Elasto-plastic behaviour of a rigid timber shear wall with slip-friction connectors

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ABSTRACT: The use of rigid engineered timber panels, such as cross-laminated-timber, in construction is increasing around the world, particularly in Europe and Australasia. Typically the panels rely on nailed or screwed steel plates for hold-downs and shear keys. However, this can mean the level of ductility is difficult to quantify. Furthermore ductile wall behaviour will inevitably be associated with permanent damage to the connections. There have been calls from designers for a solution in which the level of ductility can be predicted and achieved with confidence. The authors propose a novel, yet simple, slip-friction device that limits activated forces on a structure during an earthquake by allowing it to slightly rock. An experimental LVL wall was fitted with these devices acting as hold-downs. The shear key consisted of steel rods bearing against upright steel plates along the base of the wall. Under cyclic displacement tests, the wall demonstrated excellent elasto-plastic behaviour. The predicted wall strength from theory, matched, in general, the forces measured, while ductility levels can be as large as the designer desires, within obvious limits. Even under only self-weight, the wall readily descended at one end, while uplifting at the other. The results suggest that structures of engineered lumber can perform with reliable levels of ductility and remain free from damage.

1 INTRODUCTION

The design of seismic resistant structures relies significantly on limiting base shears to acceptable levels, and this can be achieved by a variety of means. Traditionally, through allowing the structure to deform plastically at pre-defined locations in the structure, catastrophic collapse can be avoided and damage confined to specific locations, but nevertheless the structure often ends up in an irreparable state following a design level earthquake. To improve the earthquake performance of buildings, recent years have seen a movement towards adoption of damage avoidance principles, i.e. low-damage design. Ways to implement such a concept are varied, and these include the semi-rigid joints developed by Clifton (2005) that permit non-linear behaviour without actual permanent material damage to the structural frame. Allowing structures to uplift and rock (Ma, 2010), or even considering the deformation of the soil foundation itself (Qin et al., 2013) are other ways of limiting base shear, and thereby reducing the possibility of damage, while also allowing structures to be designed more economically.

The mechanics of rocking rigid structures has been given considerable attention by researchers, and as early as 1963, Housner (1963) analytically described the behaviour of such 'inverted pendulum' structures. Makris and Konstantinidis (2003) have already established that the behaviour of a rocking *rigid* block cannot be described in the same manner as that of an SDOF oscillator - the restoring mechanism of an SDOF oscillator originating from its inherent elasticity, whereas the re-centring action of a rocking block comes from its self-weight.

Qin et al. (2013) and Acikgoz and DeJong (2012) have addressed the behaviour of flexible rocking structures (as opposed to rigid blocks), comparing these structures with those of comparable linear elastic oscillators and rigid rocking structures, and have revealed that flexible rocking structures have

a response different from both rigid rocking, and linear elastic structures.

Timber has seen increasing popularity as a construction material in recent years due to its beneficial characteristics in regard to sustainability, as well as its aesthetic properties. The rocking concept has seen implementation in construction using timber. Significant research has been carried out on rocking timber walls at the University of Canterbury in New Zealand. For these walls, post-tensioned cables assist in re-centring - as well as restraining against overturning up to a certain level, and small 'U' shaped steel dampers provide additional energy dissipation (Newcombe et al., 2011). Devereux et al. (2011) describes the design of such walls, and their inclusion in the NMIT Arts and Media Building situated in Nelson, New Zealand.

Loo et al. (2012a) have proposed a related concept for both flexible and rigid timber structures, but with damping provided mostly by slip-friction (or slotted-bolt) connectors that act as shear wall hold-downs. Base shear is capped by limiting the maximum overturning moment allowed, through adjusting the slip-force (F_{slip} = T) in the hold-downs connectors (see Figure 1a). The proposed connectors have force-displacement characteristics that are essentially elasto-plastic (Figure 1b), and provide energy dissipation in the form of Coulomb damping (Figure 1c). The damping effect of these friction devices are expected to override the unpredictability inherent in pure rocking, which relies on damping mainly through impact of the base of the wall with the foundation, and is sensitive to the material type of the wall and foundation. An additional possible benefit is that in applying the concept to rigid timber structures (such as those of CLT panel construction), as well as limiting damage, there is the benefit of reducing the extremely high response accelerations inherent in stiff and rigid structures (Loo et al., 2012a).

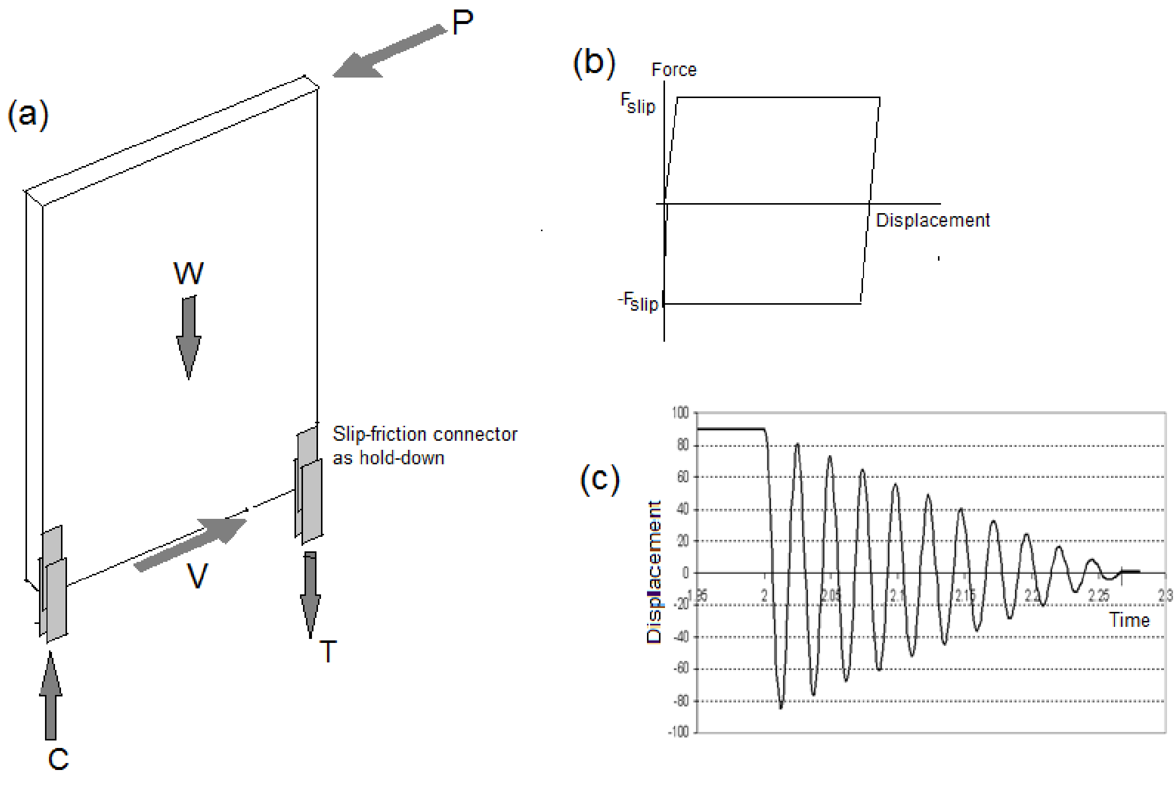

Figure 1. (a) General concept, (b) hysteretic behaviour of slip-friction connector, and (c) Coulomb damping characteristics on an SDOF system provided by the slip-friction connector.

Numerical studies carried out by Loo et al. (2012b) show that maximum drifts are likely to be within code mandated limits, and that residual drifts are expected to be small, even under a nominal amount of vertical loading.

This paper provides a brief discussion of experimental work on a new type of slip-friction connector proposed by the authors, and the results from tests on an experimental rigid timber wall. The slip-

friction connectors used are first described, followed by a discussion on the experimental set-up and shear key configuration, and a result from testing is provided.

2 SLIP-FRICTION CONNECTORS

Slip-friction (or slotted-bolt) connectors typically come in two types, asymmetric and symmetric. Both types consist of three main plates. The two mechanisms are shown in Figure 2, and are discussed in some detail by Loo et al. (2014a).

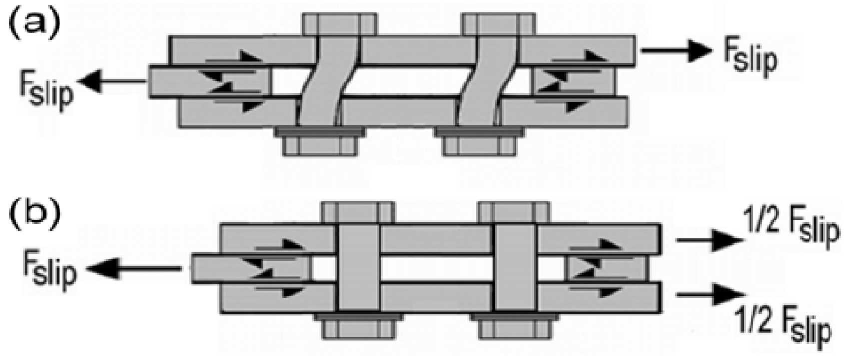

Figure 2. Slip-friction connector with (a) asymmetric mechanism, and (b) symmetric mechanism.

Asymmetric connectors have been used by Clifton et al. (2007) at the beam column joints of moment resisting frame (the sliding-hinge joint), and have been tested by Bora et al. (2007) as hold downs for pre-cast concrete walls. Both types of connectors have until recently been used in conjunction with thin brass shims (2 to 3 mm thick) between the main steel plates, in order to facilitate sliding and reduce wear. However, Khoo et al. (2012) has found that replacing the cartridge hard brass typically used with shims of abrasion resistant steel such as Bisalloy 400, in asymmetric connectors, obtains excellent sliding characteristics. For symmetric connectors Loo et al. (2014a) has carried out tests on connectors with a centre-plate of Bisalloy 400, and G350 mild-steel outside plates. This concept simplifies the fabrication process because shims are not required, and sliding takes place directly between the main plates. Problems from corrosion are reduced because there are thus only two interfaces as opposed to four (in the case where shims are used), and thus the opportunity for the ingress of corrosive elements is reduced. Furthermore, thin shims naturally have a greater likelihood of incurring corrosive damage throughout their small thicknesses. From a suite of component tests (Loo et al., 2014a) it was found that the connectors provided the required hysteretic behaviour, with excellent elasto-plastic characteristics (see Figure 3).

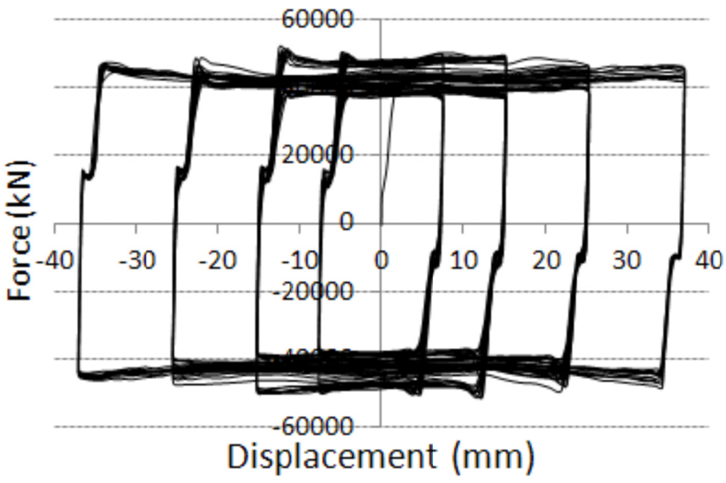

Figure 3. Hysteretic behaviour of connector with Bisplate 400 slotted centre-plate.

The next section briefly describes the experimental timber shear wall, with the slip-friction connectors of Loo et al. (2014a) implemented as hold-downs.

3 EXPERIMENTAL SET-UP

The experimental wall and its set-up is briefly touched upon in this section. Loo et al. (2014b) provides a more wide-ranging and comprehensive discussion. A 2.44 x 2.44 m wall panel was assembled from two separate 1.22m x 2.44 m, 45 mm LVL panels. Screw connections connected the panels through vertical studs and top and bottom plates fabricated from the same panel material. The wall was designed to remain elastic up to a racking force of approximately 120 kN (the actual maximum tested force was around 60 to 65 kN). The tests carried out were quasi-static in nature, and the actuator and general set up are shown in Figure 4a. The data acquisition system and hydraulic controller are shown in Figure 4b. To provide for lateral stability during testing, LVL planks (200 x 25 mm) screwed to timber members (150 x 50 mm) were used to form a 'channel' through which the wall would move with several millimetres clearance on either side, and one side of this channel can clearly be seen in Figure 4a.

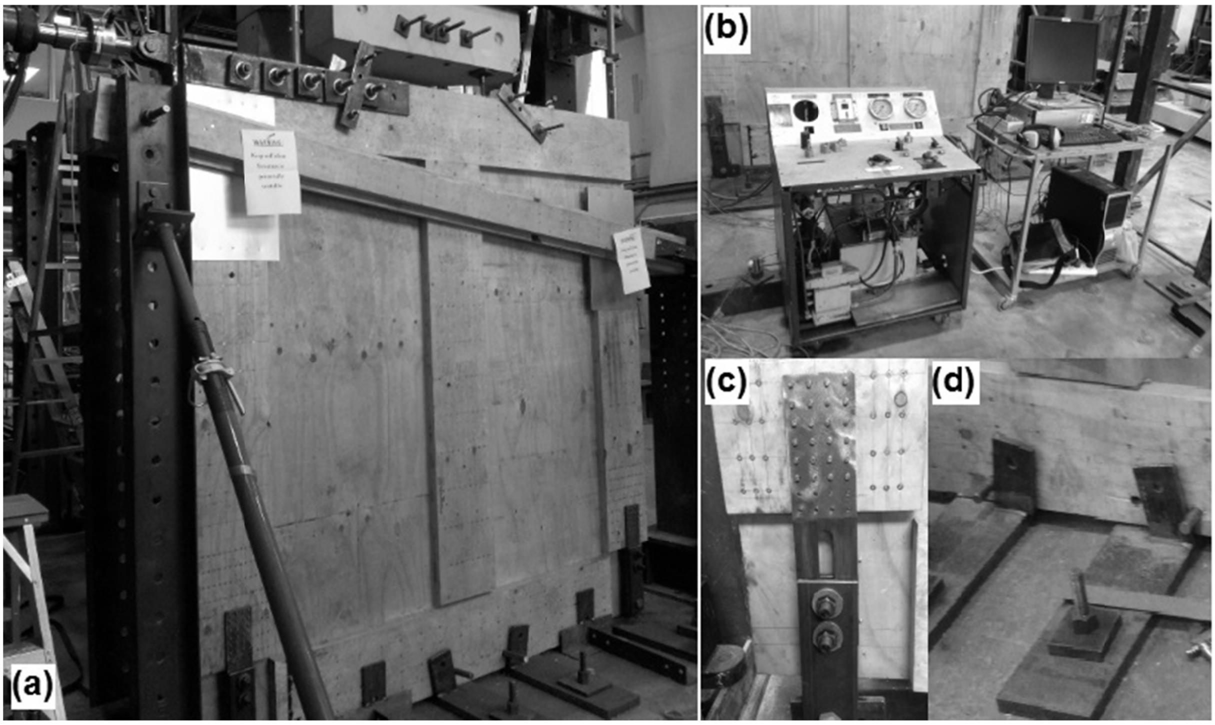

Figure 4. (a) General setup – note the connection of the actuator to the top-left corner of wall, with force transfer plates bolted against Belleville washers, (b) actuator controller and data acquisition system, (c) slip-friction connector, and (d) shear key.

The design of the bracing channel needed to take into account the small eccentricity of the loading of the connectors (attached to one side of the wall), with respect to the centre plane of the wall. However, the horizontal component of force on the walls is actually quite limited, as the effects from the connectors serve to counteract one another. If we consider the localised effect at either wall end, and assume a maximum connector slip-force of say 65 kN, the moment induced by eccentricity could be balanced out by a force of only 2.5 kN from the bracing (near its support). This 2.5 kN would create a maximum shear stress in the beam of around 0.9 MPa. The characteristic shear strength of LVL in NZ is 5.3 MPa to 6.0 MPa. If we consider the actuator itself, it was observed that it did not go off direction and remained perpendicular to the strong wall against which it pushed (and pulled) off. However, for the sake of this discussion it should be noted that even if it was, say 10 degrees off angle (highly unlikely, and almost physically impossible given the restraining nature of the joint at the strong wall), the component in the out-of-plane direction to the wall would only be around 10.4 kN. As this force would be applied close to the support of the beam, there would be few, if any bending effect, and the shear stress at the support would only be around 3.9 MPa.

The hydraulic actuator, applied force to the wall through a 900 x 100 x 10 mm thick mild steel plates on both sides of the wall. These mild steel plates were bolted to the wall using five 25 mm diameter bolts acting in double shear. Although bolted connections typically provide a somewhat 'loose'

connection, this was avoided by placing all five bolts in tension and thus mobilizing friction between the plates and the timber surface. This was achieved through the placement of Solon 16L150 Belleville washers at both sides of the bolts and tightening them to their flattened deflections.

LVDTs at the bottom corners recorded vertical uplift and a draw-wire was used to record horizontal displacement at the top corners of the wall.

The slip-friction connectors were attached to the end chords of the wall by riveted connections (see Figure 4c), and secured at the other end to the foundation using a single 24 mm bolt acting in double shear between steel brackets welded to the foundation. The shear-key assemblage (see Figure 4d) consisted of two solid steel 25 mm diameter pins inserted through the base of the wall, and bearing against upright mild steel plates on both sides of the wall. These mild steel plates were welded to the 32 mm thick foundation steel plates.

4 SHEAR KEY

The shear key is briefly described in this section. For a detailed discussion refer to Loo et al. (2014b). The shear key consists of two 25 mm diameter steel rods inserted through the base of the wall, with the two rods bearing against vertical steel plates welded to the foundation. Note that the edges of the plates against which the steel rods bear, are sloped at a slight angle (12 degrees to the vertical, this in order to reduce frictional effects and to facilitate overturning of the wall).

The forces on the wall are shown in Figure 5. Note that the forces provided by the slip-friction connectors and the forces on the steel rods of the shear pin, are the maximum potential mobilised forces.

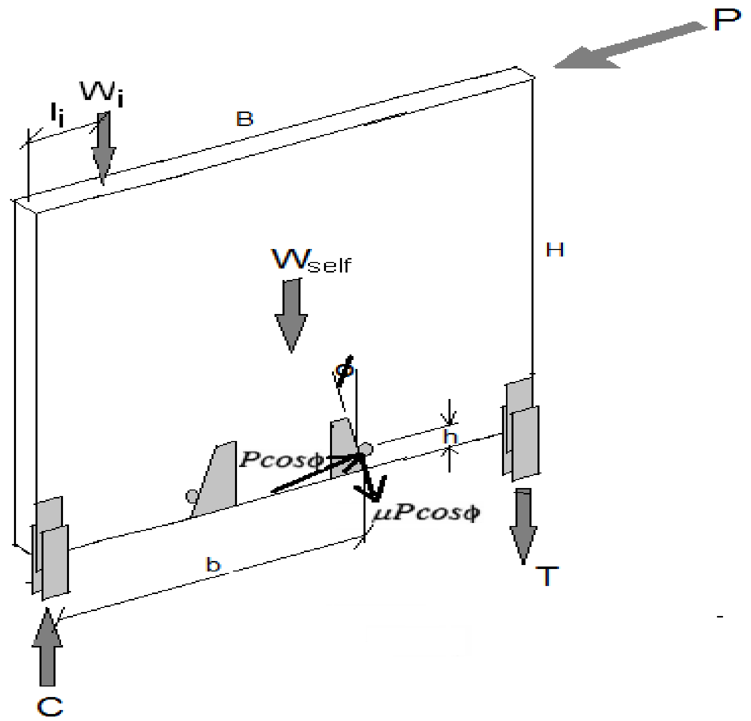

Figure 5. Forces on shear wall, with racking force P applied at the top corner. Note that the forces represent the mobilised forces during overturning movement.

From Figure 5, an expression for racking force, P, can be derived:

$$P = \frac{F_{slip}B + \frac{W_{self}B}{2} + \sum_{i=1}^{n} W_i l_i}{H - K_{mrp}} \quad (1)$$

, with K_{mrp} a multiplier representing the effects of friction and geometry of the shear key and wall:

$$K_{mrp} = (h^2 + b^2)^{1/2} \cos \emptyset \left[\mu\cos\left(\emptyset - \tan^{-1}\left(\frac{h}{b}\right)\right) - sin\left(\emptyset - \tan^{-1}\left(\frac{h}{b}\right)\right)\right] \qquad (2)$$

The wall strength, P can be plotted against Ø for various coefficients of connector strength, F_{slip}. Figure 6 shows the relation of P with Ø for the experimental wall, with parameters μ= 0.61, h = 0.06 m, b = 0.91 m, W_{self} = 2.8 kN. It can be seen that the strength of the wall reduces with increasing angle between the shear key and the vertical, but this is not necessarily a negative consequence – rather it indicates that the relatively unpredictable frictional effects arising from within the shear key are somewhat mitigated.

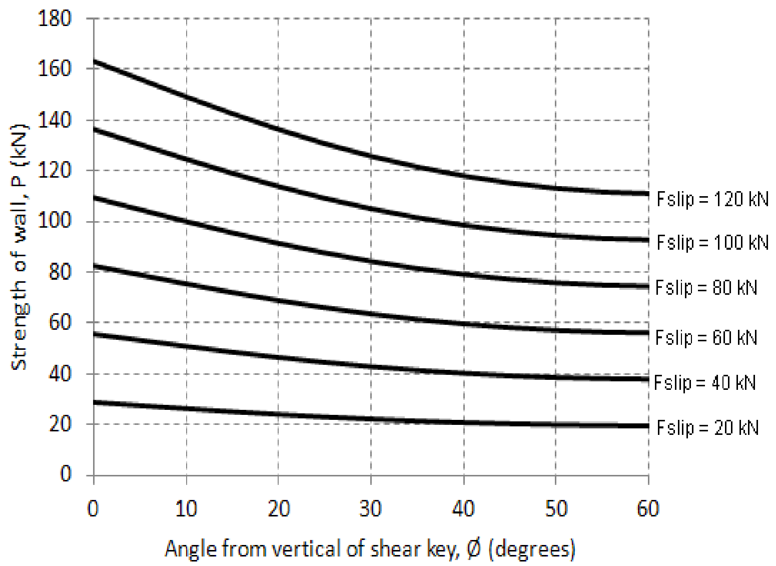

Figure 6. Variation of wall strength, P, as a function of shear key inclination from the vertical, and slip-threshold of the slip-friction connectors.

5 RESULTS

A series of tests were carried out on the wall, at incrementally higher slip-friction threshold forces (achieved through tightening of the Belleville washer stack to correspond to various deflections). Estimates of the peak racking forces were arrived at from Equation 1, and the observed strengths generally agreed with the predicted. Figure 7 shows a typical force-displacement result. It can be seen that the forces on the wall were essentially the same, for both loading directions, and this testifies to the precision achieved in adjusting the slip-friction connector forces at both ends of the wall. The behaviour of the wall closely corresponds to the ideal elasto-plastic case.

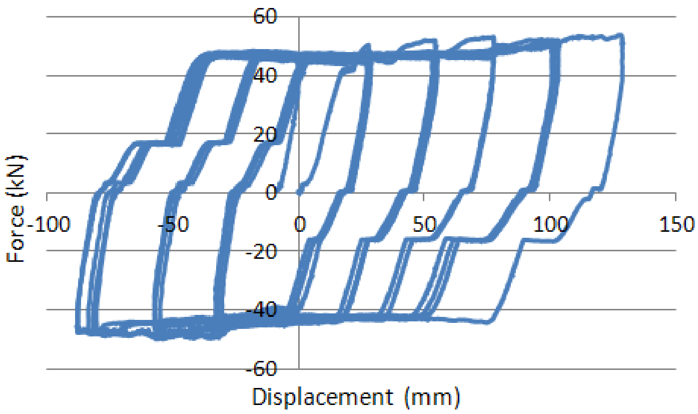

Figure 7. Hysteretic behaviour of experimental wall

Results from the tests showed that the measured strengths corresponded closely with the strengths predicted from Equation 1 (see Figure 8).

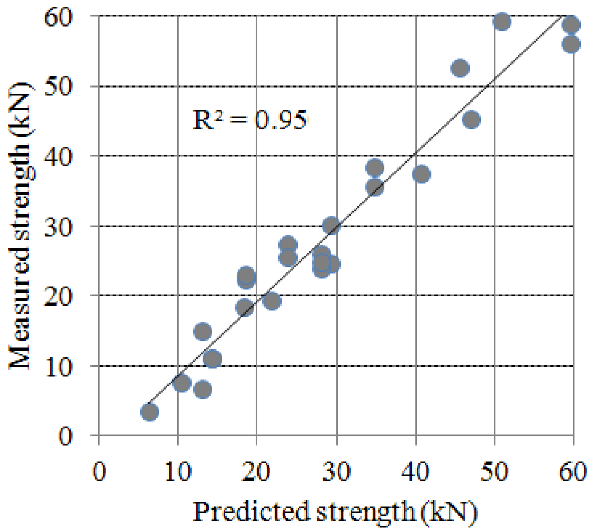

Figure 8. Wall strength, P: Measured strength compared with predicted.

It was also found that in general, the wall would descend at one end, while ascending at the other end. Thus the wall will not incrementally 'climb' up the slip-friction connector under loading, and the 'climbing' phenomenon would only be expected to occur if the absolute difference in slip-force between the two connectors was greater than the sum of the self-weight and vertically imposed forces on the wall. A comprehensive presentation and discussion of the results can be found in Loo et al. (2014b).

6 CONCLUSIONS

Experiments on a 2.44 m x 2.44 m wall were carried out. The slip-friction connectors capped base shears on the wall to their expected levels (also taking into account the effect of friction from the shear key). Excellent elasto-plastic behaviour was achieved, and re-centring potential is promising - given the fact that the wall would readily descend at one end, while uplifting at the other, this effect happening under what was essentially only the self-weight of the wall.

The concept has potential to be considered in the future design of structures of rigid timber construction, such as those of cross-laminated timber.

Further research will involve testing the current experimental rig with different shear key configurations and dispositions, as well as numerical and experimental shake table research to confirm the performance of such a structure, especially in regards to maximum and residual drifts.

7 ACKNOWLEDGEMENT

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