A Comprehensive and Analytic Study of Timber Construction

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Abstract - *This thesis describes a feasibility study into the use of a new method of timber construction developed in New Zealand. This new method combines the use of an engineered wood product (Laminated Veneer Lumber) and post-tensioned ductile connections. Three case study buildings are presented in concrete, steel and timber all representing current design and construction practice. A fourth building, referred to as the "timber plus" structure, is also considered with the addition of timber architectural components.*

The case study timber building consists of two lateral resisting systems. In one direction post-tensioned LVL moment resisting frames are used, with post-tensioned cantilever walls in the orthogonal direction. Timber-concrete composite floor units are also used.

The method of design and detailing of the timber building is shown with member sizes being found to be comparable to that of the concrete structure. Sub-assembly testing is performed on some key connections with excellent results. Construction time is evaluated and compared to the concrete structure with similar construction times being achieved. Finally the costs of the case study buildings are calculated and compared. The costing found the four options to be similar in price with the Timber and Timber plus buildings showing only a 6% and 11% increase in total cost respectively.

***Key Words*: Beam to column connection, Moment versus drift study, Axis depth versus drift study, post tensioning force versus drift study**

1. INTRODUCTION

Timber is one of the most ancient building materials in the world. Multi storey timber buildings date back for thousands of years. The 5 storey, 57m high To-ji pagoda in Kyoto Japan was constructed in 1695 and is to this day the highest timber building in Japan. The older and taller Sakyamuni pagoda (Figure 1.1) in Yingxian province, China was constructed in 1056 and stands at 67.3m, it is the tallest ancient timber structure in the world.

Medieval Europe also used a large amount of timber framing for the construction of multi storey timber buildings. The oldest surviving of these dates from the 12th century. The Knochenhaueramtshaus (Figure 1.2) in Hildesheim, Germany was once considered the most beautiful half-timber building in the world, with 8 storeys standing 25.72m tall. A half timber building consists of wood framing filled with plaster, brick or stone. After being almost completely destroyed during the Second World War it was reconstructed in 1987 to the original design.

Although timber construction has had a long history throughout the world, in latter years it has been falling behind 'modern' construction material such as concrete and steel.

Modern timber construction largely consists of residential structures. This is mainly due to the use of large wall panels being necessary for seismic resistance. Timber moment connections have previously been avoided due to difficulty of construction and significant costs. However, as global focus shifts towards sustainability and environmental concerns timber construction is an obvious choice for the future.

2. LITERATURE REVIEW

Study into the performance of multi storey timber structures can be separated into two categories; light timber framing, and heavy timber construction. The use of light timber framing for housing in New Zealand has been well documented (Garret 1990). This research culminates in NSZ 3604 for the design of light timber framed buildings, which covers non-specific design of buildings fitting within the scope of the standard.

Considerable work has also been performed regarding the design of multi-storey ply shear walls (Stewart 1987, Deam 1997) and hysteretic loops and analytical models have been developed. However, it is required that large walls be used for this method to ensure adequate lateral resistance. This can mean that for medium rise buildings a considerable number of internal walls will be required to resist lateral loading. This in effect 'locks' the internal space of the building making a change of usage impossible. In addition, modern commercial structures often require open plan in internal spaces, making the use of walls impossible

This method of construction under inelastic lateral loading displays a large amount of pinching behaviour (a significant loss in stiffness due to the inelastic damage around each nail allowing movement), leading to a considerable loss of stiffness during cyclic loading.

The use of cross laminated (cross-lam) panels has also become popular for use in medium rise buildings in Europe (Ceccotti 2008), with rapid erection being realised using pre-fabricated tilt up panels. However, this system still requires an extensive number of walls making it unsuitable for open plan structures

The development of a multi storey building system for timber relies on the development of either a moment connection or a braced system. Although considerable development in the construction of moment resistant knee joints for portal framed structures has been achieved (Hunt and Bryant 1988, Van Houtte 2003) a feasible frame connection remains illusive. Fairweather (1992) and Buchanan and Fairweather (1993) attempted to develop moment connections concentrating plastic deformation at the interface between the beam and column member. These connections suffered possible brittle failure due to the variability of the Glulam members.

3. CASE STUDY BUILDING

The case study building used for the project is a six storey structure that is to be built at the University of Canterbury for the Biological Sciences department (Figure 1). The actual building is to be constructed in pre-cast concrete. The building has two distinct lateral resisting systems in order to resist loading in both the north-south and east-west direction. In the long (east-west) direction a moment resisting frame will be used. In the short (north-south) direction structural walls will be used.

The structure has been designed to be in the Christchurch region in what can be considered to be a moderate seismic zone. The foundations are in reasonably good conditions considered to be a shallow soil. For all design the current New Zealand design codes have been used. Where these have not been adequate, particularly in the case of the timber structure, other relevant international codes have been utilized.

Fig. 1: Original concrete building (courtesy of Courtney Architects)

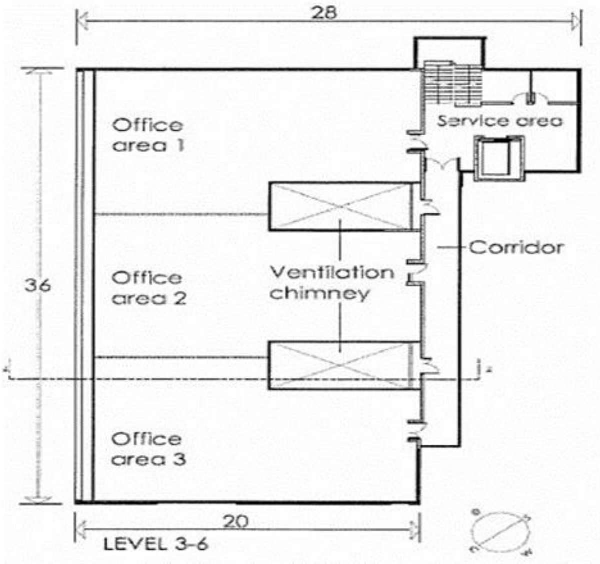

Fig. 2: Architectural floor plan of case study building (Perez 2008)

TIMBER STRUCTURE:

The basic form of the Timber building (Figure 3a) will remain similar to that of the concrete structure with the use of frames and walls. The structural system will be altered to use a new method of connection currently under investigation at the University of Canterbury. This combines the use of un-bonded post tensioning cables and sacrificial mild steel in order to achieve force resistance. This system is essentially damage free after a major event and will return to zero residual displacement; these are major advantages for any structural system. The floor units are timber-concrete composite with 65mm of reinforced concrete poured onto 17mm ply sheets which are supported by LVL joists. Figure 3b shows a typical flooring plan for the timber structure.

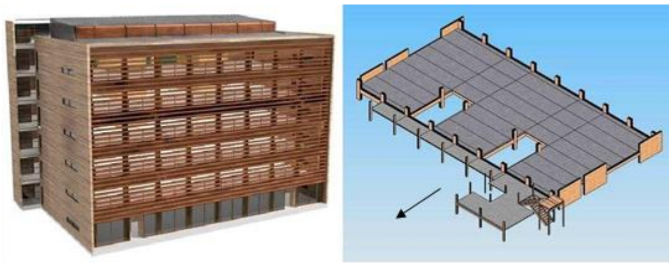

Fig. 3: Timber Building

4. RESULTS & DISCUSSIONS

Connection Testing:

The following chapter outlines tests performed to assess the performance of key connections adopted in the design of the timber case study building. The first tests assess methods to resist shear at the base of a wall or column member due to lateral loading. Secondly a series of four simple pushout tests is performed to find an initial indication of characteristic strength of the beam to floor diaphragm connection.

4.1 Testing of Angle Shear Keys:

During the testing of the single LVL hybrid wall (Smith 2006b) a major issue relating to the attachment of shear keys to resist seismic shear at the base of the wall was recognised. This will be an issue for both wall to foundation and column to foundation members. It was originally stated that the use of circular shear keys is preferable to the stiffer angular shear key previously used as movement and rolling over the keys is allowed. Although this is adequate for short term seismic testing it is necessary to revisit this idea if serviceable loading is to be considered. Due to the stress concentrations created on the end of the member (Figure 4) considerable damage occurs over the long term.

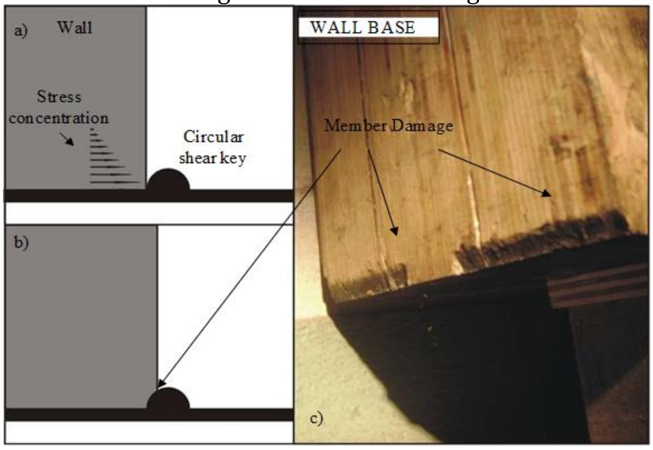

Fig. 4: a) Stress concentration at base of member b) Damage to member c) Damage in base of wall member after testing

The new design consists of a simple piece of angled steel, designed to allow the rocking of the column through yielding. The design procedure of the key is laid out in Figure 5.

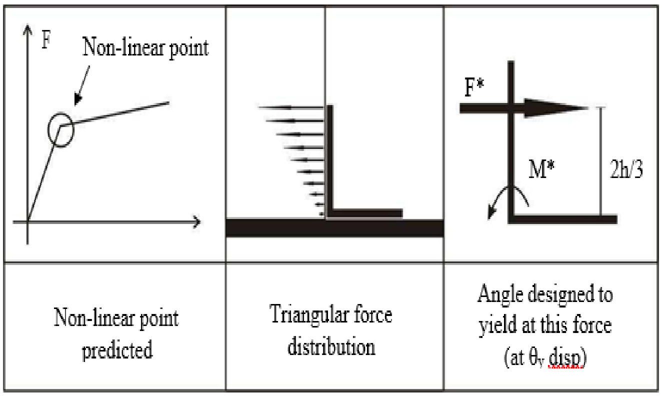

Fig. 5: Design of foundation shear key

Some simple seismic testing on column to foundation joint was carried out in order to validate this design procedure which is shown in Figure 6

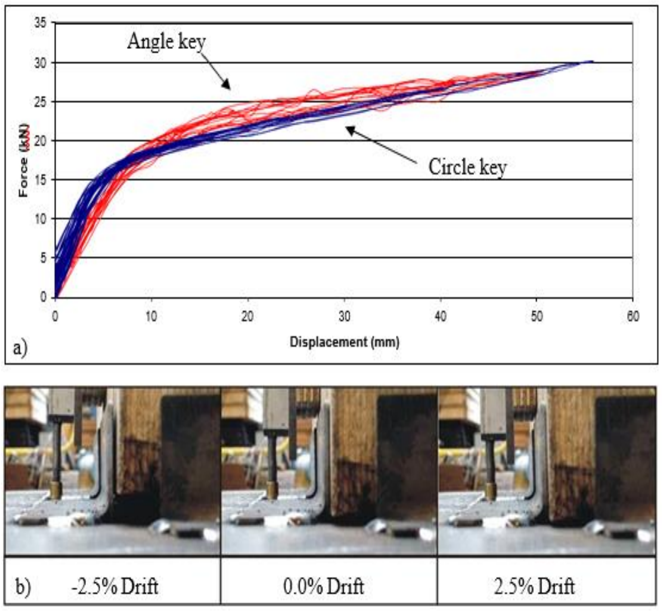

Fig. 6: a) Force versus Displacement with angled and circle shear key b) Angle during testing

4.2 Pushout Testing of Floor to Beam Shear Connection:

One of the crucial connections in the any building is that of the floor diaphragm into the seismic resisting system. In most seismic designs it is assumed that the floor diaphragm acts as a rigid block and that the connection between this floor and the frames and walls always remains elastic. Initial designs of the connection between the flooring and the seismic frames used the empirical values found by Siebold (2004), however, these tests were performed with a coach screw placed in the top of the member as shown in Figure 7. a. The method of connection used in this design has the screw placed parallel or perpendicular to the grain as shown in Figure 7.b.

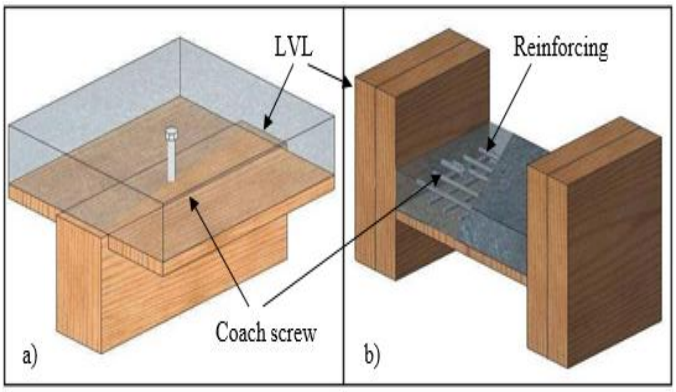

Figure 7: a) Siebold pushout specimen b) Test Specimen used

Four tests were carried out using this configuration: Two with the grain running perpendicular to the applied force and two with the grain running parallel.

Perpendicular to Grain Testing:

The results from these two pushout test are shown below in Chart 1.

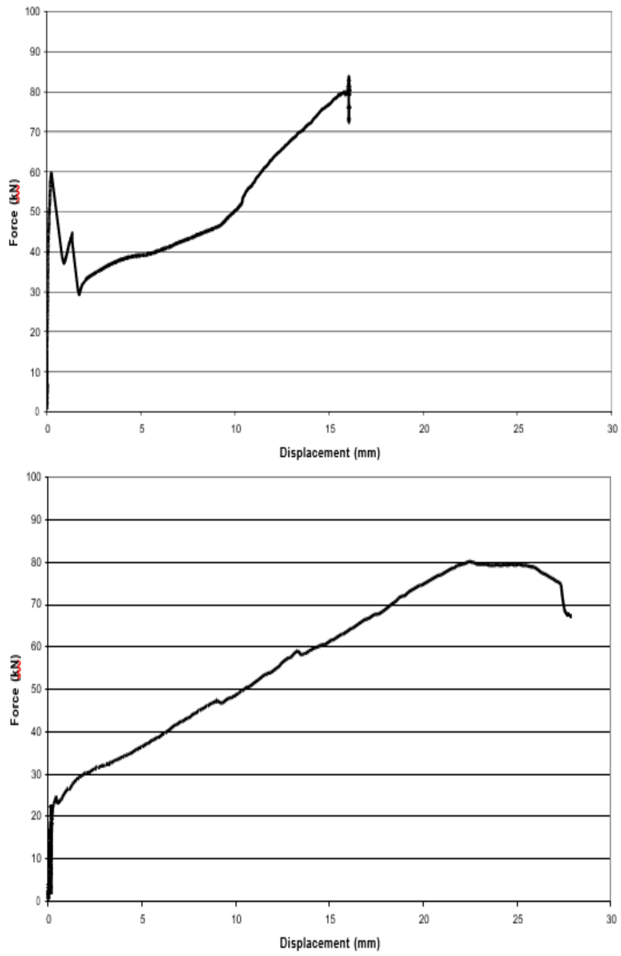

Chart -1: Pushout shear testing perpendicular to the grain for 2 specimens

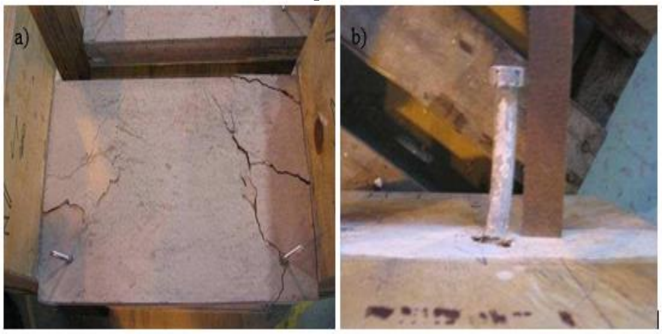

Fig. 8: Pushout specimen after testing: a) Damage to the concrete b) Deformation of coach screw inside concrete

Parallel to Grain Testing:

The results from these two pushout test are shown below in Chart 2.

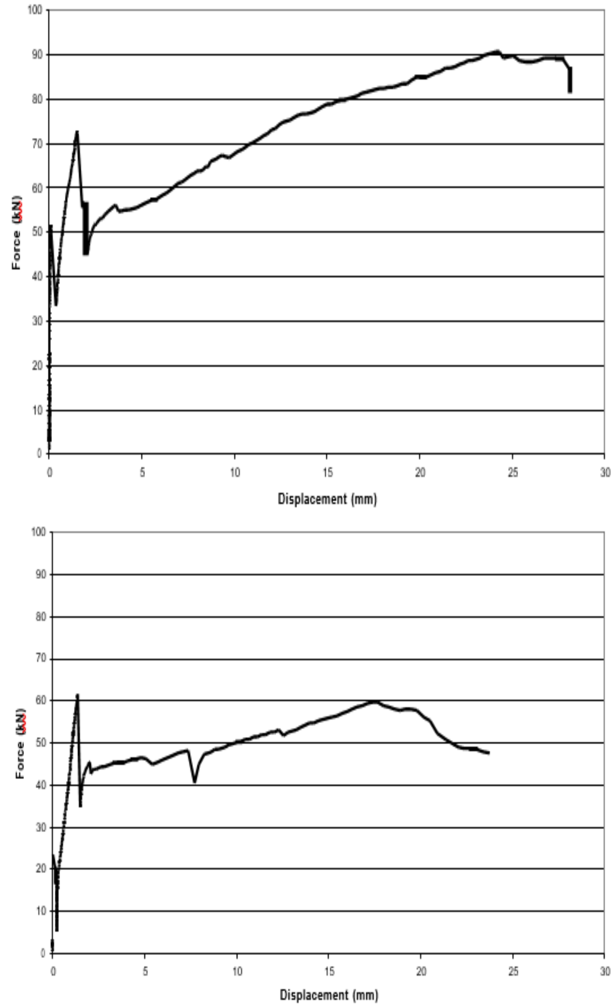

Chart -2: Pushout shear testing parallel to the grain for 2 specimens

Fig. 9: Failure of push-out specimens

Discussions:

It can be seen from the pictures in Figure 11 that when the failure mode of this connection occurs, it is very different to that of the Seibold test (shown in Figure 9). It is clear from these tests that the failure mechanism is in the timber with splitting of the block in the direction of the veneer. The failure of the modified connection clearly occurs in the concrete which cracks and a clear shear failure can be seen in the coach screw.

4.3 Testing of Beam to Column Connection using Differing Interfaces:

During the design of the beam to column connections for the case study timber building it was decided to armour the column face in order to eliminate the weakening effect that the compression perpendicular to the grain has on the overall strength of the connection. Although this effect has been speculated (Newcombe 2008) it has never been specifically tested.

This beam to column connection has been designed to withstand many cycles with the connection remaining elastic, avoiding significant inelastic compression of the LVL. This means the ratio between the connection capacity and the beam strength and column strength is not optimized.

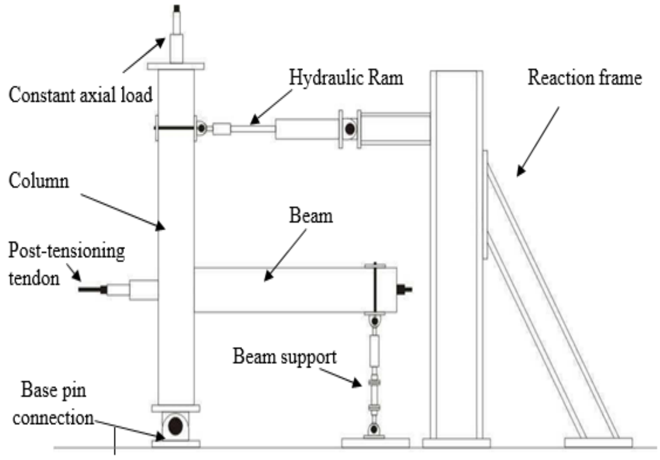

Fig. 10: Beam to column test setup

The first three tests were performed with the application of a 10mm steel plate armour between the face of the column and the end of the beam as shown in Figure 11 for 20%, 40% and 60% initial yield stress post tensioning force. Then, the same tests were conducted without steel armour at the interface with the same post tensioning force and the results are given below.

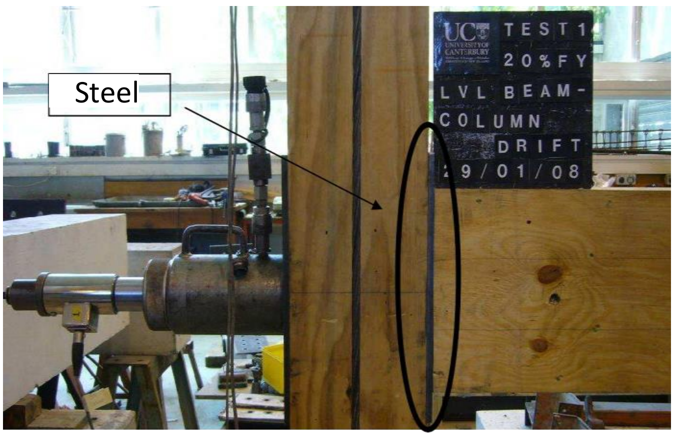

Fig. 11: Steel plate attached to the face of the column

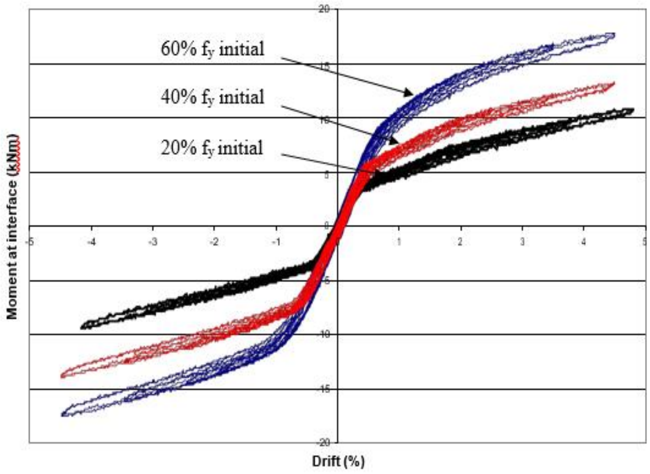

Chart -3: Moment at interface versus drift for beam to column testing with armouring.

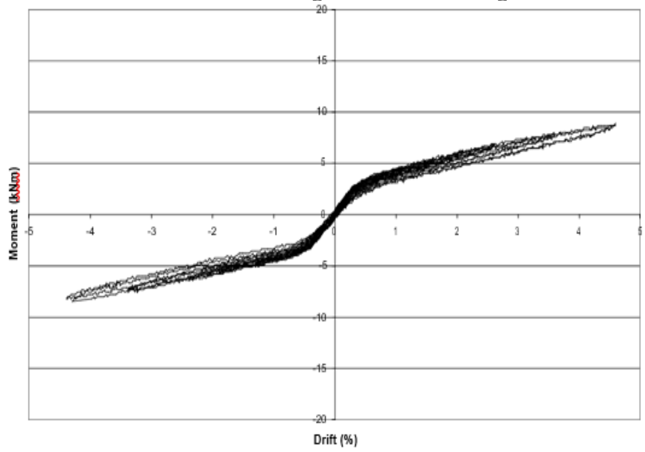

Chart -4: Moment at interface versus drift of non-armoured column with 20% initial PT.

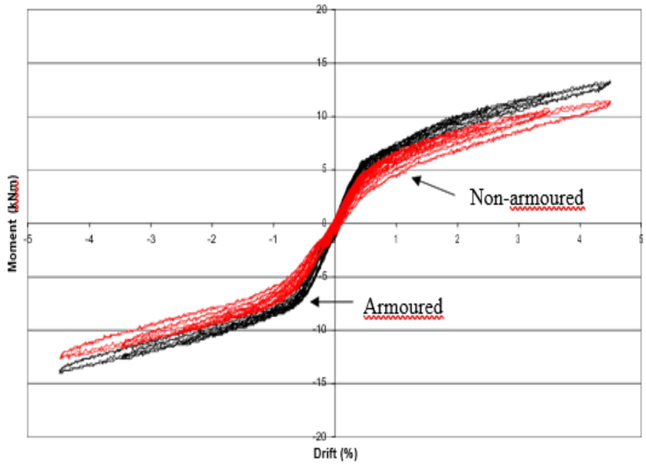

Chart -5: Moment at interface versus drift of armoured and non-armoured column with 40% initial PT

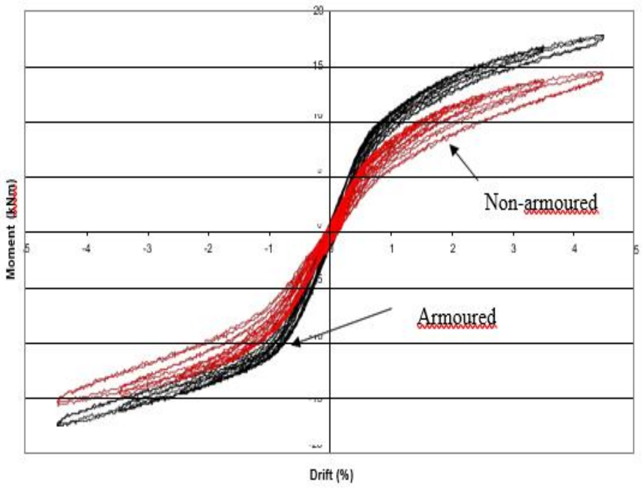

Chart -6: Moment at interface versus drift of armoured and non-armoured column with 60% initial PT

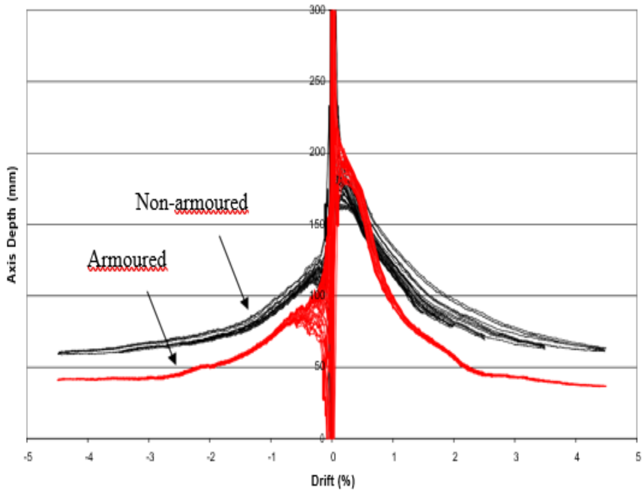

Chart -7: Axis depth (mm) at interface versus drift of armoured and non-armoured column with 40% initial PT

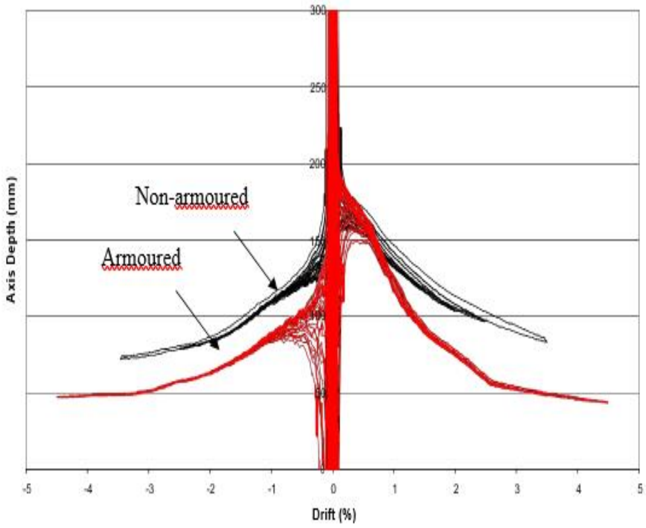

Chart -8: Axis depth (mm) at interface versus drift of armoured and non-armoured column with 60% initial PT

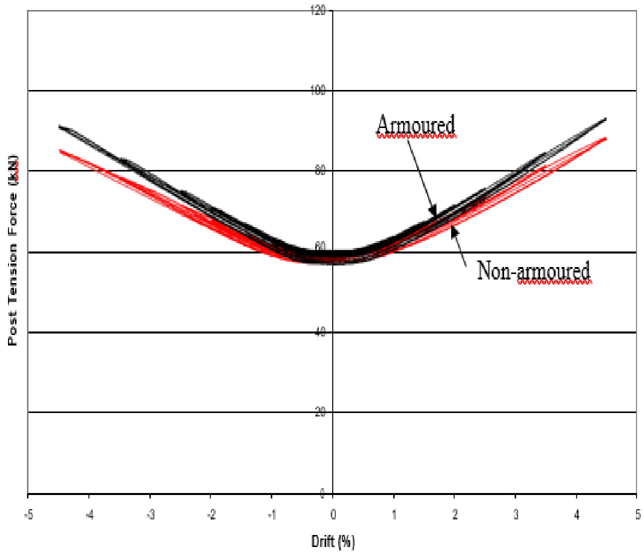

Chart -9: Post Tensioning force (KN) at interface versus drift of armoured and non-armoured column with 40% initial PT

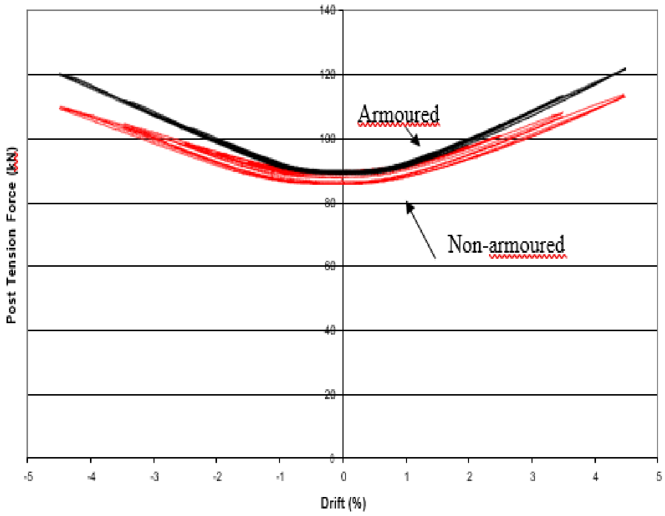

Chart -10: Post Tensioning force (KN) at interface versus drift of armoured and non-armoured column with 60% initial PT

5. CONCLUSIONS

Based on the present experimental investigations the following conclusions are drawn:

1. The usage of angled shear keys at the base of a wall or column is preferable to that of the half circular shear keys as it reduces stress concentrations and damage.
2. A minimum characteristic strength of 10kN is suggested for the beam to floor diaphragm coach screw connection due to this being the minimum value of the observed onset of non-linear behaviour, however, larger values than this may occur followed by a sudden slip failure.
3. The placement of steel armour at the beam to column interface causes a significant increase in both 'yield' moment and maximum moment (at 4.5% drift) by reducing the neutral axis depth.

4. Altering the initial post tensioning value in the column has the effect of increasing 'yield' drift, 'yield' moment and maximum moment (at 4.5% drift).
5. The design procedure developed as part of the post-tensioned timber research project at the University of Canterbury (Newcombe et al. 2008) adequately predicts the moment response of a beam to column connection but is more suited to a timber to timber connection. Further research is needed to asses the effect of inelastic behaviour in the joint.
6. The placement of corbels on the underside of the beam does not effect the moment response of the beam to column connection.
7. The placement of a floor unit on the beam to column subassembly caused unsymmetrical hysteretic behaviour to occur (from damage to the floor) due to the unsymmetrical nature of the specimen. Further research is required to understand the effects of the flooring unit on an interior joint.

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