Feasibility and Detailing of Post-tensioned Timber Buildings for Seismic Areas

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ABSTRACT: This paper describes the structural design and selection of construction detailing for low-rise multi-storey timber buildings using a new and exciting structural timber system. This system, originally developed for use with pre-cast concrete, combines the use of un-bonded post-tensioning techniques and additional sources of energy dissipation. This system eliminates residual displacement, while greatly reducing the damage to structural members during a significant seismic event. The paper shows how this new structural system can be used with large size structural timber members manufactured from laminated veneer lumber (LVL) or glulam timber, for use in multi-storey buildings, with lateral load resistance provided by post-tensioned structural timber frames or walls, separately or in combination.

An extensive on-going research program at the University of Canterbury, New Zealand has tested a wide range of beam-to-column, wall-to-foundation and column-to-foundation connections under simulated seismic loading, all giving excellent results.

As part of this contribution, a case study of the design methods, construction options, cost and feasibility of a six storey timber office building in a moderate seismic area is carried out. The structural design of this building allowed investigation of different methods of structural analysis, and the development of many construction and connection details offering feasibility of rapid construction. Total building cost was evaluated and compared to equivalent steel and reinforced concrete options.

INTRODUCTION

Recent developments in the field of seismic design have lead to the development of damage control design philosophies and innovative seismic resistant systems. In particular, jointed ductile connections for precast concrete structures (Priestley 1991, 1996, Priestley et al., 1999, Pampanin, 2005) have been implemented and validated. These solutions rely on a discrete dissipative mechanism placed in specific locations in the structure.

A precast concrete seismic resisting system developed in the U.S.-PRESSS program (PREcast Seismic Structural System), coordinated by the University of California, San Diego, for frame and wall systems has been shown to be particularly effective. This system, referred to as the hybrid system, combines the use of unbonded post-tensioned tendons with grouted longitudinal mild steel bars or any form of dissipation device (Fig. 1a). While the post-tensioning provides a desirable recentering characteristic, the dissipation devices allow adequate energy absorption by the system. During lateral movement a controlled rocking will occur at the beam-column (Fig. 1b), wall-foundation or column-foundation interface. This will be characterised by the so called flag-shaped hysteretic behaviour displayed in Figure 1c.

The hybrid system is in principle material independent and similar solutions have been proposed for steel moment-resisting frames (Christopoulos et al. 2001). Recently it has also been proposed to extend these solutions for low rise multi-storey timber construction. Testing on subassemblies carried out at the University of Canterbury by Palermo et al. (2005, 2006a, b) and Smith et al. (2007) have proved to be very successful and represent a viable option for multi-storey timber buildings. The new post-tensioned timber construction system is the subject of an international patent application.

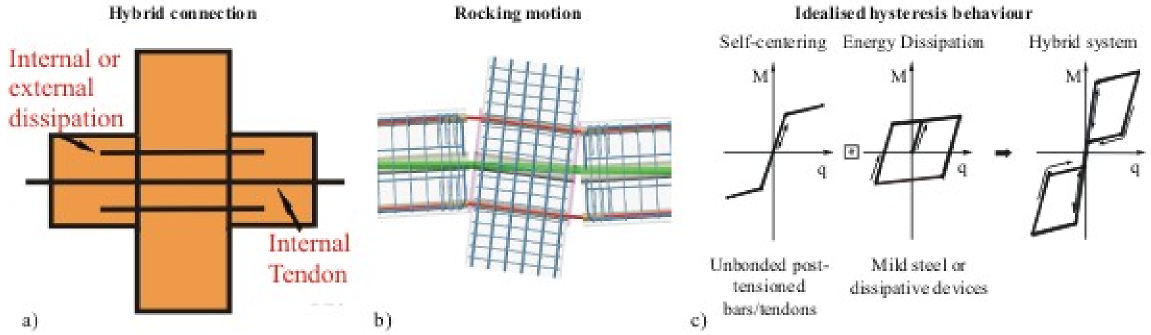


Figure 1: a) Hybrid connection; b) rocking motion mechanism (courtesy of S. Nakaki); c) idealised flag-shaped hysteresis behaviour.

The displacement-based design (Priestley 2002) method uses displacement spectra rather than the acceleration spectra. This method of design is already used widely for the design of structures and can also be utilised for the design of the Hybrid LVL system (Priestley et al. 2007).With timber’s low embodied energy and ability to act as a carbon sink, this method of construction is also highly sustainable.

This paper gives a preliminary overview on previous subassembly testing, then presents the design of a 6-storey hybrid timber framed building. The design of the members and connection details is described. The proposed construction techniques are also presented. Further to this, the costs of the building are compared to similar buildings consisting of concrete and steel structural members.

DEVELOPMENT OF LAMINATED VENEER LUMBER HYBRID SYSTEM

An extensive and ongoing experimental campaign is being carried out on beam-to-column, column-to-foundation and wall-to-foundation subassemblies for the implementation of laminated veneer lumber (LVL) hybrid solutions (Palermo et al. 2005, 2006a, b, Smith et al. 2007). The results of these tests, some of which are presented in Figure 2 below, have yielded extremely pleasing results. An extensive number of rocking connection options have been considered with internal and external attachment of the dissipation devices as well as post-tensioned only connections.

A stable “flag shaped” hysteretic loop is observed in all the cases in Figure 2 with negligible residual displacement, confirming the self-centring characteristics of the hybrid systems. The equivalent yielding point corresponds to the actual yielding of the dissipation devices, while the total moment capacity of the system increases with the increasing drift levels due to the elongation of the tendons. No degradation of stiffness and no structural damage are observed and a maximum drift level of 4.5% is achieved during all tests apart from that of the wall which was stopped due to the tendon approaching yield. This rapid increase in tendon tension will not occur in the actual assembly due to the increased length of the tendon in a real building, reducing the variation in strain.

Further to this testing, coupled and parallel wall systems have been investigated. For multiple wall systems, dissipative devices can be based on the relative motion between two adjacent walls, not the gap formed between the wall and the foundation. Different methods of dissipation have been investigated. The most effective of these was the use of U Shaped Flexural Plates (UFP), as seen in Figure 3.

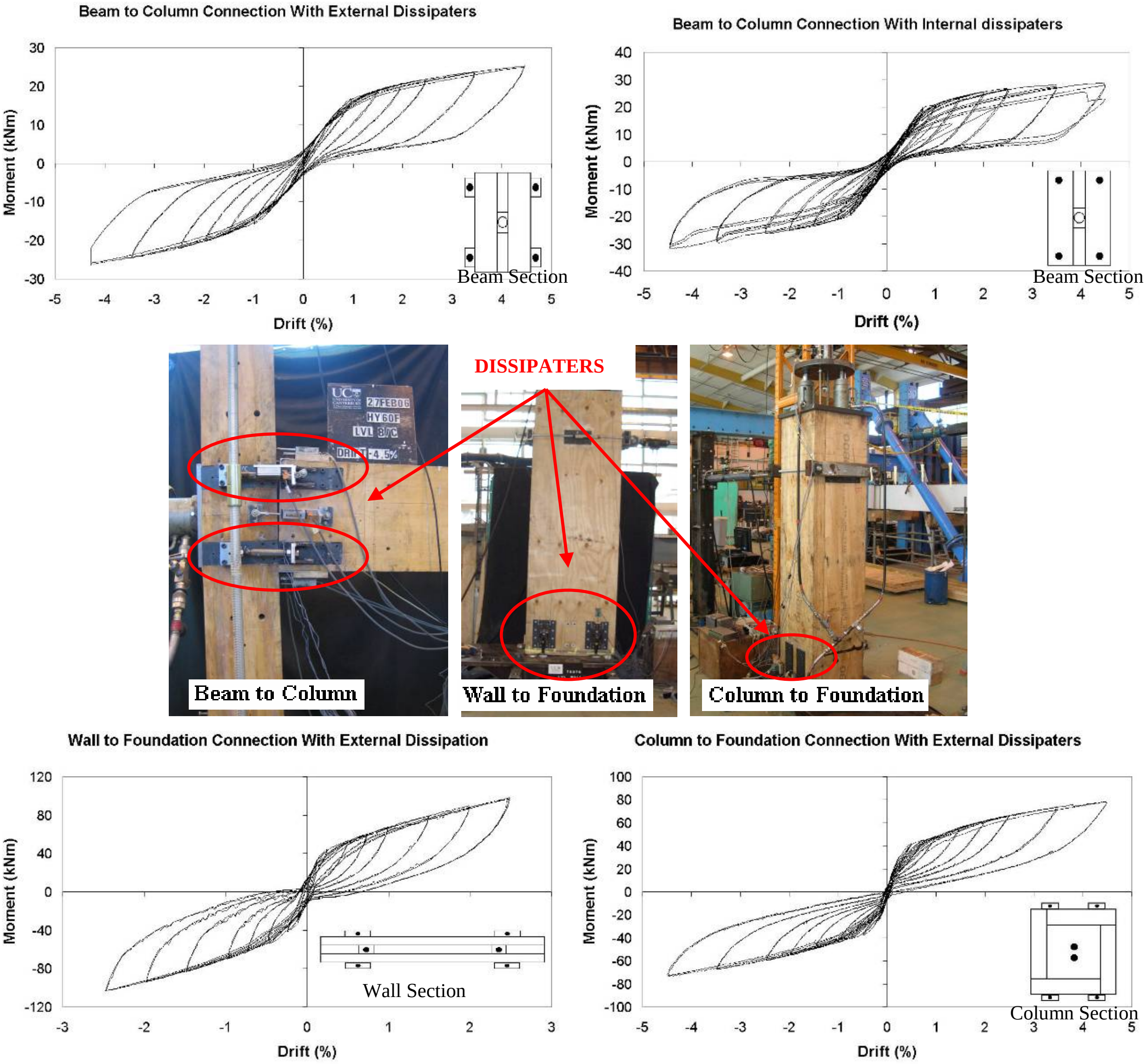


Figure 2: Hysteretic loops from previous sub-assembly testing.

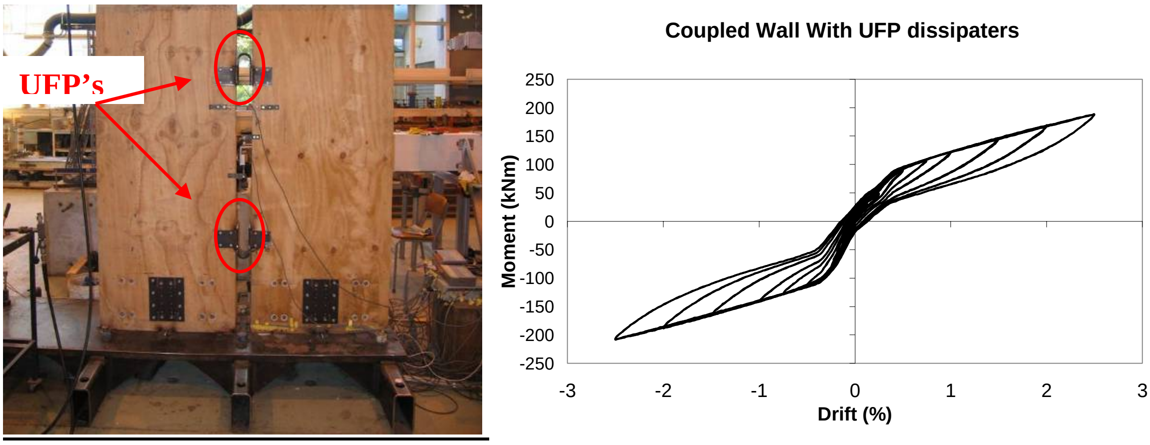
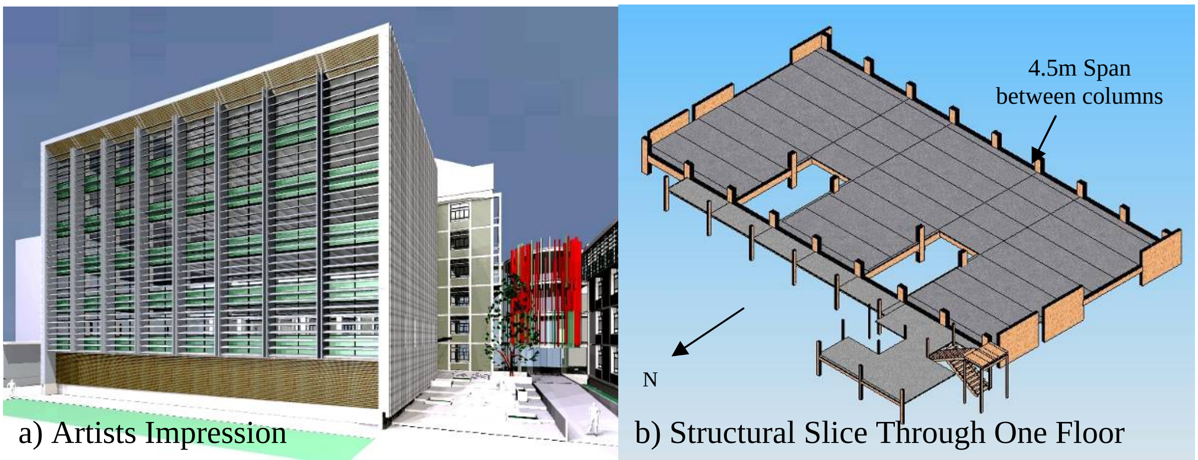


Figure 3: Result from coupled walls with UFP dissipation

SEISMIC DESIGN OF A VIRTUAL SIX STOREY BUILDING

The current feasibility study is based on a six storey reinforced concrete building that is being built at the University of Canterbury for the Biological Sciences department, the architectural rendering of which is shown in Figure 4a. A structural subassembly is shown in Figure 4b.



a) Artists Impression

b) Structural Slice Through One Floor

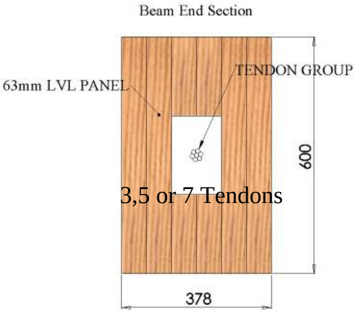
Figure 4: Biological Sciences Building

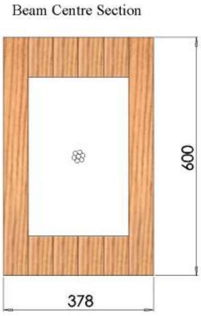
The structural system has been altered slightly from that of the original concrete structure. Seismic forces will be resisted by frames in the east-west direction and by walls in the north-south direction. The floor will span in the east to west direction and will be seated on four gravity beams which sit on central columns and exterior columns.

Member design

In order to calculate the applied lateral forces on the structure a Direct Displacement Based Design (DDBD) (Priestley 2002) approach was used. Drift limits were set at 2% inter storey drift in the frame direction and 1% in the wall direction. Conservatively, it was chosen to use an equivalent viscous damping value of 5% although it has been proven through experimental testing that values of at least 10% to 12% are realistic for a timber hybrid system. Base shear forces in both directions were calculated and distributed up the building in accordance with the assumed first mode displacement of the structure.

Internal actions were calculated using a linear elastic analysis programme. Following DDBD principles (Priestley 2002) the base shear was evenly distributed along the building and the corresponding moments were applied at the base. From these internal forces the column, beam and wall members were designed. The geometry of these members is shown in Figure 5.





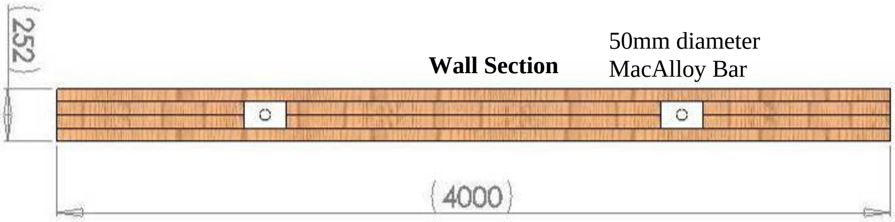


Figure 5: Structural Members for the Hybrid Timber Building

The member sizes are comparable to those of the original concrete structure, however, this was mainly due to architectural considerations. With the use of this type of system it would be possible to double the distance between the seismic columns without increasing the member size. Further, as the beam sizes are controlled by the moment demand at the end connections it is possible to remove a large portion of wood from the interior of the beam in the mid-span regions (as shown in Figure 5), reducing the amount of LVL required, thus reducing the weight and cost of the member. It is recommended that the columns and walls remain solid in order to reduce the elastic flexibility of the system. The column cross section shown in Figure 5 shows that there are no tendons required in the column member. This is due to adequate re-centring force being provided by the column's gravity loading. Due to the reduction in moment demand at the beam to column interface at the upper storeys the number of tendons required is reduced in the top levels.

Connection design

Due to the anisotropic nature of timber the connection detailing for the building has presented a challenging problem in the design of this system. In general it is desirable to load wood in compression parallel to the grain, rather than in compression perpendicular to the grain, to obtain greater strength and stiffness. This is even more important if wood is loaded in tension perpendicular to the grain, in order to prevent weak and brittle splitting failures. The following paragraphs outline some of the proposed connection details.

Timber-concrete composite floor

Presently at the University of Canterbury a new form of timber composite flooring is being developed. This consists of timber panels prefabricated off-site with 65 mm concrete topping cast in situ. The timber panels are made from two adjacent 63×400 mm LVL joists spaced at 1200 mm centres with a nailed plywood sheet. Notches cut in the joists will be filled by concrete during the concrete casting and one coach screw will be inserted at the centre of each notch to improve the behaviour. Due to the bearing of the concrete on the timber in the notches, excellent composite behaviour between the concrete topping and the LVL joists will be achieved, providing a significant increase in the stiffness of the system. The concrete topping also improves the acoustic separation between floors, as required between tenancies, and provides excellent diaphragm behaviour. For further information regarding this system refer Buchanan et al. (2008) and Yeoh et al. (2008).

Gravity connections

The transfer of gravity forces at the beam supports throughout the building will be principally in bearing. This ensures that a minimal amount of tension will occur in the connections. The floor joists will be supported on steel joist hangers which are common practice in low-rise timber construction, and have been proven easy to install and cost effective. The gravity loadings from the main floor beams will be transferred to the columns with the use of corbels as shown in Figure 6.

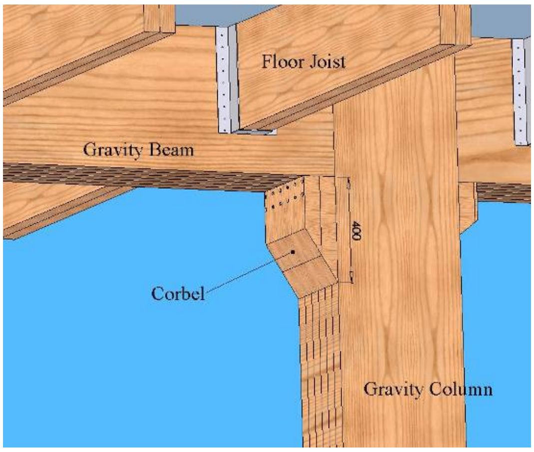


Figure 6: Corbel Connection

In-Plane Floor Shear Transfer

In a timber-concrete composite flooring system the in-plane shear due to diaphragm action will be transferred along the top of the floor in the topping concrete. It is therefore necessary to connect this flooring into the seismic resisting system. In order to achieve this, two systems were used as shown in Figure 7a) use of coach screws inserted in the side face of the beam which is then cast into the concrete topping; and b) reinforcing bars connected to fasteners in the solid wall using bolts and threaded couplers. It is assumed in the design of these couplers that the flooring will remain rigid and the connection will remain elastic during force transfer. Experimental testing has shown that the coach screw can reach a minimum of 20kN before any slip occurs (Figure 8). Once this slip does occur, the concrete fails and ductile behaviour is exhibited. Due to the larger amount of shear transfer into the walls, a form of drag bar was devised, as shown in Figure 7b.

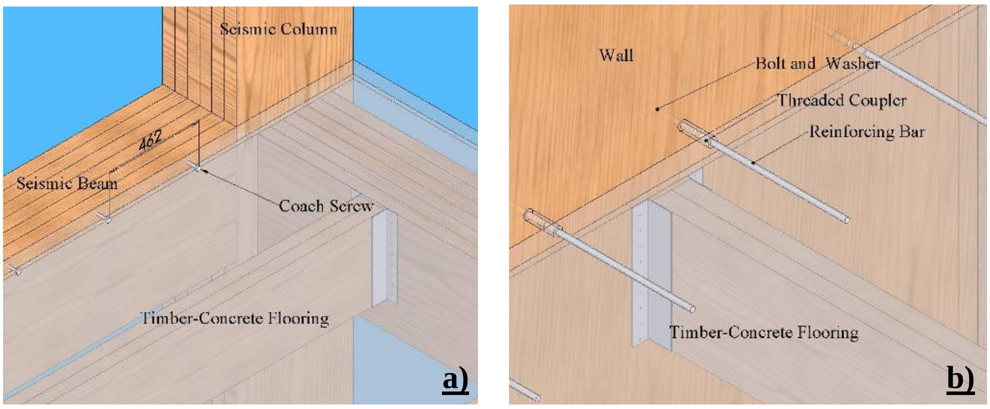
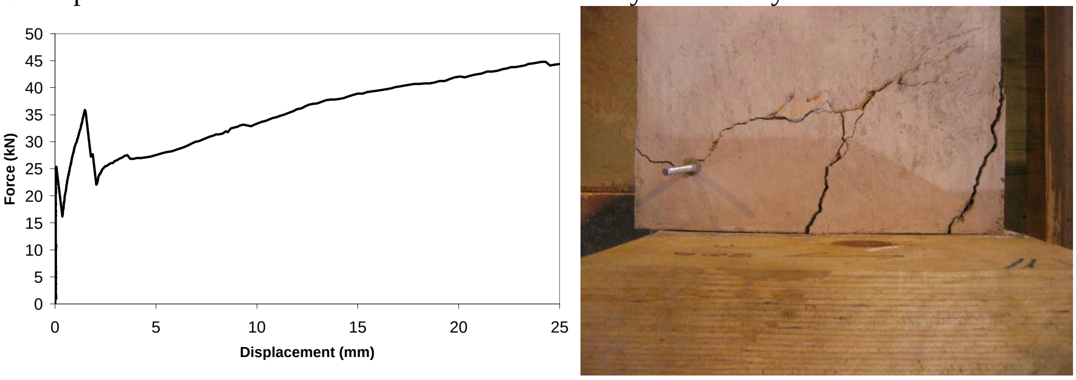


Figure 7: In-Plane Shear Transfer Mechanisms

In order to calculate the shear capacity of the drag bar connections, a modified version of Johanssen's yield theory is used. For the design it is assumed conservatively that the bearing will solely occur on the coupler section. The "rope effect" will be provided by a 50mm square washer at the end of the bolt. Given an appropriate development length of the bar inside the concrete topping, it is assumed that this will act as a fastener in a rigid medium and bearing failure in the timber will occur on the timber. It is possible to manufacture these attachments easily in a factory and assemble them on site.



a) Force displacement graph b) Plan view of test specimen

Figure 8: Shear Testing of Single Coach Screw for Diaphragm Connection

<u>Column to Foundation connection</u>

In order to reach the desired moment capacity at the base of the column, steel dissipating bars are used. These bars are 32 mm diameter, and due to the large axial forces generated when they yield, it is necessary to use an internal epoxied solution. These epoxied bars have a necked fuse length of 100mm which will provide hysteretic damping to the system during the rocking movement of the column through axial tension and compression yielding. It is suggested that this necked fuse length be cut down to approximately 80% the bar diameter, to ensure that the yielding occurs only in the fuse length, controlling the dissipation and ensuring that re-centring occurs. The epoxy around the fuse will prevent the fuse from buckling during compression cycles. The embedment length of the bars is calculated using the equations devised by Van Houtte (2003) for the axial strength of epoxied rods in LVL. To reduce the length of the required embedment, a threaded rod is used rather than a deformed reinforcing bar. The attachment of these bars into the foundation also requires careful consideration. To avoid an unnecessary increase in the required depth of the foundation, a steel 'shoe' shown in Figure 9 has been devised. This will be attached to the column base during the manufacture of the column and simply bolted to the foundation on site.

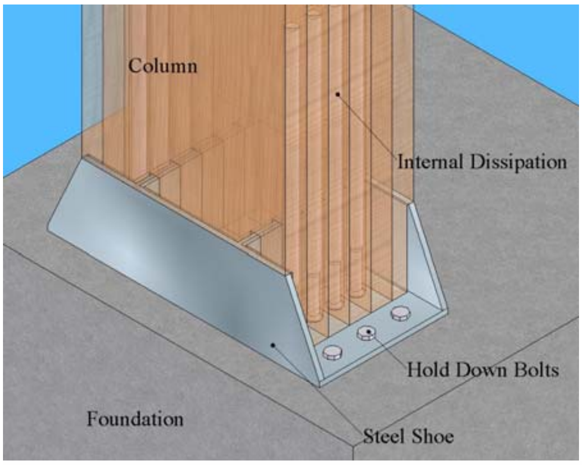


Figure 9: Column to Foundation Connection

Wall to Foundation connection

It is required that the wall to foundation connection has a considerably larger moment capacity than that at the base of the columns. To ensure that this demand is met a large amount of dissipater steel must be used and in order to ensure re-centring, two full-height 50mm MacAlloy bars are placed inside cavities in the wall. These MacAlloy bars will always remain elastic, even under a severe earthquake. A large foundation is required to resist the base moment, so the dissipation steel can be grouted directly into the foundation on site.

Construction

Various construction options are being discussed for the proposed system. Due to its modular nature and the low mass of the structural members, it will be possible for construction to be fast and inexpensive. The currently proposed construction sequence for the main structure is as follows:

- Pouring of foundation
- Erection of columns and walls for first three and a half stories in height
- Placement of beams in first three floors, stressing of beams in one operation
- Placement of timber flooring panels and pouring of concrete topping
- Exterior cladding of bottom three floors
- Erection of next three floors columns and walls
- Placement of beams in top three floors
- Stressing of beams in top three floors and stressing of walls
- Placement of timber flooring panels and pouring of concrete topping
- Construction of roof, and exterior cladding of top three floors

This sequence of construction will ensure that the cost of post-tensioning is kept to a minimum.

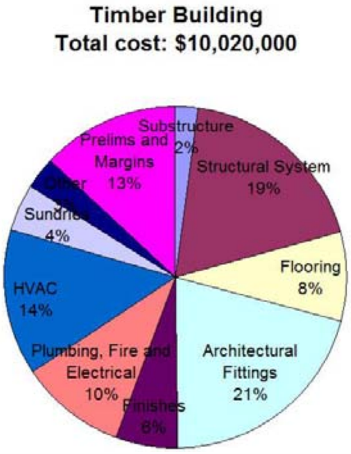


Figure 10: Cost Breakdown of Timber Building

Cost

A preliminary cost estimate for the building has been carried out comparing the timber building to steel and concrete structures that have been designed to the same seismic and architectural standard. The preliminary total cost of these buildings is shown in Figure 10 and Table 1.

Table 1: Total Cost of Building Options

	Timber	Steel	Concrete
Total Cost	$10,020,000	$9,370,000	$9,430,000

As shown above the steel and concrete building options cost approximately $500,000 less than the timber option. Although the structural timber system costs more than a steel or concrete structural system, it represents a small portion of the building's total cost, ensuring that the total cost difference is modest. This cost difference will be offset to some extent by the rapid construction time using light pre-fabricated sections on site and the sustainability benefits of building in wood, such as carbon storage. The cost of buildings like this will decrease in time as the technology matures.

CONCLUSIONS

A new and exciting method of timber construction has been presented. A six storey building design has frames in one direction and walls in the other. Post-tensioned LVL timber beams, columns and walls are of similar sizes to the original concrete design. These members and the composite timber flooring panels are pre-fabricated off site. The internal post-tensioning, together with energy dissipaters at wall and column bases, will ensure that the building has only small displacements during an earthquake, with no residual structural deformations.

Construction will be fast with costs kept to a minimum. The overall building costs are comparable to steel and concrete options. Based on testing and analysis to date, the feasibility and sustainability of the post tensioned hybrid solution is evident.

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