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THE CHALLENGES OF DESIGNING A HIGH-CAPACITY DUCTILE CLT SHEAR WALL SYSTEM FOR A 6-STOREY CLT BUILDING

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ABSTRACT: To design mass timber buildings to moderate or high seismic demands, strong, stiff, and ductile lateral load-resisting systems are required. One such system recently developed and tested at the University of Canterbury is the mixed-angle screw hold-down connection for Cross Laminated Timber (CLT) shear walls. Through the use of large European self-tapping screws installed at mixed angles with respect to the grain, the strong and stiff performance of screws installed at 45° can be combined with the ductility and displacement capacity of screws installed at 90° to the grain, providing an overall strong, stiff, and ductile connection. This paper explores the design of mixed-angle screw hold-down connections for CLT shear walls in a 6-storey (Concrete podium, 5-storey CLT wall) structure designed for a high seismic hazard. Key challenges from the design of this building and their associated learnings are presented. Challenges that had to be overcome included design strength prediction, overstrength prediction, stiffness prediction, and possible requirements for displacement amplification due to pinched hysteretic behaviour. These challenges are discussed in detail with reference to research findings, and some future outlooks for design are given.

KEYWORDS: timber connections, shear wall, seismic, ductile, self-tapping screws

1 – INTRODUCTION

Worldwide, the number of large mass timber buildings being constructed continues to increase. As the number of projects, increases so does the industry confidence, and thus, timber buildings are getting, larger, taller, and more complex.

In areas with significant seismic hazard larger structures may be required to develop inelastic or ductile behaviour to take advantage of the associated reduction in seismic design forces to allow for an efficient design. In New Zealand ductile CLT (Cross-Laminated Timber) shear wall systems have been a research focus of both the University of Canterbury and the University of Auckland. Through these universities the PRES-Lam system [1] and the Tectonus system [2] have been developed, and seen use worldwide [3], [4]. Most recently research at the University of Canterbury has focused on high-capacity CLT wall systems, with component testing of high-capacity dowels [5] and mixed-angle screw hold-down connections [6], as well as system level testing of coupled CLT shear walls with [7] and without [8] post-tensioning.

This large research effort has provided a great deal of data and solutions, but there are still many challenges that require careful consideration when taking these innovative and well researched systems and implementing them in an actual design.

This paper provides a summary and discussion of the challenges faced in the design of the 6-storey Haven Road Apartments building, which utilises a ductile CLT shear wall system using mixed-angle screw hold-down connections developed at the University of Canterbury.

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2 – BACKGROUND

2.1 Haven Road Apartments

Haven Road Apartments is a 6-storey (5 CLT platform construction, one reinforced-concrete podium) structure located in Nelson New Zealand (Figure 1a). The Project team consisted of ENGCO (Christchurch New Zealand) as design engineer, and PTL (Christchurch New Zealand) as peer reviewer.

Many options were considered for the lateral load resisting system at the concept stage. Given the large number of intertenancy walls typical of a multi residential structure (Figure 1b), a shear wall system was selected as the most efficient system. Both a CLT and concrete solution were considered. The CLT shear wall option was chosen for the relative light weight, carbon selling point, and the speed of construction.

CLT shear walls were well distributed keeping the relative stiffness in each direction similar for this L-shaped building, limiting torsional response and making it a more comprehensible system analysis for modal response spectrum analysis. Due to the relative high stiffness of the podium structure, the analysis was carried out as a two-stage approach in accordance with ASCE 7–16 Section 12.2.3.2.

Shear connections for the walls were provided with large castellated joints, with the floors seating in these castellations. This allowed for a high speed of construction, and eliminated any of the common compression issues due to perpendicular to grain loading of the floors under walls in platform style construction.

Choosing CLT for floors and all other gravity walls provided the optimal system for passive fire protection by charring and allowed for high speed of construction in a site with difficult access, storage, and propping constraints. For some individual walls with heavy loads, encapsulation with Gypsum plasterboard linings was still required to reach the minimum 30min fire resistance rating of the sprinkler protected building.

The choice to use mixed-angle screws as the hold-down system was carefully considered. The relatively high seismic demand of Nelson required a high-capacity hold-down solution beyond what can reasonably be provided by off the shelf cold-formed steel brackets. To reduce the seismic demand and allow efficient design, a system with ductility of $\mu = 2$ was preferred.

Further constraints were applied by the concrete podium transfer structure sitting below and supporting the CLT wall system. The transfer beams below CLT walls needed to remain practically sized with limited depth available due to planning limitations in the overall height of the building. To keep these members a reasonable size, the overstrength of the connections needed to be limited to an acceptable level. A stiffer hold-down was also preferred to limit the building drifts. Of the systems considered, the strong, stiff, and ductile behaviour of mixed-angle screw hold-downs was found to be the best fit for these requirements.

From a constructability perspective, the previous experience of the design team with mass timber structures and various forms of hold down connections resulted in a preference for the use of self-taping screws. Self-tapping screws allowed for the easy and straight forward construction of the structure and avoided some of the construction issues encountered with previous projects that used dowelled hold-downs.

Overalls the system chosen had the ease and simplicity required to get the project over the line for a budget conscious client, with sustainability, speed of construction, and demonstrated options for repairability in the event of a large earthquake being easy wins for the client.

2.2 – Mixed-Angle Screws

European self-tapping screws with large threads and small shank diameters are an increasingly popular timber fastener due to their ease of installation and high capacity. Research by Blass et al. [9] has shown that European self-tapping screws performed most efficiently at an inclined angle to the grain where they can resist a portion of the

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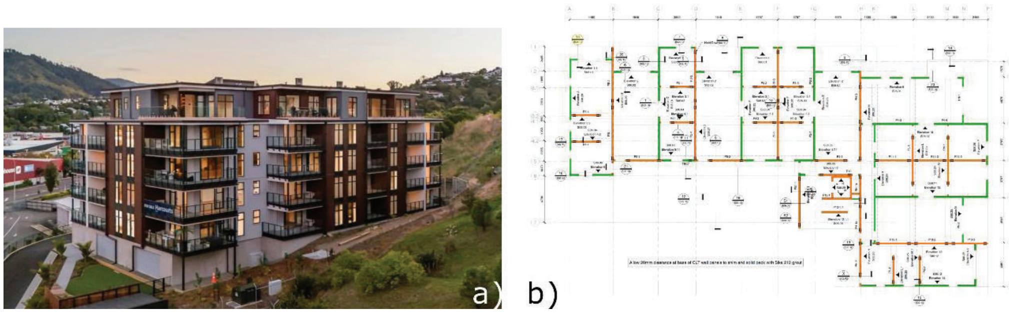

Figure 1 - a) Completed Haven Road Apartments, b) Floor Plan with CLT shear walls in Orange and CLT loadbearing walls in Green.

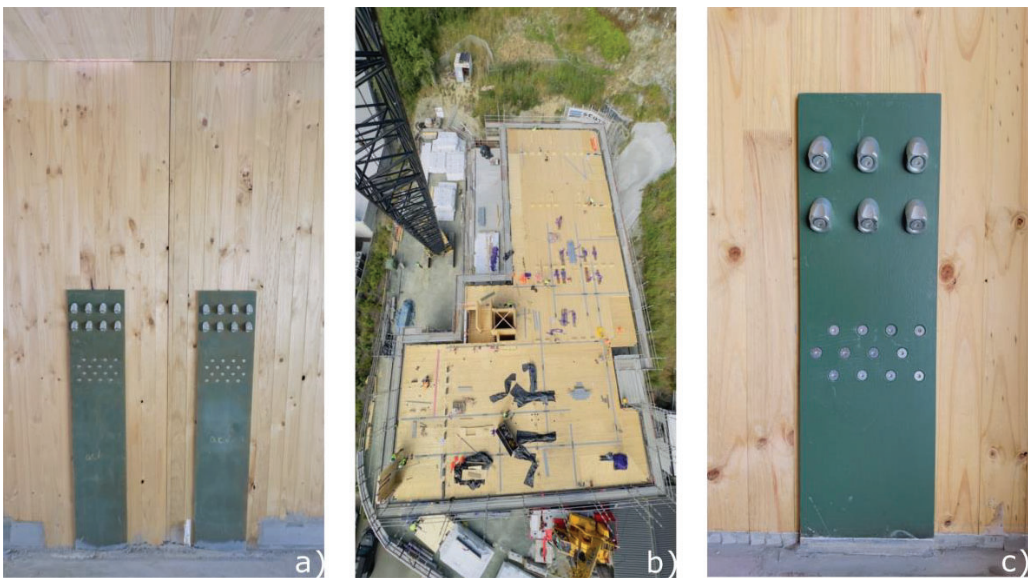

Figure 2 – a)/c) Typical Hold-Down Connections installed on Haven Road Apartments, b) Site under construction showing floor plan

load axially rather than in the typical dowel type action seen in nails or coach screws. However, as the angle of the fastener to the grain is reduced from 90° the ductility and capacity for inelastic displacement of the screw drops making these connections less suitable for ductile seismic design. To achieve the performance benefits of screws on inclined angles, and also allow for a level of ductility Tomasi et al. [10] proposed a timber-to-timber joint which incorporates both screws at an inclined angle and screws at 90° to the grain. By installing screws both at an inclined (commonly 45°) angle to the grain and a 90° angle to the grain, the performance of the two sets of screws can be superimposed. The inclined screws resist load axially and provide high strength, initial stiffness, but have little to no ductility or displacement capacity. In contrast, the 90° screws act as dowel type fasteners in shear with high strength and comparatively low initial stiffness, but provide high displacement capacity. By combining the two sets of fasteners in a single joint an overall strong, stiff, and ductile behaviour can be achieved. Further research by Hossain et al. [11] has extended this mixed-angle screw concept to in-plane joints in coupled CLT wall systems, and this was further extended by Brown et al. [12] for orthogonal panel joints.

Most recently, research at the University of Canterbury has extended this concept to the hold-down connection as shown in Figure 3/Figure *4*. Through extensive experimental testing, Wright et al. has identified the

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optimal ratios of inclined to 90° screws [6], provided guidance on the design prediction and overstrength predictions [13], [14], shown the connection can be repaired post-earthquake [14], [15] and discussed the impact of pinched hysteretic behaviour on the structure under Nonlinear Time History Analysis (NLTHA) [14]. In addition, the performance of mixed-angle screw hold-downs in CLT wall systems was confirmed with comprehensive large scale wall testing by Moerman et al. [8] as shown in Figure 5.

Overall, the performance of these mixed-angle screw connections has been proven through rigorous experimental testing. The system is overall strong, stiff, and ductile with significant capacity benefits over small light gauge nailed hold-downs and significant stiffness benefits over large scale dowelled connections.

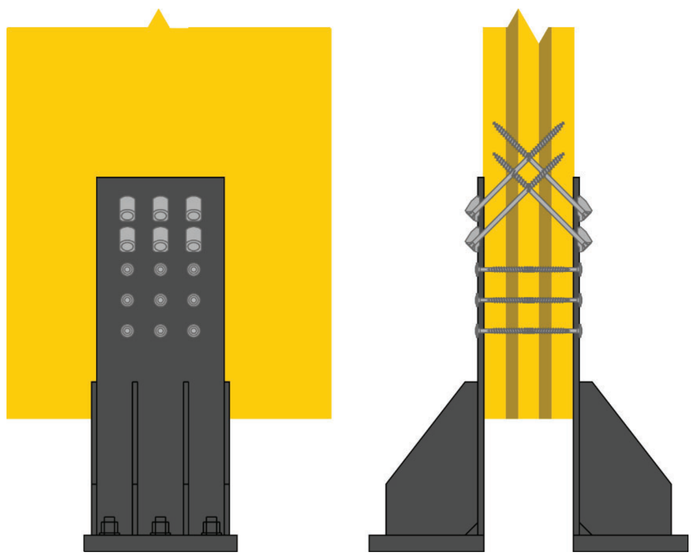

Figure 3 - Example connection layout used in experimental testing. Approximately 600 kN ultimate capacity

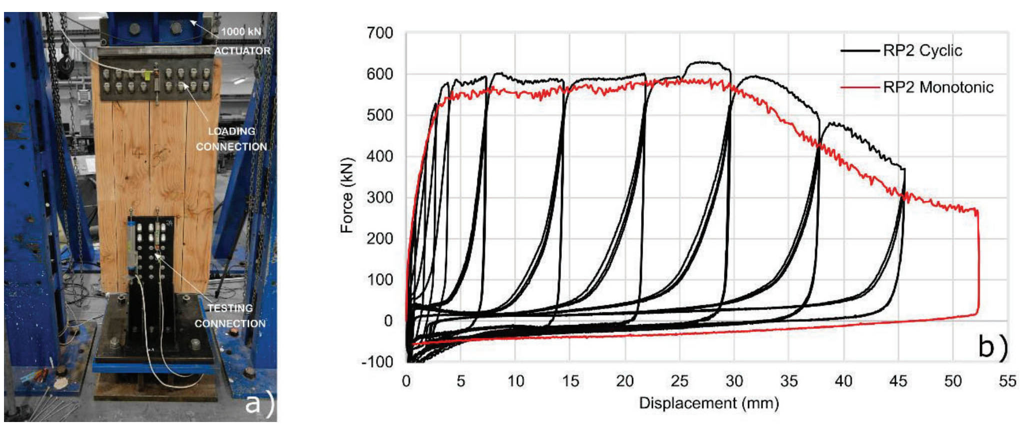

Figure 4 – Component testing of mixed-angle screw hold-down connections a) Testing setup, b) Force displacement results for RP2 test set results [6]

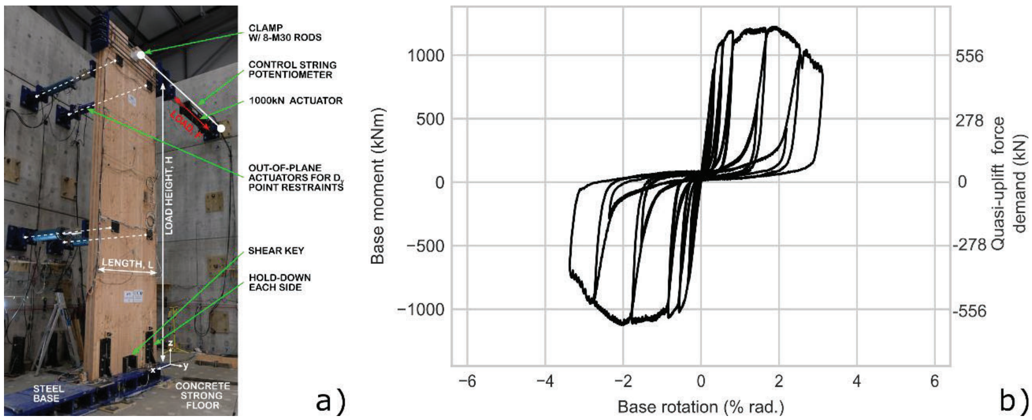

Figure 5 – System testing of mixed-angle screw hold-downs in large scale CLT wall systems a) Testing Setup, b) Moment rotation plot [8]

3 – KEY CHALLENGES IN DESIGN OF DUCTILE CLT SHEAR WALLS

In the design of the mixed-angle screw hold-down ductile shear wall systems for Haven Road Apartments, three key challenges were identified:

1. How to accurately predict the connection strength and overstrength for seismic design.
2. How to accurately predict the connection stiffness.
3. How to account for the pinched hysteretic behaviour of the connection.

Through close collaboration between the design engineers and the researchers these challengers were able to be resolved and their solutions with reference to research and design practice are discussed below.

4 – DISCUSSION

4.1 - Prediction of Connection Strength and Overstrength

A key issue faced in the design of Haven Road Apartments was the accurate prediction of connection yield strength and the associated connection overstrength. In the Haven Road Apartments project this was of particular importance as overall height constraints limited the available depth for the concrete podium transfer structure. This meant the section sizes in the concrete podium transfer structure had to remain efficient whilst also retaining adequate margin of strength to ensure capacity design principles were adhered to and a ductile failure mode achieved in the yielding connections of the timber walls above.

Connection Strength

For mixed-angle screw connections, Wright et al. [6] and Brown et al. [12] have proposed connection strength prediction methodologies based on simple summation of the contributions of inclined and 90° fasteners. However, both authors note that this does not reflect the true behaviour of the connection as the peak force contributions of inclined and 90° fasteners may not occur at the same displacements. To provide further clarity Wright et al. [14] examined the performance of the individual fasteners in small scale testing and subsequently calibrated beam on foundation models. From this it was determined at the displacement corresponding to maximum force in the inclined screws, the force in the 90° screws can be predicted by the European yield model approach excluding the contribution due to the rope effect. This omission is justified through examination of the displacements required to activate the full rope effect.

Using the approach from [14], the strength prediction for a mixed-angle screw connection is therefore:

$$R_{inclined} = R_{ax}(\cos\alpha + \mu_f \cdot \sin\alpha) + R_v(\sin\alpha - \mu_f \cdot \cos\alpha) \quad (1)$$

$$R_{90} = 2\sqrt{M_y \cdot f_h \cdot d} \quad (2)$$

$$R = R_{inclined} + R_{90} \quad (3)$$

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Where α is the angle between screw axis and direction of load and μ_f is the static coefficient of friction between steel and timber.

Wright et al. [14] noted that this prediction methodology provides accurate predictions using input values for withdrawal strength based on experimental test data. When current design equations for withdrawal strength from manufacturer ETAs or Eurocode 5 are used, this methodology provides highly conservative predictions of connection capacity, due to the inherent conservatism in the withdrawal prediction equations.

To avoid this conservatism and allow for efficient design of the Haven Road Apartments, results from experimental testing were used to inform design. The results used were extracted from Test Set RP2 in [6]. This Test Set RP2 tested a mixed-angle screw connection with 12 Spax 12x260 mm partially threaded inclined screws and 24 Spax 10x180 mm partially threaded 90° screws. Considering the provisions for prototype testing in NZS 1170.0 this resulted in a connection factored design strength of 453 kN.

A summary of these testing results alongside design predictions is shown below in Figure 6. These results show the analytical factored design prediction significantly underestimates the connection strength, being 47% lower than the factored design prediction based on experimental test data, and 59% lower than the mean values found in experimental testing. Given the discrepancy between these values, using the experimental testing values resulted in a much more efficient design than possible considering current analytical predictions.

Connection Overstrength

Using the analytical design prediction method, Wright et al. [14] recommends a connection overstrength of 2.07 for the specific ratio of inclined to 90° screws used. Accounting for the strength reduction factor (material safety factor) this means that elements governed by capacity design, need to be designed for approximately 3 times the ductile design demand. For Haven Road Apartments this was unachievable within the geometric constraints required for the concrete transfer structure. Using the experimental test data available, an overstrength of 1.45 was derived providing a much more achievable overstrength. Overstrength values and comparisons to experimental testing results are shown in Figure 6.

For efficient design in cases where test data is not available, one possibility is a cap or limit on overstrength actions. The New Zealand timber standard, NZS AS 1720.1:2022 [16] introduces a limit to capacity design actions by stating capacity design actions need not exceed the design actions for an equivalent elastic structure. In the case of the Haven Road Apartments which were designed for a system ductility of 2, this cap ranged from 1.7 to 2 times the design demand. In some cases where the hold-down connection strength was larger than the demand this cap governed connection overstrength design. To achieve a ductile response of the structure, the hold-down connection must yield

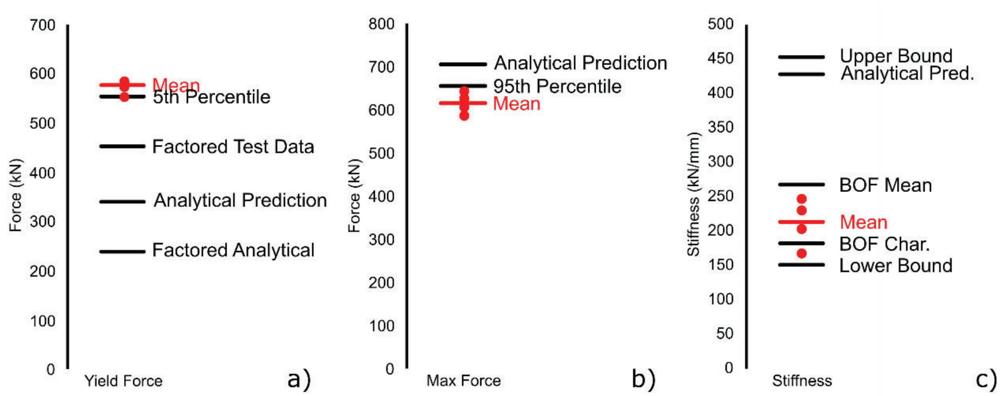

Figure 6 – Plots showing experimental test data (Red) overlaid with predictions (Black) for a) Connection Strength b) Overstrength c) Stiffness

before any capacity designed elements reach their ultimate capacity. In cases where this cap governs, designers should consider the structure may not respond in a ductile manner if the earthquake actions exceed the design actions, reducing the robustness of the structure.

4.2 - Prediction of Connection Stiffness

For predicting the stiffness of mixed-angle screw connections, Wright et al. [6] proposes the following equations based on derivations by [17]:

$$K_{ser,inclined} = K_{\parallel} \cdot \cos\alpha \left(\cos\alpha + \mu_f \cdot \sin\alpha\right) + K_{\perp} \cdot \sin\alpha \left(\sin\alpha - \mu_f \cdot \cos\alpha\right) \quad (4)$$

$$K_{ser,90^\circ} = \frac{\rho_m^{1.5} \cdot d}{23} \quad (5)$$

$$K_{ser} = K_{ser,inclined} + K_{ser,90^\circ} \quad (6)$$

Where $K_{\parallel}$ is the stiffness parallel to the fastener axis, $K_{\perp}$ is the stiffness perpendicular to the fastener axis, α is the angle between screw axis and direction of load, and μ_f is the static coefficient of friction between steel and timber.

For stiffness perpendicular to the screw axis, Eurocode 5 provides an approximate formula for dowel type connections (Eq. 5). However, previous research has found that this equation is not appropriate for all cases and can significantly overpredict the stiffness observed in experimental testing [18], [19], [20]. For stiffness parallel to the screw axis, European technical approvals (ETA) for each screw provide a formula for withdrawal stiffness. Similar to the above findings, using Eq 4-6 [6] found that for mixed-angle screw connections the stiffness was overpredicted.

To provide a more accurate prediction of stiffness, a beam on foundation model can be implemented. Previous research has shown these models may be used for prediction of both strength and stiffness of dowel type fasteners [21], [22], [23], [24]. Wright et al. [14] presents a simplified model using commonly available software SAP2000 for mixed-angle screws.

A comparison of the results from analytical prediction equations, BOF models, and testing results are shown in Figure 6c. These results show the analytical method significantly overpredicts compared to the stiffness observed from experimental testing. Results derived from the BOF model provide a comparatively accurate fit to the experimental data.

As a simple alternative to a beam on foundation model, Haven Road Apartments used an upper and lower bound approach to the stiffness. Considering the experimental testing results from the University of Canterbury [6], it was concluded that the connection yield displacement typically falls between 2 and 3 mm of hold-down uplift. Taking these values as an upper and lower bound to yield displacements, and using the experimentally derived design force, an upper and lower bound stiffness was derived. To find the most adverse lateral loads imposed on the structure, the upper bound stiffness was used. To find the most adverse displacement of the structure, the lower bound stiffness was used. By considering both an upper and lower bound, conservative design is achieved for all cases. Comparing these upper and lower bound values to those from experimental testing in Figure 6c a good fit was observed, however the upper bound stiffness was low due to conservatism in the design yield strength. To allow for this, 1 mm yield displacement was used for the upper bound. Considering the values presented in Figure 6c this provides an acceptable upper and lower bound for the experimental test results.

4.3 - Accounting for Pinched Hysteretic Behaviour

Ductile seismic design allows for the reduction in lateral forces due to the concept that an inelastic ductile structure can yield and drift to a similar displacement to that of an equivalent elastic structure. This response of inelastic structures was first reported by Veletsos and Newmark [25] and is described by the equal displacement rule, and the associated equal energy rule for short period structures. In design this equal displacement concept is implemented through the use of force reduction factors such as k_μ in NZS1170.5 [26], q in Eurocode 8 [27], and R in ASCE 7 [28]. The original derivation of these concepts [25] is based on oscillators with an idealised fat elastoplastic hysteresis loop (typically observed in structural steel), and doesn't take into account the significantly reduced hysteretic damping in the pinched hysteretic response common for timber connectors such as dowels, nails or self-tapping screws loaded in shear.

To account for this reduced hysteretic damping, the recent New Zealand standard NZS AS 1720.1:2022 [16] proposed a displacement amplification factor, k_{dt}, which has been derived based on the principles of Priestly's Direct Displacement Based Design method [29].

$$k_{dt} = \sqrt{\mu} \quad (7)$$

For a ductile CLT shear wall system with a system ductility of $\mu = 2$ this results in a $k_{dt} = 1.41$. To determine if this displacement amplification factor is

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applicable Wright et al. [14] investigated the performance of 3, 6, and 9 storey example structures using nonlinear time-history analysis. For system ductility of 2 structures it was found that the displacement amplification factor was not applicable, with the median peak displacement being overestimated by 142%, 71%, and 90% for the 3, 6, and 9-storey structures respectively as shown in Figure 7. This was justified with reference to previous findings from Stewart et al. [30] for plywood shear walls, which show that the increase in period of vibration of the structure due to the pinched hysteretic response decreases the seismic demand, and thus, mitigates to some extent the impact of decreased hysteretic damping.

Based on these findings the displacement amplification factor was deemed to not be applicable for the Haven Road Apartments project.

4 – OUTLOOK FOR FUTURE DESIGNS

Three key challenges and their solutions for the ductile shear wall design of Haven Road Apartments have been discussed. For the prediction of connection stiffness and the impacts of pinched hysteretic behaviour, clear solutions were identified and discussed with reference to literature.

For the accurate prediction of connection strength and overstrength, the low predicted strength and high overstrength was only resolved through the use of experimentation test data to provide a less conservative connection strength. Going forward this issue remains unresolved for future projects. For the design of timber connections, the natural and inherent variability of timber means that design predictions should rightly be conservative to account for the observed scatter in experimental testing data. However, for seismic design using ductility and capacity design principles, the level of conservatism observed and discussed in this paper is very large and does not lead to efficient and effective design. In the realities of a commercial environment an alternative non-timber option may be a more cost effective and efficient solution.

To reduce the conservatism in connection strength prediction equations, and make ductile timber design more appealing, two approaches are presented for reader consideration.

One approach could be to implement a probabilistic or reliability based approach where the large number of screws and timber layers penetrated can be taken into account, with factors being applied to increase the

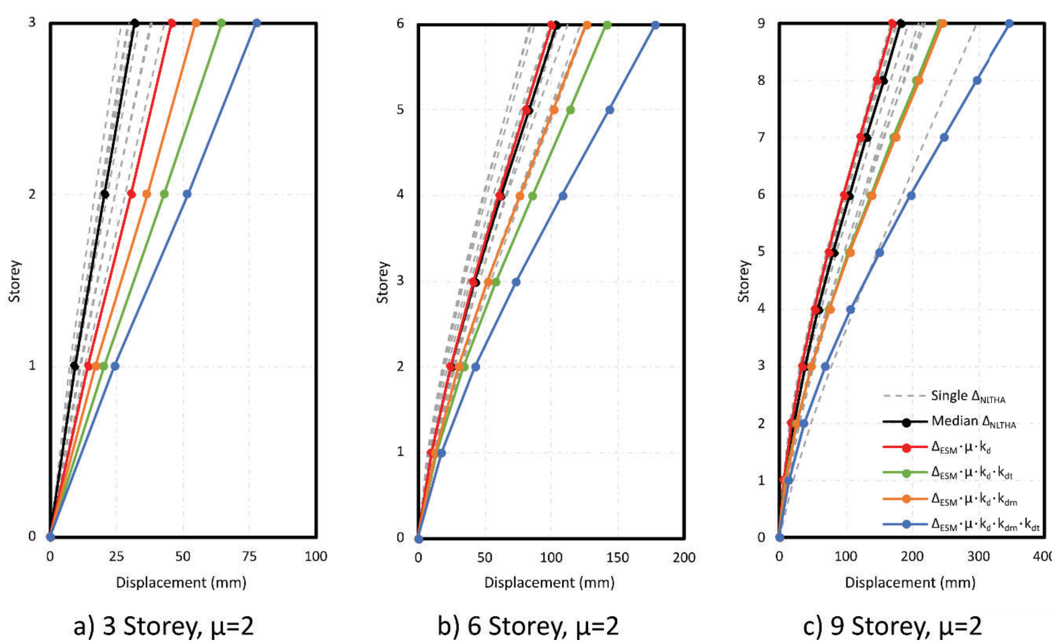

Figure 7 – Maximum displacement results from NLTHA of 3, 6, and 9 storey example buildings compared to predictions with various amplification factors [14]

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strength on the basis that, by definition, most fasteners will exceed the 5th percentile design strength. This could be through consideration of a parallel support factor similar to k_4 in NZS 3603:1993 [31], or through the consideration of a Eurocode 5 k_{sys} factor similar to that used by [32] where allowance is made for the number of layers penetrated.

A second approach could be to consider separate less conservative prediction equations for elements designed to yield under seismic actions. Specifically for the design of mixed-angle screw connections less conservative equations for the axial withdrawal of self-tapping screws are required. By allowing for separate equations a more appropriate level of conservatism could be applied for seismic design compared to design of gravity structure.

6 – CONCLUSIONS

This paper has presented the key design challenges of the 6-storey Haven Road Apartments building in Nelson New Zealand. How these challenges were overcome has been discussed and key recommendations for future designs include:

- The analytical strength predictions available for mixed-angle screws connections significantly underpredict the connection strength compared to experimental testing.
- To limit the overstrength of mixed-angle screw connections, less conservative predictions of input parameters such as screw withdrawal strength are required.
- Stiffness of the mixed-angle screw connections can be accurately predicted through the use of numerical beam on foundation modelling or through the use of simple upper and lower bound estimates.
- The impact of pinched hysteretic behaviour on mixed-angle screw connection's seismic performance is in the case considered mitigated by an associated elongation of the building period of vibration.

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