDERSON CREEK BRIDGE NO. 2

Henderson Creek Bridge No. 2 was located on State Highway 16 (SH16) in Auckland as indicated in Figure 1. It was constructed in 1962 by the New Zealand Ministry of Works to provide a crossing over Henderson Creek for two lanes of traffic between Auckland and Kumeu. The creek is a tidal zone, with maximum fluctuations in water level of just over 2 m. Henderson Creek Bridge No. 2 was deconstructed as part of the SH16 Lincoln Road Interchange Project which is being delivered by Fulton Hogan for the New Zealand Transport Agency (NZTA).

The bridge was a five span reinforced concrete structure, with the largest spans equal to 24.4 m. The overall length was 93 m, and the width was 9.35 m. The bridge deck (approx. 0.18 m thick) was cast in place to form an integral connection with four reinforced concrete girders at 2.24 m c/c. Each girder had an integral connection with 1.8 m x 1.09 m pier caps. The superstructure is supported at each pier by four 0.91 m diameter reinforced concrete piles and two reinforced concrete piles at each abutment. At the piers each pile is spaced at 2.24 m c/c. Reinforcement consists of 24 300 MPa 31.75 mm diameter longitudinal bars, and 300 MPa #6 spiral transverse reinforcement at 114 mm c/c over full length of the pile.

Each abutment approach is supported on fill overlying Tauranga group silts, clays and sands. At each pier location there is a layer of alluvium at the surface, the thickness of which reduces moving from the abutments towards the centre of the creek. Across the site there is thin layer of highly-to moderately-weathered East Coast Bays Formation up to 2 m thick below these surface layers. The bridge pile foundations are supported by moderately-to un-weathered East Coast Bays Formation, with raw SPT N >50 in this layer.

Figure 1. Test location a) in Auckland, b) close up of Henderson Creek Bridges No. 1 and 2.

3 TESTING OVERVIEW

3.1 Test Piles

Pile 1 and Pile 2 from Pier D shown in Figure 2 were tested. As a pile foundation for the new bridge was installed prior to testing, and there was less than a few diameters from the middle two piles, these were not tested. The top of the pile cap of Pier D was 6.55 m above ground level, based on measurements taken the day of testing.

Figure 2. Elevation of Pier D as viewed from Abutment F (Photo taken 20th May 2014).

Prior to testing, the bridge deck and beams were removed from the entire bridge. To isolate the test piles at Pier D from each other, the pier cap was sawn between the piles. The removal of the pier cap was part of the scheduled demolition works and left a clear gap of approximately 1.0 m between the top of each pile in the pier. The pier cap at Pier C, which was used as a reaction during testing as outlined in the next section, was not altered prior to testing, and all four piles remained connected.

Four boreholes and three cone penetrometer tests (CPTs) were located within 35 m of Pier D. These were used to characterise the soil profile at the test pile location. At the surface there is a 1 m thick layer of soft alluvium, with raw SPT N values of 2-11. Below this is a 1 m layer of highly- to moderately-weathered East Coast Bays Formation with CPT tip resistance (q_c) rising from 1 to 10 MPa through the layer. The lower stratum, and bearing layer for the piles is moderately to unweathered East Coast Bays Formation with raw SPT N >50.

3.2 Testing Methodology

A schematic of the testing procedure is shown in Figure 3. At the test piles in Pier D, a steel frame was hooked over and bolted into the pier cap above each pile creating a connection point for load to be applied. At Pier C, a similar steel frame was connected to the pier cap providing an anchorage point. Post tensioning strand was connected between the steel frames and the load was applied using a hydraulic jack located within the steel frame at Pier C. Testing was performed in two stages: 1) a "pull-back" phase where a static load was progressively applied to the top of the test pile and 2) a "snap-back" free-vibration phase after the load was released.

Connection to the top of Pier C and Pier D was made using steel loading frames, which were previously used in the bridge testing outlined in Wood & Phillips (1989). Figure 4 and Figure 5 show examples of how these steel loading frames were attached to each pile/pier at the Henderson Creek site. Two pre-stressing strands were attached between the steel loading frames which hook over the top of each pile/pier. The loading frames were bolted down using mechanical anchor bolts passing through a steel angle welded to the sides of the loading frames. Load was applied to the testing arrangement using a hydraulic jack located inside the steel frame hooked over Pier C (Figure 5). The hydraulic jack was engaged using either a manual pump or an electric pump. At Pier C, the ram pushed a sliding block between the steel loading frames in order to tension the pre-stressing strands. To allow for unrestrained movement during the snapback free vibration, some of the initial slack in the cables was taken up using the hydraulic ram.

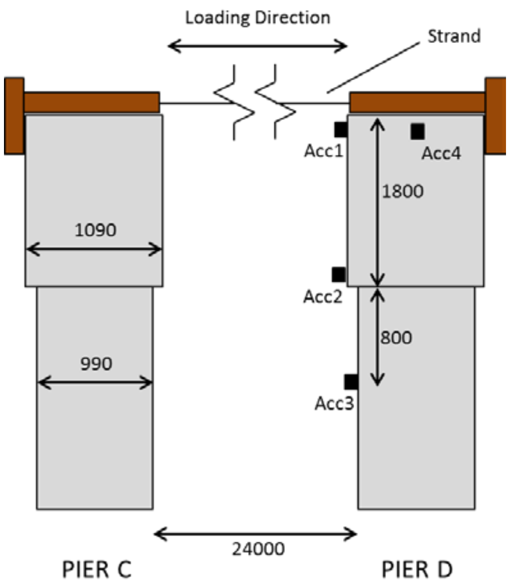


Figure 3. Schematic of test setup. Dimensions in mm, Acc = Accelerometer.

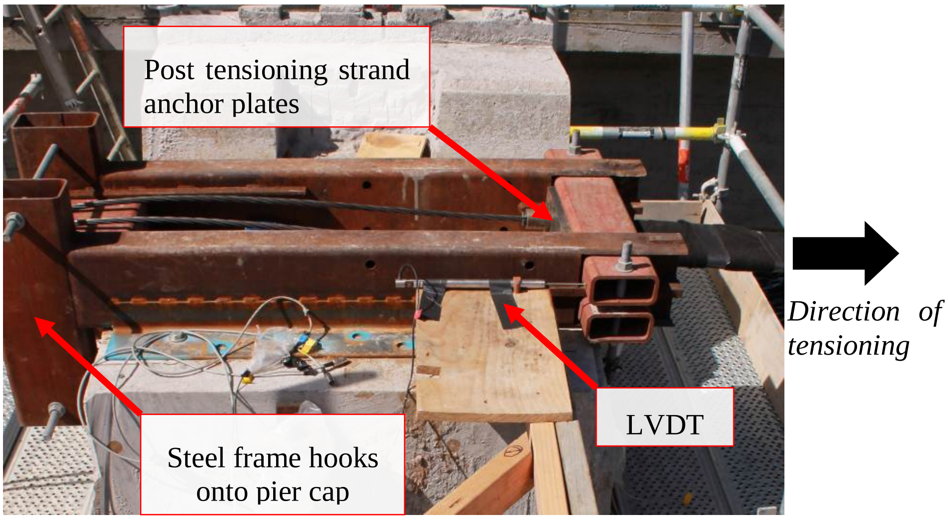


Figure 4. Steel loading frame on test pile at Pier D.

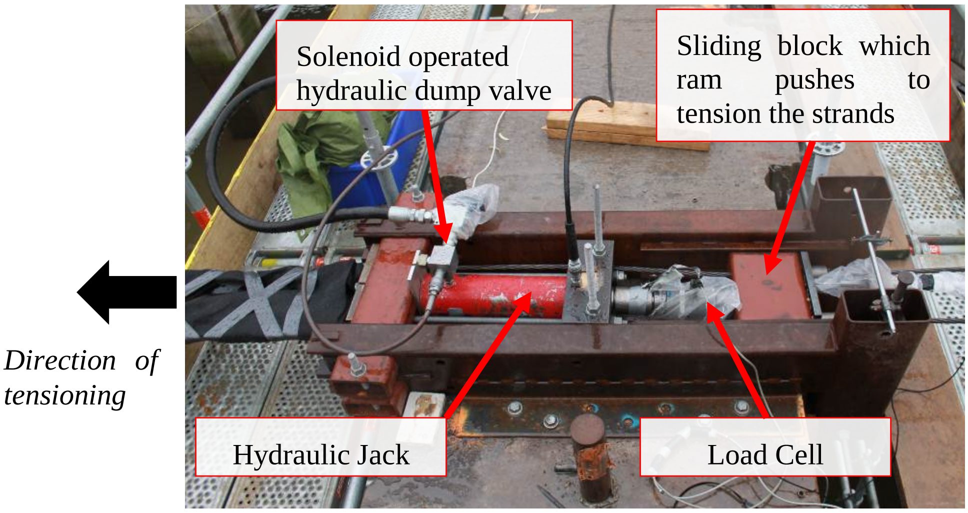


Figure 5. Steel frame with hydraulic jack and solenoid operated dump valve on reaction Pier C.

The load was released by allowing the hydraulic fluid to flow quickly out of the jack using a remotely triggered solenoid valve. The aim was to have a “near instantaneous” release of the load to allow free vibration of the pile. The hydraulic fluid was collected in a controlled manner to avoid contamination of the nearby stream. Testing of this release mechanism was successfully performed at the University of Auckland test hall facilities prior to the pile tests at Henderson Creek.

3.3 Instrumentation

A schematic of the instrumentation setup for testing is shown in Figure 3. A Linear Variable Displacement Transducer (LVDT) was positioned at the top of the pile to measure displacements during both the “pull-back” and “snap-back” phases of the test (Figure 4). It was not feasible to measure the displacement of the test piles at ground level as this was below the creek water level during testing. Accelerometers and tiltmeters were placed at three locations down the height of the test pile in the direction of loading as indicated in Figure 3. An additional accelerometer was also installed perpendicular to the loading direction to capture any out of plane movement. Additional accelerometers were planned to be installed further down the length of the column, however access constraints and water level heights meant this was not possible. The load applied during testing was measured using a 40 tonne load cell connected to the hydraulic jack in the loading frame at Pier C (Figure 5).

3.4 Test Sequence

Multiple pull-backs and snapbacks were performed at each test pile-column over a range of maximum load levels.

Table 1 summarises the loading sequence that was applied to each test pile. The initial aim was to increase loads to a maximum of 200 kN, however limitations with site access and the test setup meant that a reduced maximum load was obtained in each test sequence. Loads were progressively increased in the first stage of testing up to Test 5 to capture the change in the dynamic response during snap back. Loads were then reduced to investigate any changes in stiffness and damping of the system as a result of soil nonlinearity and pile cracking.

Table 1. Loading sequence for each pile with target and achieved load.

	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Test 8	Test 9
Target Load (kN)	50	50	100	150	200	50	150	100	50
Pile 1: Load (kN)	47	51	97	142	166	50	148	97	48
Pile 2: Load (kN)	52	52	98	149	148	48	103	101	50

4 FORCE-DISPLACEMENT BEHAVIOUR

The force-displacement behaviour for the entire test sequence of both piles is shown in Figure 6 and Figure 7. The force-displacement curves shown in Figure 6 and Figure 7 have been modified from the raw data collected during the tests to account for the loss of hydraulic fluid on some tests, which resulted in a mid-test slow loss of load. These areas have been removed from the data. Additionally, two of the tests for Pile 2 (Test 5 and Test 7) experienced crushing of the loading block which meant that the target load was not met. A portion of the force-displacement curves for these tests has also been removed after the point where loading block crushing initiated.

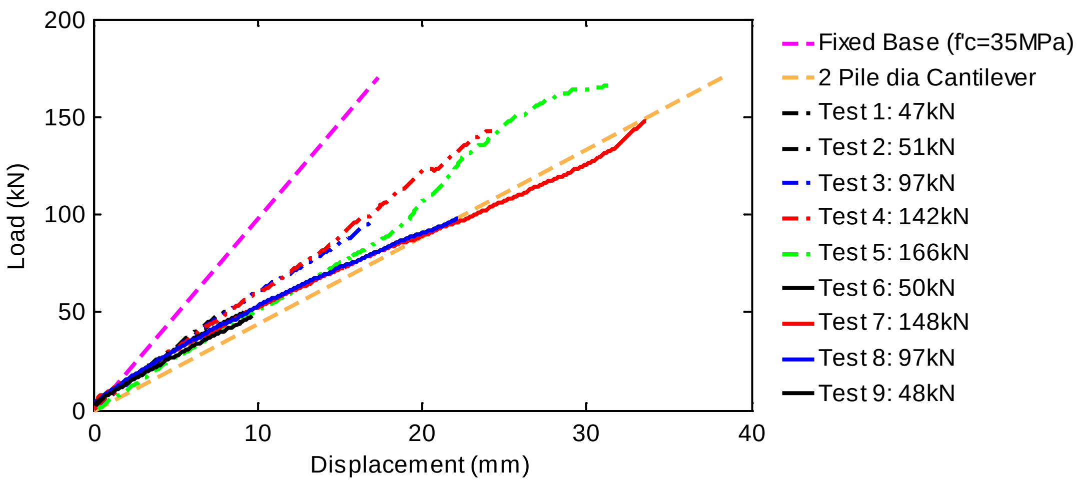


Figure 6. Force-Displacement behaviour of Pile 1.

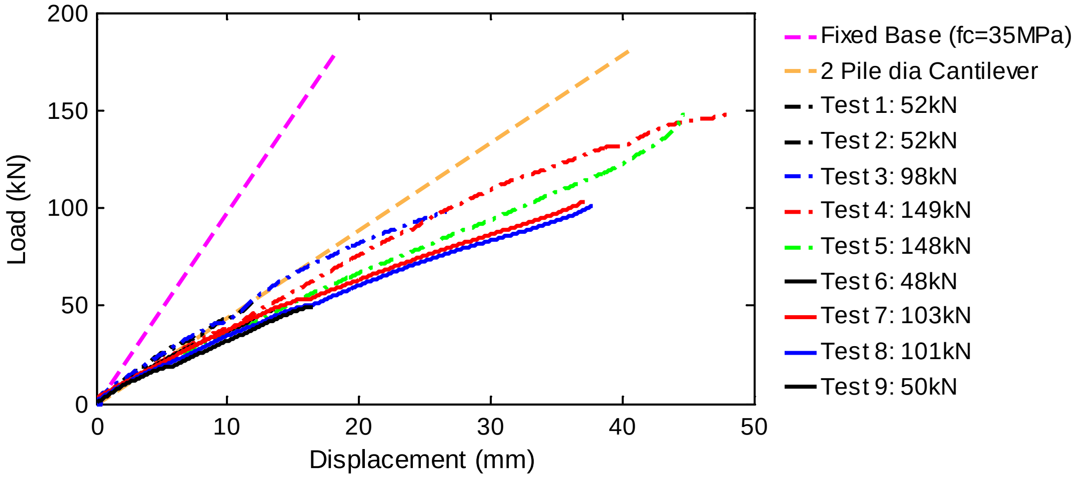


Figure 7. Force-Displacement behaviour of Pile 2.

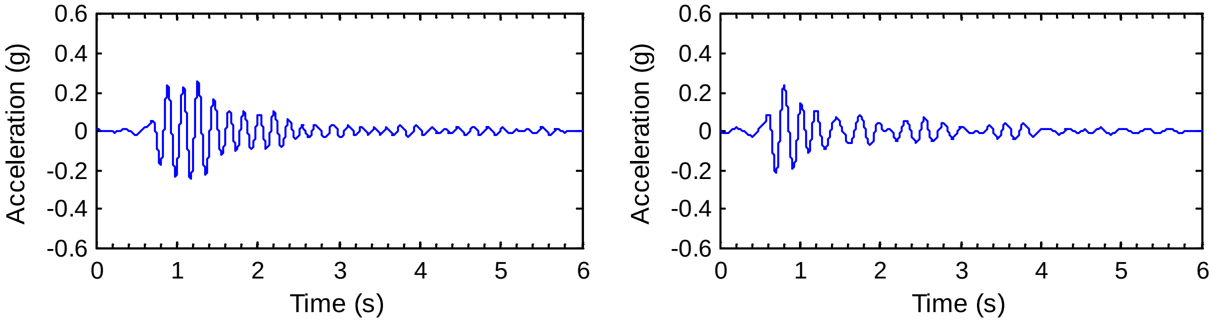
The loading sequence brought the piles to just below 1% drift and at load levels above the cracking moment of the pile section (17 kN-m) but below the initiation of yield in the longitudinal reinforcing, which was expected at a moment of 1630 kN-m (which would have required a 250 kN load). This level of loading is representative of the test piles at the serviceability limit state. The initial stiffness of the system did not change dramatically from one test to the next, which is most likely due to the stiffness of the piles dominating the response. All tests showed a reduction in the stiffness of the system with increasing load, with the snap-back testing most likely adding to these effects.

There was a clear reduction in the stiffness of the system after Test 4 in the loading sequence. The secant stiffnesses measured from zero to the maximum load for the two 100 kN tests (Test 3 and Test 8) dropped from 5.9 to 4.4 kN/mm for Pile 1 and from 3.7 to 2.7 kN/mm for Pile 2, an approximately 25% decrease for both piles. This increased flexibility was likely a result of increased cracking in the concrete section, and the development of gaps around the pile due to soil compressive nonlinearity after the maximum load was reached in Test 4 and 5. Following Test 5 there is again no significant change in the response between each successive test, as the load applied is less than the previous maximum.

In addition to the response of the piles, the stiffness of two idealised systems is shown in Figure 6 and Figure 7. Both systems represent the horizontal stiffness of the piles assuming a lower bound for the 28 day concrete compressive strength of 35 MPa, 0.75 I_g of the section, and a fixed base. The first idealisation assumes the fixed base at ground level, while the second idealisation assumes the fixed base is located two pile diameters below grade, roughly the location of the top of the moderately weathered East Coast Bays Formation. As expected, the ground level fixed base idealisation significantly overestimates the stiffness of the piles, however, the location of the fixed base two pile diameters below grade provides a good approximation of the stiffness observed in both piles. This idealisation likely approximates the pile behaviour well because the weathered rock is expected to provide significant fixity to the pile, particularly at a depth of two metres where pile displacements and rotations are expected to be small at this level of loading, and because of soft nature of the overlaying alluvium.

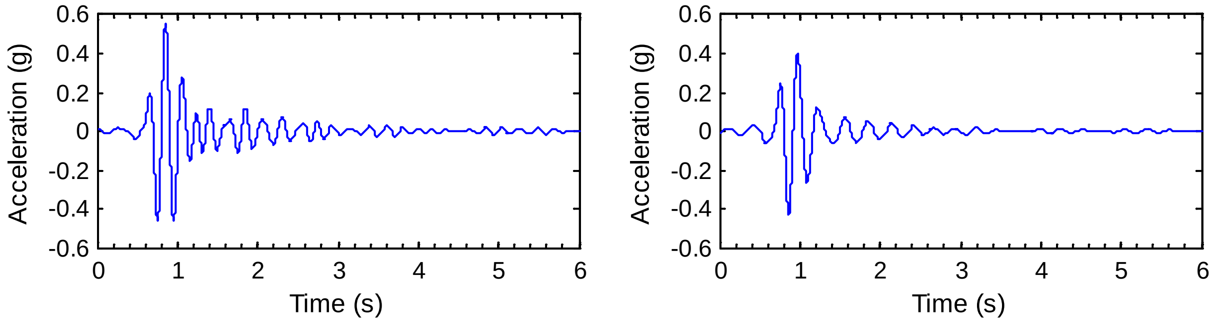
5 FREE VIBRATION BEHAVIOUR

The free vibration response from selected snapback tests is presented in Figure 8. Snapbacks with release loads of approximately 50 kN were found to have a slow amplitude decay while snapbacks with release loads above 100 kN exhibited a large initial amplitude reduction and then a slow low amplitude decay similar to the lower release load tests. There is also a slight increase in natural period between the initial high decay portion of the free vibration response and the low amplitude decay. The initial amplitude reduction and period shifts can be attributed to the nonlinear response of the soil-pile system dissipating energy through compressive nonlinearity of the soil and the creation of gaps on either side of the pile. Following this initial response, the pile responds in an elastic manner as a lightly damped cantilever and oscillates at similar amplitudes to that of the 50 kN snapbacks.



(a) Snapback Pile 1: 47 kN Load

(b) Snapback Pile 2: 52 kN Load



(a) Snapback Pile 1: 166 kN Load

(b) Snapback Pile 2: 148 kN Load

Figure 8. Selected free vibration responses from Pile 1 and Pile 2 snapbacks.

Two system identification techniques were used to characterise the entire free vibration response and to determine the indicative natural frequency and equivalent viscous damping of each snapback test. These techniques were the Eigen Realisation Algorithm (ERA) (Juang and Pappa 1985) and Stochastic Subspace Identification (SSI) (van Overschee and De Moor 1996). The natural period and damping ratios identified by these techniques are shown in Figure 9 and Figure 10, respectively. The natural period of Pile 1 ranges from 0.19 to 0.21 s as the load increases. Pile 2 had a significantly larger variation of natural period from 0.2 to 0.27 s. The large natural period variation in Pile 2 occurs after in the tests following Test 4 and likely results from the crushing of the loading block, affecting the clean release of the load during snapback. The lack of a clean release suggests that there was an interaction between the test pile, the loading rig, and the reaction pier during the initial portion of the free vibration response. Acknowledging this issue with the snapback data, the natural periods and damping ratios for Pile 2 have only been included for Tests 1-4 in Figure 9 and Figure 10. The damping of Pile 1 exhibits a clear trend of 3-4% at 50 kN load increasing to 10% above 150 kN. Similar to the natural period of Pile 2, the damping ratios identified for Pile 2 are highly variable, but there is a trend of 10% to 15% equivalent viscous damping ratios for Tests 1-4. Similar loading levels exhibit similar damping ratios and appear to be relatively unaffected by the loading history of the soil-pile system.

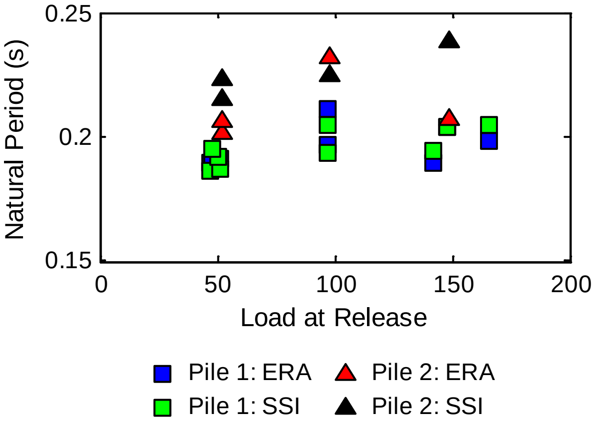


Figure 9. Natural period of test piles with respect to release load.

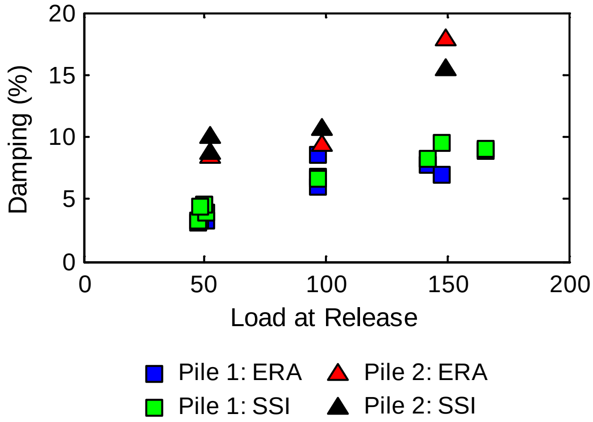


Figure 10. Equivalent viscous damping of test piles with respect to release load.

6 CONCLUSIONS

Monotonic loading and snapback testing was performed on two concrete piles of the decommissioned Henderson Creek Bridge No. 2 to determine the stiffness and damping of the soil-pile system. Piles were tested to the serviceability limit state under monotonic loading. The piles exhibited only slight degradation in initial stiffness for any given test but had approximately a 25% reduction in secant stiffness between tests performed to the same load levels before and after maximum loading. By assuming the softer deposits over the East Coast Bays Formation provided no stiffness to the system, the soil-pile stiffness could be approximated with an equivalent cantilever two pile diameters longer than the test piles. Free vibration testing showed that there was an increase in equivalent viscous damping between the minimum and maximum levels of loading, with damping ratios ranging from 4-10% for the 50 kN peak load to 10-15% for the 150 kN peak loads. It is suspected that this trend would increase at higher load levels, but plateau once the soil-pile stiffness levels off as both materials yield.

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