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Retrofit Options for Residential House Foundations to Resist Earthquakes

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ABSTRACT: Previous work by Irvine and Thomas has shown that seismic performance of timber framed houses in Wellington is often compromised by poor, no or degraded connections between piles and bearers or foundation walls and joists. The same problems were evident in a number of houses visited in Christchurch after the recent Canterbury earthquake, resulting in avoidable damage. Retro-fit hardware that is cheap simple, easy to install using nail guns or self drilling timber screws has been developed and tested at BRANZ. This hardware is easy to fit, even where space is limited restricting the use of hammers, but is ductile in both tension and compression. It is strong enough to develop the maximum passive earth pressure that can readily be achieved under lateral loads for ordinary piles resulting in significant improvement in seismic performance for poorly braced foundations at low cost.

1 INTRODUCTION

The majority of buildings in New Zealand are light timber frame domestic dwellings. These generally perform well in earthquakes, with their inherent flexibility, low mass with resultant low inertial forces and good bracing in the superstructure provided by internal gypsum plasterboard linings. A weakness is the lack of foundation bracing and/or connections between the bracing and piles, piles and bearers and foundation walls to bearers or joists. These weakness became apparent with damage from the moderate 1987, Edgecumbe earthquake (BRANZ, 2003). Many houses were built prior to the introduction of formal construction standards and have little or no foundation bracing. Inspections of damaged houses after the recent Canterbury Earthquake have shown flaws in foundations that have resulted in more significant damage, such as brick veneer cladding supported on a perimeter concrete wall and the timber framing supported on piles physically separated from the wall. A significant proportion of the houses inspected by the authors had no mechanical connection between concrete foundation walls and the framing above, and many ordinary piles had no mechanical connection to the bearers above. Houses on piled foundations built to the Light Timber Framed Construction Standard" NZS3604, introduced in 1978, appear to have performed very well.

Most damage to light timber frame houses from the Canterbury Earthquake was due to liquefaction and lateral spreading. Shaking damage was not widespread which can be attributed to the relatively low levels of shaking in Christchurch itself and the low population density closer to the epicentre. The type of foundations used in Christchurch, with mostly flat or gently sloping sites, are slab on ground or perimeter concrete walls resisting gravity and lateral loads with internal ordinary piles, which resist gravity loads only. These foundations are usually quite low with 600 mm or less of ground clearance below framing and have little difference in height across the plan of the building. This is in contrast to Wellington and other cities where fully piled foundations are more common. With a large proportion of sloping sites, heights of foundations may vary considerably across a building, causing significant torsional effects, resulting in higher displacements and/or loads on some foundation elements.

In 2006-7 the adequacy of Wellington timber dwellings' foundations, including the sub-floor bracing, sub-floor fixings and general condition of the foundations was assessed as a Master of Architecture thesis research project by James Irvine (Irvine 2007, Irvine and Thomas 2007). The adequacy of a sample of 80 foundations of dwellings was assessed against the current "Light Timber Framed

Construction Standard" NZS3604:1999 (SNZ 1999). All foundation types were assessed for bracing and fixing capacity, the general condition and compliance with NZS3604:1999. A specific analysis of the fixings showed serious inadequacies in over 70% of dwellings. The bracing adequacy of different foundation types was assessed including contributions from non-designed bracing such as ordinary piles and large concrete anchors such as steps and chimney bases. Observations illustrated that ordinary piles were the primary bracing mechanism for 16% of dwellings, and concrete anchors in 11% of cases. Most piled dwellings had inadequate bracing with around 80% having less than half the level of bracing required by NZS3604. Concrete foundation wall dwellings were generally adequate, despite commonly utilising heavier cladding materials. Overall, 39% of dwellings were under bracing requirements when non-designed bracing was excluded from calculations. Retrofit options were assessed and costed, and losses with and without upgrade were estimated. The cost of specific damage and collapse to residential dwellings was estimated to reduce from $3.81billion to $2.1 billion if retrofit options costing $300 million were completed, giving a benefit/cost ratio of 5.7.

Savings in indirect costs and reduced impacts for the maximum credible earthquake located on the Wellington fault are estimated and summarised in Thomas and Irvine (2008). The reduction in requirements for temporary and long term accommodation for evacuees from destroyed and damaged houses was estimated to be reduced from about 43,000 people to 16,000, and a reduction in fatalities from 120 to 24, although this is highly dependent on the timing of an earthquake. Including indirect effects and building cost inflation increased the benefit cost ratio of about 6 to 17. This analysis was rather crude and contains many assumptions, but does show that upgrading house foundations to resist earthquakes is of major benefit.

The desirability and practicality of upgrading residential house foundations can be further enhanced by developing methods or devices that can readily be fitted to existing houses. A disincentive to performing remedial work is cost. The cost is in two parts; the cost of installing any remedial measures, and the cost of assessing and designing remedial measures for a particular building. Putting more effort into assessment and design will often result in lower cost of remedial work, but these require a high level of expertise. To assess a building at a level higher than merely comparing details with current NZS3604 requirements requires the expertise of a professional engineer, or perhaps an engineering technologist with specialist training. Assessing the contribution to seismic performance of non-designed elements such as chimney bases and concrete steps, for example, requires both a detailed inspection of the connection details and a good understanding of how they might work. Upgrading to standard NZS3604 details is something a tradesman is quite capable of doing, but replacing ordinary piles with anchor piles, or piles suited for bracing is a difficult task under an existing floor, even with good clearance. Strengthening bearer to pile connections is of little use if the pile is an "ordinary pile" sitting on a concrete pad or embedded in a concrete pad that is only 200 mm deep. These details are easy to install prior to the floor being built, but swinging a hammer, while lying on your back with 300 mm clearance or less between the ground and a bearer is not so easy.

2 RETROFIT OPTIONS

A number of retro-fit options for different foundation types have been developed and are summarised by Cooney (1982). These involve strengthening connections of framing to foundation walls, or installing either in-situ concrete walls or plywood sheets nailed onto timber framing between piles. If foundation walls do not exist, then the first option is not possible. Casting in-situ concrete walls under existing framing can be difficult. A high level of skill and care is required to ensure that the concrete is in contact with the framing above the wall. Installing in-situ walls or timber framing and plywood is only practical under exterior walls, particularly with low ground clearance. Although NZS3604 requires foundation bracing lines at not more than 6.0 m centres, typical timber floor systems are likely to have sufficient strength and stiffness to span much greater distances, so bracing external walls only may suffice. For retrofit options to be viable they must be inexpensive, durable, easy to fix even in confined spaces, be tolerant of geometry and mis-aligned elements and hard to install poorly. In order to reduce the number of fixings it is desirable that connections to resist lateral loads work in tension and compression. It is also desirable that standard components be used as much as possible.

If existing foundations consist only of piles it is difficult to know what the capacity of each pile is to resist lateral load. It is relatively easy to provide a high capacity connection at the top of a pile, by using the NZS3604 standard 12 kN or 6 KN connections. The lateral load capacity of a pile is then determined by the capacity of the pile to resist overturning at its base as it acts as a cantilever. However, the pile capacity can be dramatically increased by creating a moment resisting connection at the top of the pile and then the failure mechanism, becomes sliding of the pile and footing in the soil rather than over-turning, which has a much lower capacity. The system then acts as a moment resisting frame rather than cantilever piles. The moment capacity required to develop the strength of the pile/soil connection in sliding depends on the height of the pile above ground, footing width, depth and soil strength. The pile length often varies across a building and to a lesser extent so may the soil strength. The embedment of piles often varies on sloping sites and it is not possible to ascertain the footing width and embedment depth on existing buildings without excavation. Two methods of providing connections were developed and tested; a plywood gusset connection and a steel pipe/nail plate connection.

2.1 *Plywood Gusset Connection*

A moment resisting connection was developed using a triangular plywood gusset nailed onto a bearer and pile. The test specimens used 500 mm by 500 mm by 17 mm thick ply fixed with 16 no. 2.87*50mm D-head gun nails over a 35 mm packer to allow for misalignment between the faces of the bearer and pile. The basic test set-up only permitted a test of shear capacity and time limitations precluded further testing. The test set up and the load displacement plot is shown in Figure 1. The specimen was cyclically loaded in 1 kN increments using the strong floor in the Structures laboratory at BRANZ.

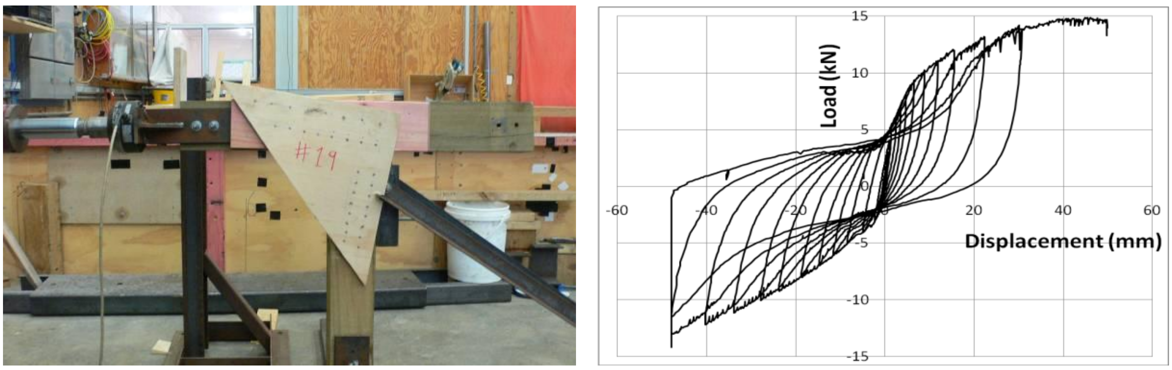

Figure 1. Plywood Gusset Test Specimen (left) and Load vs Displacement (Note positive force and displacement is towards the right in the photograph).

The results show a capacity of 13.5 kN in one direction and 15 kN in the other, similar to the 12 kN of the standard connection in NZS3604. The hysteresis loop shows degrading stiffness which is typical of timber connections, but the loops are “fat” showing a high level of ductility, due to nail-slip. The moment capacity of this joint limited by the close spacing of the rows of nails, in turn limited by the width of the pile and depth of the bearer, about 2.0 kNm. A larger gusset and nail group is required to develop the sliding strength of the piles in the ground as found in later tests.

2.2 *Steel pipe/nail plate connection*

Nail plates or straps are frequently used in timber connections as they have high tensile and shear strength, are easily fixed and can be bent to allow for misalignment of timber members in joints or to accommodate complex joint geometries. These straps can be fixed with nails or screws, but are not suitable for nail guns. In nail plate joints compression strength is provided by bearing of timber elements against each other as the nail plates are thin and buckle under low compression loads.

The easiest way to create a moment resisting connection at the top of a pile is to use a brace, but this must have some strength in compression. The advantages of the nail straps have been utilised by running two nail straps through a hollow steel pipe, to provide compression capacity. The two straps can then be splayed apart at the end of the pipe allowing nailing or screwing to timber members in a range of orientations as shown in Figure 2.

Figure 2. Steel pipe/nail plate connection, connected at 90°from pile to bearer (left) and at about 45° from pile to joists (centre) and close up of connection (right)

Specimens were mostly tested using 8 gauge screws as it is envisioned that these devices would mostly be used in confined subfloors where cordless drills were more practical than hand driven nails.

Initial tests were carried out in the “DARTEC” universal test machine at BRANZ. The model set-up is shown in Figure 3 (left), with the load displacement plot (right). This test specimen consists of a 33.7*4.0 steel tube and two 25*1.0mm Grade 300 Mitek straps, into a 125x125 pile and 140x90Bearer, with two 8 gauge screws in each strap end.

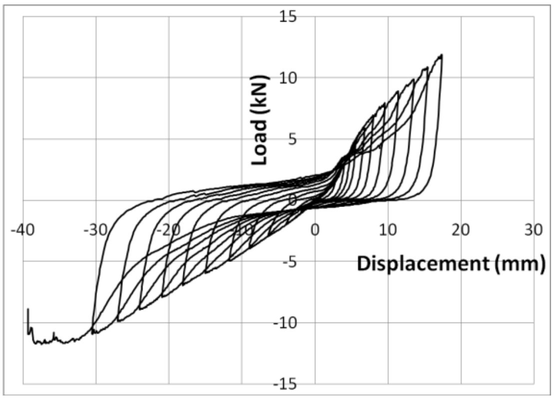

Figure 3. Dartec test specimen (left). and Load vs. Displacement, note positive force and displacement is when the joint is closing, and the specimen is in compression.

The capacity is about 12 kN in tension and compression compared to a pile to bearer joint lateral shear capacity of 8.5 kN and a moment capacity of 3 kNm. Larger moment capacities can be achieved by using a longer brace, but this was not possible due to limitations of the test set-up. The system is ductile in both tension and compression but is stiffer in compression, where the tube ends push on the folded out strap and timber behind. In tension, the straps bend around the screw heads and straighten out resulting in more deformation.

2.2.1 *Large scale testing of steel pipe/nail plate connections*

The next step was to test a large specimen consisting of four piles and a section of floor, with the piles embedded in soil. This allowed the effect of the soil/pile interactions to be taken into account and for different arrangements of braces. Four tests were carried out in pairs. The pile sets were initially reacted against each other then the first one to fail was braced off cantilever piles to allow the other piles to be tested to failure. Two potentiometers were installed to measure overall displacement and two were installed on piles to measure pile displacement relative to the floor framing, and from which the angle of pile rotation could be calculated. The soil capacity was not tested but the soils had not been modified at the site since it was tested in March 1994, by Thurston (1996). The average shear vane test result was 194 kPa at 150 mm depth and the average penetration per blow for a Scala penotrometer was 57 mm, at 0 to 300 mm depth. The site does not therefore meet the “good ground” requirements of Chapter 3 of NZS3604.

The general layout of the test specimens is shown in Figure 4. Framing and nailing were as per NZS3604:1999, including Amendment 1 & 2. Piles were pre-cut 125*125mm with 600 mm clearance between the bearer and the top of the concrete footing. Bearers were 90*75 mm spanning 1.35 m and joists were 190*45 mm spanning 2.0m. Joists to bearer connections were 2 no. 100*4 mm skew nails. Flooring is standard 20 mm flooring grade particle board fixed with 60*2.8 mm ring shanked flooring nails at 150 centres on sheet edges and 300 mm centres on intermediate supports. With overhangs, each piece of floor was 3.2 m long (along joists) and 2.5 m long (across joists). Each floor section was loaded with 400 kg of bricks and/or concrete pavers to simulate a factored live load and dead load of superstructure of 0.5 kPa. Footings were the minimum size permitted in NZS3604, of 200 mm square and 200 mm deep. The top of the concrete footing was a minimum of 50 mm below ground level, but varied from 50 to about 80 mm due to a slight slope in the site.

Figure 4. Large scale test set-up.

Three arrangements of braces were tested, as well as a control specimen with no braces. The pile-to-bearer connection consisted of 2 no. 100*4 mm skew nails and a 25*1 mm Grade 300 nail strap with 3 no. 8 gauge screws on each side of the pile and bearer in all tests including the one with no braces. One specimen had braces located between the piles and bearers with loading parallel to the bearers (Figure 2, left), and the other two specimens had braces running from the piles directly to joists, at 45 horizontally to the bearers and joists (Figure 2 ,right). In the last two specimens the direction of loading was along the joists in one, and along the bearers in the other.

The specimens were tested cyclically by pushing the specimens apart and then together in steps of 2 kN, using a hand pump. The pump did not allow a gradual release of pressure so when the peak load had been reached the jack released load very quickly, so the unloading leg of the load-displacement curve was very rapid compared with the loading leg where few data points were captured. The specimen without bracing resisted a lateral load of 6 kN in both directions but failed at about 8 kN or 2.0 kN/pile with the load displacement plot shown in Figure 5(a). Failure was due to rotation of the pile footing in the soil.

In all cases specimens with braces performed far better than those without. Where the braces were connected directly to the joists (c and d) the capacity was higher than the 14 kN achieved when the braces were connected to the bearers (b). All of the 4 pile-to-bearer braces acted in the same direction and the capacity appeared to be reduced when the joint closed as the brace was forcing the bearer and pile apart at the joint. The joint is “sloppy” in tension because there is a gap between the strap and the bearer immediately above the pile because the pile is wider than the bearer. Better performance may be achieved by varying the position of the brace so on some piles it is on one face and on the opposite face of other piles. Both tests c) and d) were stopped due to the hydraulic jack reaching the end of its travel, but for d) a spacer was installed to allow further travel in one direction only, but this limit was also reached. The capacity reached was about 20 kN or 5 kN per pile, 2.5 times that reached without bracing. The pile footings were sliding rather than over-turning. The capacity is limited by the passive earth pressure of the soil strength. Lower capacities would be achieved in poorer soils. but as previous penetrometer testing has shown, this site did not meet the NZS3604 criteria for "good ground". It is expected that most houses would be founded on better ground than at this site.

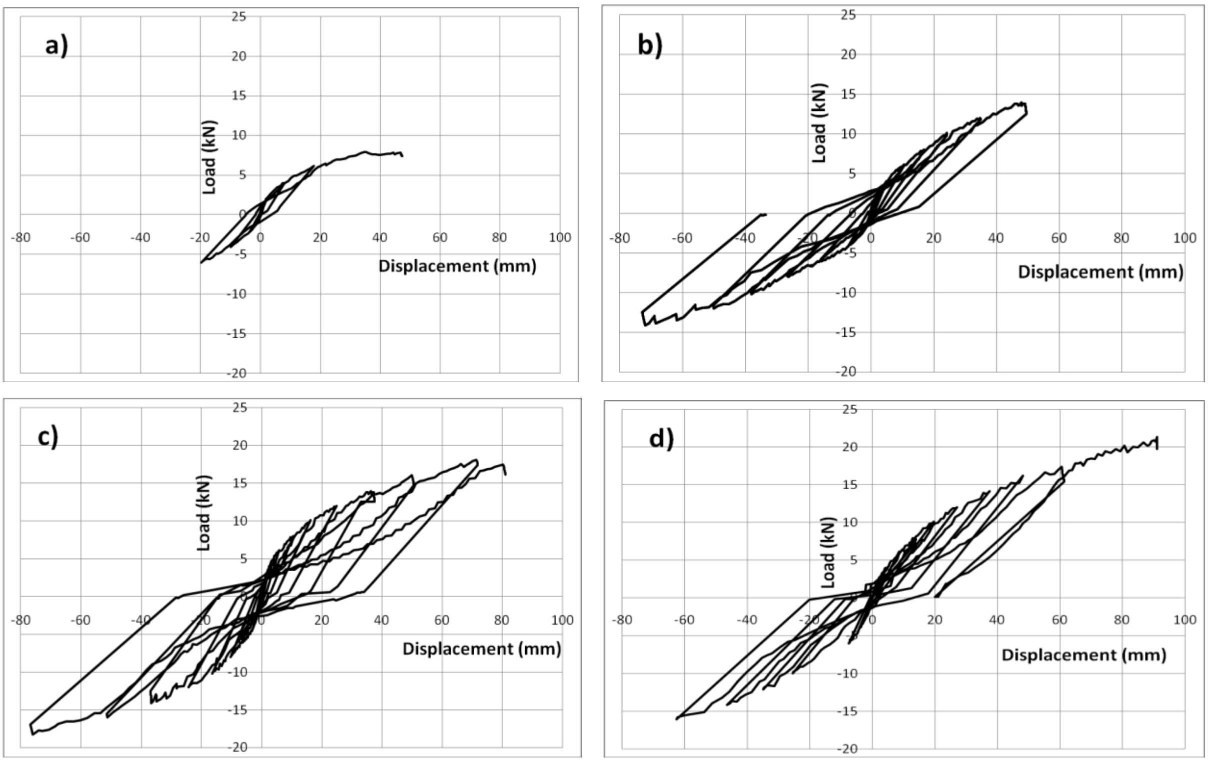

Figure 5 Load Displacement curves for large-scale pile test specimens. The scale on each graph is identical for comparison. Graph a) is with no bracing, b), with bracing between piles and bearers, c) and d) are for specimens with braces from piles to joists, c) with load parallel to the joists and d) with the load perpendicular to the joists.

Pile capacity increased to 5 kN or 100 bracing units (BU). For a dwelling with typical pile spacings, similar to those tested this represents 25 BU/m^2. From Tables 8.8 and 8.9 of NZS3604:1999, this is sufficient to resist earthquake loads in Zone A for all single storey buildings, all two storey dwellings with lightweight wall claddings, and some with combinations of light, medium and heavy weight roof and wall claddings. It should be noted that houses with heavy claddings are required to have concrete perimeter walls by NZS3604, and preceding code requirements. These braces are therefore adequate for most piled dwellings, with short piles and close pile spacings founded on reasonable soils.

3 WALL TESTS

Lack of connections between concrete foundation walls and sub-floor framing was evident in the survey carried out by Irvine and Thomas, was seen by the author in houses inspected after the Canterbury Earthquake and previously identified by Cooney (1982). Timber braces can be bolted to concrete foundation walls and to floor joists or blocking between joists as recommended by Cooney (1982). The braces described previously were modified by folding the two straps over the diagonally cut end of the tube and an 8 mm hole drilled through the tube and straps. Braces were fixed to concrete walls using M8 "Ankascrews", a proprietary concrete anchoring bolt.

The straps at the other end of the brace can then be fixed to floor joists running parallel to the wall or blocking between joists running perpendicular to it. Sections of floor similar to those described previously were located on two pairs of parallel walls, ones set 1.35 m apart supporting bearers with loading perpendicular to the bearers and the other set 2.0 m apart supporting joists (Figure 6).

Figure 6. View of underside of wall specimens

The bearers for one pair of walls (rear of figure 6) rested on top of the wall. Although this is not the most common method of supporting bearers running perpendicular to a foundation wall, it has the least lateral load capacity compared with other methods such as insetting the bearer into a slot in the wall or supporting the bearer on a cast in-pier that sits lower than the top of the perimeter wall. This was shown dramatically in the test as the failure mode was roll-over of the bearer (Figure 7 (left)). The other wall specimen with joists supported by the walls and the braces fixed to blocking between joists performed more satisfactorily. The braces ran in opposite directions on each wall resulting in significant twisting of the specimen (Figure 7 right). Hysteresis loops are shown in Figure 8.

Figure 7. Left, failure of bearers in roll-over. Right twisting of other wall specimen

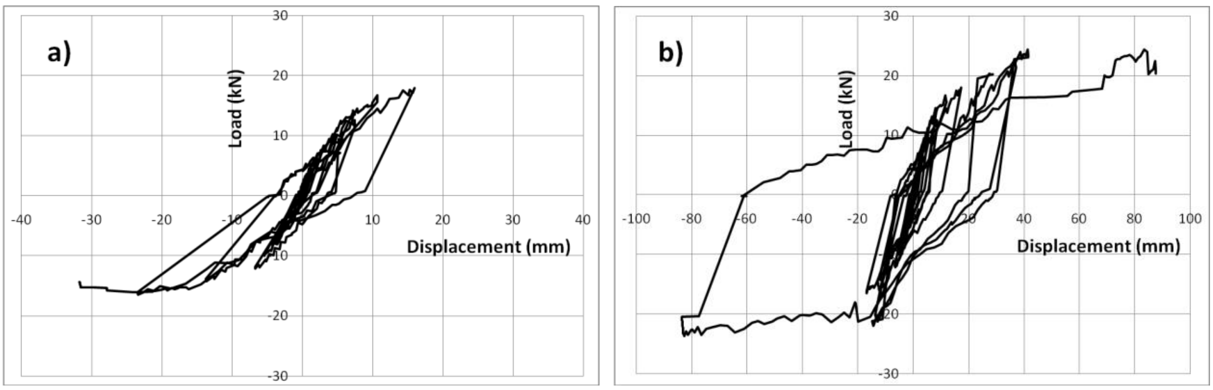

Figure 8. Load Displacement plots for walls specimens; a) bearer perpendicular to walls, b) joist perpendicular to walls. Note that although the vertical scale is the same for comparison, the horizontal scale is larger in a).

At low load levels the hysteresis loops were quite narrow for both wall specimens, but as the load exceeds 12 kN the behaviour became more ductile. The sudden increase in deflection between the -12 and -14 kN cycle in a) is because, with the braces faced the same way and acting in compression, the floor lifted allowing rotation of the bearers. In the other wall specimen the braces ran in opposite directions for each wall, but the specimen initially showed more ductility in one direction than the other. When the load exceeded 20 kN deflections increased dramatically with little increase in load, due to the steel straps yielding to failure.

4 DISCUSSION AND FURTHER WORK

The testing carried out has shown that pipe/nail strap brace can be easily fitted and is relatively ductile. Further testing is necessary to assess the effects of different brace layouts. The effectiveness in bracing shallow cantilever concrete piles on precast concrete pads should be also carried out, as these types of foundations are common in some areas. In the wall tests the braces achieved a capacity of about 5 kN/brace. This is lower than the 12 kN per NZS3604 brace, but distributing load rather than trying to achieve a 12 kN capacity in a fixing to existing concrete that may be in poor condition is desirable.

Significant deflections occur at low loads for braced piles. As these devices are intended to improve seismic performance at higher loads this is of little significance, and “slackness” is desirable to increase the building period and hence reducing accelerations (Thurston 1996). The hysteresis curves derived from testing should be subjected to a non-linear time history analysis to determine maximum loadings based on a maximum acceptable deflection in a similar process to that used by Thurston.

With increased pile height the effectiveness of the braces decreases. Longer braces could be used, but if piles are taller, there is better access and standard NZS3604 braces can be used. The test specimens used a mild steel pipe 33.7mm with a 4.0 or 3.2 mm wall thickness, depending on the strap width. Ends were cut to suit the angle the pipe was to be placed at. Durability for the nail straps can be achieved by using galvanised mild steel or stainless steel as appropriate. However the pipes would be expensive if supplied as stainless steel and the end cuts of galvanised pipe would be subject to corrosion unless galvanised after cutting. Given the thickness of the pipe used, some corrosion may be acceptable. For commercial supply, a range of end cuts and combinations would be required to allow for different joint geometries.

5 CONCLUSIONS

The steel pipe/nail strap brace shows promise as a retrofit device for house foundations. The testing has shown the device to be ductile and meet the pragmatic requirements of being easy to fix, even in confined spaces, and inexpensive. Further work is necessary to categorise performance. Suitability for precast shallow cantilever piles and footings and durability issues require further investigation.

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