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CYCLIC TESTING OF A FULL-SIZE STABILIZED, FLAX-FIBRE REINFORCED EARTH (UKU) WALL SYSTEM WITH OPENINGS

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ABSTRACT :

Uku is a low-cost earthen construction method that has been developed to address the inadequate housing situation in rural Māori communities resulting from legal, financial and physical obstacles. Rural Māori communities are often isolated and lack affordable access to specialist equipment, technical expertise, skilled labour and building materials. In addition to this, many rural communities are located in earthquake prone regions. A cyclic load test was conducted on a full-size 5.5 metre long wall section based on a wall line, built in an existing Uku test house, incorporating a window and door opening. The current seismic design proposed for the Uku housing system uses conservative assumptions due to the lack of data pertaining to the seismic performance of the wall system. The wall test was conducted in order to ascertain the methods of failure, areas of structural seismic weakness, interactions between wall panels, and the structural performance of the wall system. The results of the tests will be used to update the existing design methodology for Uku houses. The cyclic wall test has proven that the Uku wall system has a non-linear capacity but shown that the wall exhibits non-ductile modes of failure within each individual earth panel. The test has provided a foundation on which to develop an efficient, seismically safe design methodology for future Uku housing developments in New Zealand.

KEYWORDS: Earthen structures, rammed earth, Uku, low cost housing, Indigenous engineering

1. INTRODUCTION

Māori are the indigenous people of New Zealand and constitute 15% of New Zealand's total population (Statistics NZ 2006). Through the Treaty of Waitangi signed in 1840, Māori have retained and regained portions of their ancestral lands belonging to the local iwi (tribes). This land is 'Māori land' and is the land that rural Māori are entitled to use to develop and provide for their families. Due to a combination of legal and financial obstacles, and social developments, many rural Māori live in overcrowded and substandard housing conditions (Housing New Zealand 2005). The issues experienced by rural Māori communities have been elaborated on in Cheah et al. (2008). In 2003 a research grant was awarded to develop Uku into a low-cost sustainable earthen housing system. Uku uses predominantly locally sourced materials during construction, unskilled labour from the local community (including the house owner) and incorporates many design features to reduce the negative environmental impact of house construction and occupation (e.g. modular design, passive solar design, efficient fixtures, double glazed openings). By using local low-cost materials and labour, in conjunction with a sweat-equity financing model (the owner/s physical contribution during construction of the home is valued and used as part of the down payment for the house), Uku provides an affordable and practical housing solution for rural Māori communities. In 2008 the research concluded with the construction of a 90 m^2 Uku house on the foreshore of Lake Rotoiti, New Zealand.

The four year period to develop the concept into a viable housing solution and implement the house in a rural Māori community did not provide enough time to test and evaluate the seismic capacity of the Uku building system. The preliminary focus was to determine the appropriateness of Uku for the target Māori community end-user in terms of economics, constructability and aesthetics (Morgan 2005). As a result, conservative seismic design parameters were used to determine the structural capacity of the design for the first Uku house, shown in Figure 1. The calculations submitted to and consented by the Rotorua District Council used a Structural Ductility Factor (μ) of 1.0 and a Structural Performance Factor (S_P) of 0.9. The fundamental goal of the cyclic wall test was to evaluate the actual seismic performance of the Uku wall system and the related mechanisms of failure within the wall panels, and around door and window openings. The results will be used to improve the Uku design procedure for future implementations of Uku in other rural Māori communities.

Figure 1 Photo of the Uku House in July 2008

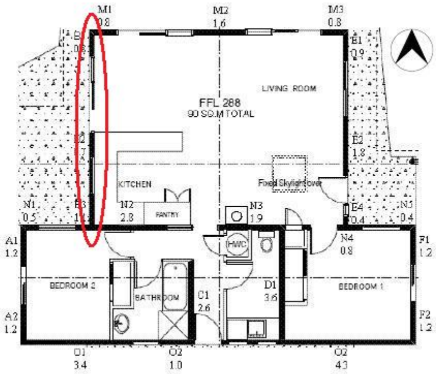


Figure 2 Floor Plan of the Uku House

Material tests conducted in 2007 established the compressive, flexural and shear strengths of the Uku material and are documented in Cheah and da Silva (2007). The test results are summarized below in Table 1. The high variability in the results was partly due to the variable thickness of the rammed-earth layers. In future, horizontal markers will be placed on the formwork to ensure a higher degree of material consistency and thinner rammed earth layers.

Table 1 Summary of Uku Material Test Results

	Average Strength (MPa)	Coefficient of Variance	Design Strength (MPa)
Compression	7.60	23.7%	4.60
Flexural	0.35	28.9%	0.18
Shear	0.73	17.0%	0.52

1.1. The first Uku house

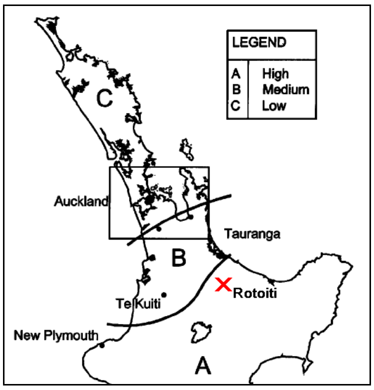


Figure 3 Earthquake Zones of the Upper North Island (Standards NZ 1999)

One of the key objectives of the research was to demonstrate the practicality of the earth housing concept by using local unskilled labour and local materials to build a full-scale, two bedroom house (90 m^2) on rural Māori land. The building site selected for the Uku house was located near the Southern foreshore of Lake Rotoiti at Haumingi 10a2b Papakainga belonging to the hapū Ngati Te Rangiunuora. The house layout is shown in Figure 2. The region is classified Zone A, the highest seismic zone in New Zealand (shown in Figure 3).

A reinforced concrete foundation slab and an exposed rafter timber diaphragm roof were used in the Uku house. A non-specific design guide does not exist for the Uku building system. Therefore *NZS 4229:1999 Concrete Masonry Buildings Not Requiring Specific Engineering Design* (Standards NZ 1999) was used as a guide for the Uku building analysis.

2. CONSTRUCTION, INSTRUMENTATION AND BRACING OF THE TEST WALL

The seismic capacity of the Uku wall system was evaluated by testing a full-size wall. The test wall was based on wall line B in the existing Uku house, and provided a direct comparison between the existing house design and the results of the seismic test. The location of wall line B is outlined in Figure 2 and the wall is shown below in Figure 4.

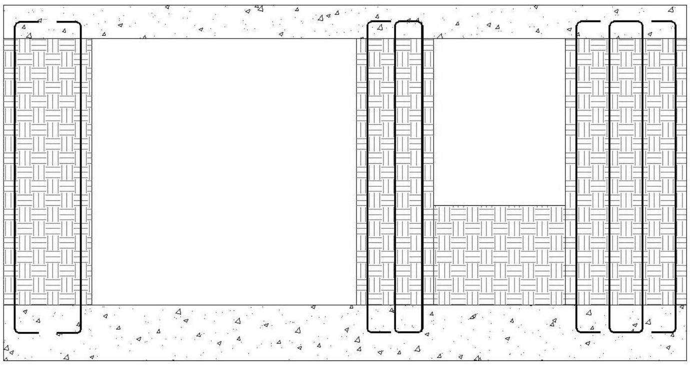


Figure 4 Reinforcing detail of wall line B

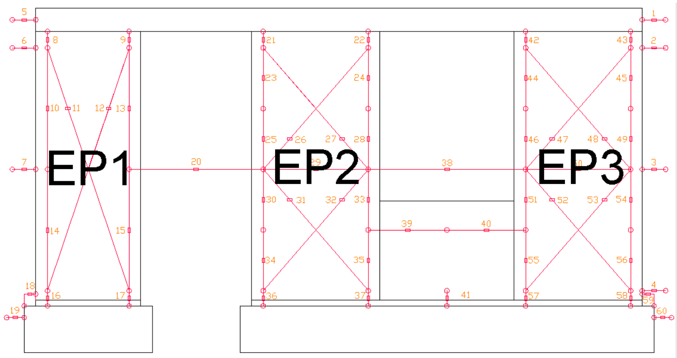


Figure 5 Uku test wall labelling and instrumentation

Wall line B is 6.2 m long in total and features a 2.4 m ranch slider, a 1.2 m window opening extending to the top of the wall and three 150 mm thick earthen wall panels (with a full height length 0.8, 0.7 and 1.1 metres respectively). The earthen test wall panels had lengths of 0.9, 1.1 and 1.1 metres respectively due to laboratory construction constraints. The earth panels were built on top of a 50 mm high nib wall laid on the concrete slab foundation to provide a shear key between the concrete slab foundation and the earth wall, and to establish the width of the wall at its base. The earth panels were connected via a concrete bond beam, 150 mm X 200 mm in cross section, cast insitu on top of the earthen panels. Permanent macrocarpa formwork is a part of the bond beam finish in the Uku house and was used for the test wall. Seismic capacity was provided by placing continuous vertical D12 steel reinforcing bars through the earth panels, from the concrete foundation to the bond beam. The 2.4 metre door opening was reduced to 1 metre due to space constraints, and the ultimate test wall load capacity was recalculated to account for the increased wall lengths. The earth panels were referred to as EP1, EP2 and EP3 respectively as shown in Figure 5.

The construction staff involved with the Rotoiti Uku house construction travelled to Auckland on the 9th of June, 2008 and built the rammed earth test wall in the same sequence as was built in the Uku house. One panel was built each day, commencing with EP2 on the 10th of June, EP1 on the 11th and EP3 on the 12th. The concrete bond beam was cast insitu on the earthen wall panels on the following day (13th June) within permanent macrocarpa formwork. Casting the bond beam less than 24 hours after EP3 was rammed was recognized, after the wall test was conducted, as the likely cause of the sliding/de-bonding failure of EP3. See section 4.3.2.

The wall was left to cure for a 28 day period, during which the wall was instrumented with 61 displacement gauges. Video surveillance broadcast the test live and streamed data over the Network for Earthquake Engineering (NEES).

Two lateral struts were attached to the concrete bond beam; one at the loading jack and one between EP1 and EP2 above the doorway. Two vertical struts were placed at both ends of the wall and were hand tightened against the top of the bond beam. The vertical struts simulated the vertical restraint provided by perpendicular earth wall lines at both ends of wall line B in the Uku house. Teflon sliding plates were used on the interface between the vertical restraints and the bond beam to prevent the vertical restraints from interfering with wall deformation along the axis of loading.

3. PRE-TEST WALL ANALYSIS

A torsional seismic analysis was completed on the Uku house in order to determine its required seismic strength. The analysis established the demand on each wall line. A lateral load of 60 kN was determined for the test wall as the load that would satisfy the bracing requirements of the Uku house design. The torsional analysis took into account the length of wall panels, their distance from the centre of mass and the accidental eccentricity of the house, and neglected the strength of wall panels less than 0.7 m in length and wall panels located under window openings.

The Uku wall system was analyzed using SAP - assuming that the walls would fail in flexure. This was reasonable based upon the flexural and shear strength tests conducted in 2007 that indicated that the unreinforced rammed earth material had high shear strength and low flexural strength. The Uku wall system was modeled as a frame with columns at the centre lines of each wall panel, and a rigid beam along the centerline of the bond beam. The effective height of each column was determined as the height of the opening on the panel wall farthest away from the direction of seismic action.

Due to the asymmetrical geometry of wall line B, two bracing line models were created for loading in both directions as shown below in Figure 6. These models were used to determine the required vertical reinforcement in each wall panel to meet the seismic demands according to *NZS 4229:1999 Concrete Masonry Buildings Not Requiring Specific Engineering Design*. The predicted wall failure mechanism was a mixed flexure-shear mode. Flexural cracks were expected to form initially and progressively curve as shear failure began to dominate response.

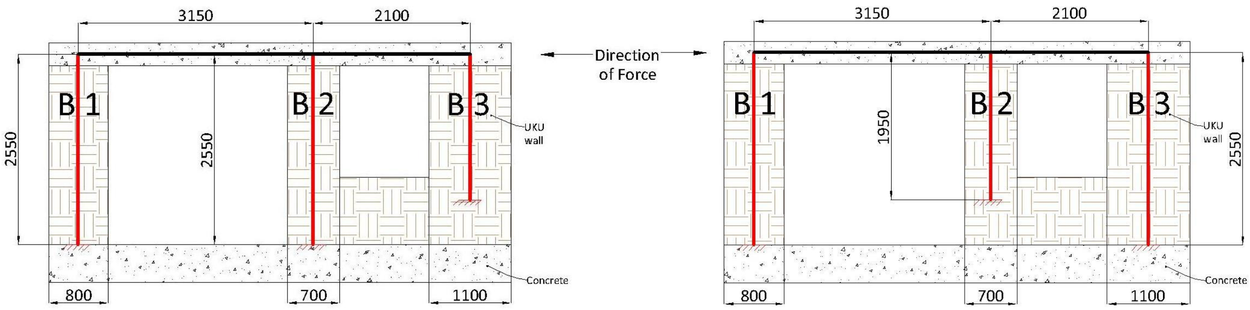


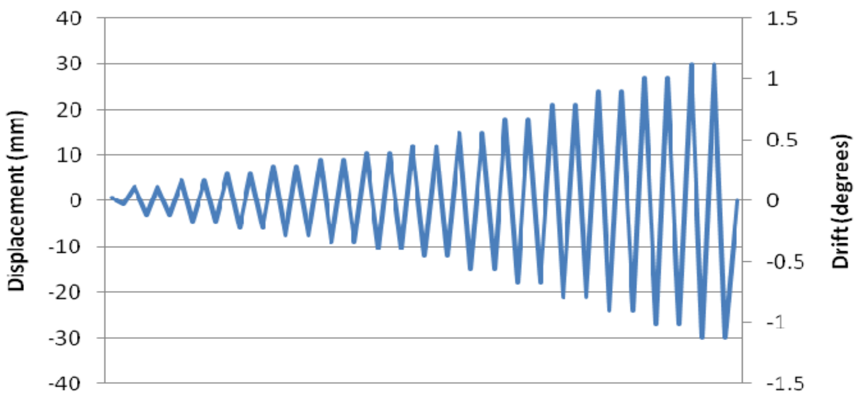
Figure 6 SAP Models of wall line B in both directions

4. QUASI-STATIC UKU WALL TEST

On Friday 11th July, after 28 days had elapsed, the Uku wall was subjected to a quasi-static test. The wall exhibited considerable non-linear strength, although each of the three panels individually failed through non-ductile shear and slip mechanisms. The maximum force sustained by the test wall was 53 kN when the load cell was pushing and 59 kN when pulling. The maximum loads attained were less than the calculated ultimate lateral loads, in both directions, of 62.8 kN when pushing (84%) and 61.0 kN when pulling (97%).

4.1. Load history

The imposed deformation history was based on the interstory drift of the test wall. At a wall height of 2.4 m, 1% of interstorey drift was equivalent to a displacement of 24 mm. A drift-based load control was used because the yield displacement of the wall was difficult to evaluate. Due to the expected low ductility of the earth panels, it was deemed beneficial to have many cycles at lower displacements, rather than to load the wall at particular levels of structural ductility. The wall was cycled twice per magnitude in accordance with the Park (1989) loading history, which is a common repetition for cyclic tests conducted in New Zealand. The cycles began with one cycle at 1/32 of a degree (0.75 mm) and increased gradually to a drift of 1.25% (30 mm). The loading cycles are shown in Figure 7.



Cycles	Drift	Displacement (mm)
1	0.031%	0.75
2	0.125%	3.0
2	0.188%	4.5
2	0.250%	6.0
2	0.313%	7.5
2	0.375%	9.0
2	0.438%	10.5
2	0.500%	12.0
2	0.625%	15.0
2	0.750%	18.0
2	0.875%	21.0
2	1.000%	24.0
2	1.125%	27.0
2	1.250%	30.0

Figure 7 Load history of the Uku wall test

4.2. Results and observations

4.2.1 Lateral-Force Displacement History

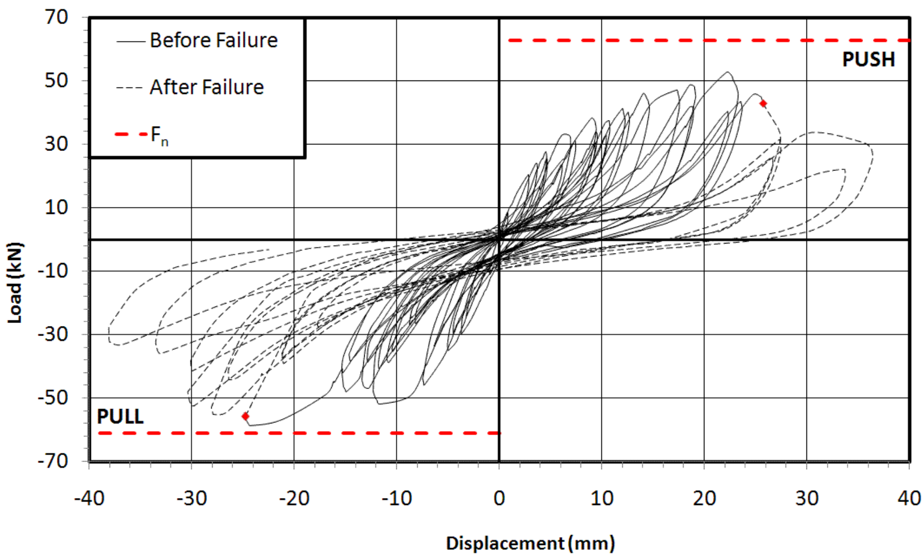


Figure 8 Lateral Force-Displacement History of the Uku Wall Assemblage

The lateral-force displacement history is plotted in Figure 8. The shape of the hysteresis loops before wall failure are similar to the hysteresis loops obtained from seismic tests of plywood sheathed timber shear walls (controlled by slip of sheathing nails) (Dean et al. 1986). The horizontal dotted lines indicate the calculated ultimate lateral load F_n. Due to the lack of symmetry in the wall line, the seismic performance in each direction was different. The maximum lateral forces sustained and the maximum available displacement of the wall is shown in Table 2 for both directions. The measured non-linear characteristics of the wall system were not due to the yielding of the vertical steel reinforcing bars but rather the sliding, rocking and crushing of the earthen panels. This is consistent with the wall exhibiting non-linear behaviour below the calculated ultimate lateral load. Due to the lack of steel yielding in this wall test the overall wall ductility could not be accurately and confidently evaluated. The results do show however, that the initial assumption of an elastic structure with a structural ductility of $\mu = 1$ was conservative.

Table 2 Summary of the wall test results

Direction	Maximum Displacement (mm)	Maximum Lateral Load (kN)
Push (+ve)	25.7	52.8
Pull (-ve)	-24.8	-58.7

4.2.2 Wall Failure Description

The wall system exhibited several failure mechanisms throughout the test. At a lateral displacement of 6 mm a fine crack formed at the corner of the window and at the control joint between EP2 and EP3. The nib wall on which EP1 was built had detached from the concrete foundation beam and rocked with EP1, with measured vertical displacements up to 2 mm. The top half of EP2 failed first with a 6 mm wide diagonal shear crack developing at a lateral displacement of -7 mm. A large shear crack developed on the opposing diagonal as the load was reversed. At a lateral displacement of 11 mm it was possible to see through the cracks in EP2. Up until a displacement of 21 mm EP1 showed minimal damage and was observably rocking on the concrete foundation beam. A sliding shear mechanism had developed between the concrete bond beam and the top of EP3 from an early stage in the test. EP2 continued to develop new shear and flexural cracks, and widened existing cracks. At a lateral displacement of 21 mm EP1 developed a large diagonal shear crack; by this point EP2 was severely cracked. Small shear cracks developed in EP3 as lateral displacements increased past 30 mm. Figure 9 shows photos of EP1, EP2 and EP3 at the end of the wall test.


Figure 9a EP1 at test end


Figure 9b EP2 at test end


Figure 9c EP3 at test end

4.2.2 Material Tests

Compression and flexural strength tests were conducted on samples extracted from the un-cracked wall panel between EP2 and EP3 and were guided by *ASTM D1633-00 Standard Test Methods for Compressive Strength of Moulded Soil-Cement Cylinders* and *ASTM D1635-00 Standard Test Method for Flexural Strength of Soil-Cement Using Simple Beam in Third Span Loading*. The results are presented below in Table 2 and are discussed in Section 4.3.5.

Table 3 Material test results of samples extracted from the failed wall

Test	Samples tested	Age (d)	Average Strength (MPa)	Lowest value (MPa)	Highest value (MPa)
Compression	5	35	3.70	1.61	6.98
Flexure	6	35	0.21	0.10	0.39

4.3. Discussion

4.3.1 The effect of the flax-fibres

The fibres held the failed earth panels together even after large cracks had developed. Flexural tests that were conducted on 600 mm beams extracted from un-cracked areas of the earth wall revealed a post crack (only fibres acting) flexural strength of approximately 28 kPa which reduced in strength exponentially as displacements increased.

4.3.2 Non-ductile failure of the wall panels

All three wall panels exhibited non-ductile failure mechanisms. EP1 rocked initially and failed in shear, EP2 failed in shear, and the top layer of EP3 de-bonded from the 4 D12 vertical steel reinforcing bars connecting the earthen panel to the bond beam. Yielding of the vertical steel reinforcing at the base of the panels was the ideal failure scenario because this would create a ductile failure mechanism. Clearly this was not the mode of failure observed in the wall test. The sliding shear mechanism that developed between EP3 and the bond beam was unexpected. Post-test observations of EP3 revealed a weaker layer of earth at the top of the panel. It was also noted that before the test the top earth layer of EP3 was paler than the rest of the wall panel. A likely explanation is that the top layer of EP3, which had been curing for less than 24 hours, absorbed addition water as the concrete bond beam was cast insitu on the earth panels. The same problem is not expected to occur in the Rotoiti Uku house because there was a considerable length of time between the construction of the wall panels and the insitu casting of the concrete bond beam. EP3 represented 37% of the total effective wall length and was likely to have had an under-represented strength contribution in the test conducted.

4.3.3 Non-linear strength of the wall system

Although all three wall-panels failed in a non-ductile manner, the overall wall system showed an appreciable level of non-linear capacity. This may be partly due to the wall panels failing at different times but also suggests that the failed wall panels retain an appreciable level of strength post-failure. Due to the difficultly in assessing the loads carried by each individual panel, the source or mechanism of non-linear strength cannot be more accurately identified.

4.3.4 Effectiveness of the vertical steel reinforcement

The D12 vertical reinforcing steel bars in each earthen panel did not yield but kept the cracked panels in-plane and provided the overall system, in combination with the flax-fibre reinforced rammed-earth and the bond beam, with a measure of non-linear strength. The steel reinforcement provided a strong and reliable connection between the concrete foundation beam, the earthen panels and the concrete bond beam.

4.3.5 Lower material test results

In 2007 the average compressive strength and flexural strength (after 117 days) measured from dry-cut Uku samples were 7.6 MPa and 0.35 MPa respectively. Post-test compressive and flexural tests extracted from an un-cracked portion of the test wall revealed significantly lower average strengths. The samples were wet-cut from the test wall and cut to shape using a diamond-tipped chainsaw due to practical reasons. The results of 35 day compression and flexural tests averaged at 3.7 MPa and 0.21 MPa respectively. The reduction in material strengths were attributed primarily to the lower curing time. Early indications show a slower strength gain over time in the Uku material. Wet extraction of the samples is likely to have also reduced the material strengths measured.

5. FUTURE RESEARCH

As a result of this wall test, two single wall panels are being built to evaluate the effect of providing nominal shear reinforcement. A revision of the existing SAP model or a new analytical model will be created now that the failure mechanisms of Uku wall systems are better understood and further strength gain over time tests are planned.

6. CONCLUSION

The wall test did not reach the calculated ultimate lateral loads required for an elastic design. The wall reached 84% in the positive direction (push) and 97% in negative direction (pull). The de-bonding failure at the top of EP3 is likely to have resulted from casting the insitu concrete beam too soon after ramming the earth panel. The strength contribution of EP3 would have decreased as a result and under-represented the actual strength capacity of the earth panell. All three wall panels exhibited non-ductile shear and de-bonding failure mechanisms which were contrary to assumptions that the panels would fail first in flexure.

The wall exhibited a significant non-linear capacity and the results indicated a level of conservatism in the assumption that the Uku wall system behaved as an elastic structure ($\mu = 1$). The non-linear capacity of the wall was mobilized below the calculated ultimate lateral load. This is consistent with the vertical steel reinforcement not yielding first but rather the earthen panels shearing, sliding and rocking.

As the first seismic test on an Uku wall, the results and observations have been valuable in improving understanding regarding the wall performance when subjected to seismic loads. The test has proven and disproven various assumptions regarding the seismic performance of Uku walls, and has provided a foundation on which to conduct further wall tests.

7. ACKNOWLEDGEMENTS

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