CROSS LAMINATED TIMBER CONSTRUCTION FOR RESISTING LATERAL LOADS ON SIX LEVEL BUILDINGS

John Chapman
School of Architecture & Planning, University of Auckland, New Zealand.
Email: jb.chapman@auckland.ac.nz

ABSTRACT

The author's intention is to contribute to the general discussion on timber multi-level commercial buildings. Interest in this topic is expected due to the environmental advantages of timber construction when compared to concrete and steel. This paper looks into three timber based systems for resisting lateral loads for buildings to six storeys, that will ensure relatively 'open' floor spaces. In this paper, three proposed lateral load resisting systems are termed 'frame', 'circular core', and 'shear walls'. Only low stresses occur in the three systems and they can be made with timber below 'structural grade' which is more economical. The concept of reinforced concrete 'socket' foundations, for returning columns to their original locations, is briefly explained. The paper considers the lateral load resisting systems from the viewpoints of structure, architecture and economics. Architecturally, the most flexible arrangement would be a 'frame' system on each external wall. It would leave the floor areas free except for internal columns; and windows can be placed within the frame construction allowing light to enter the building. Assumptions have been made, such as the deflections due to joint slippages and these will, at some stage, need to be studied and their accuracy checked.

1.0 INTRODUCTION

A worldwide interest in multistorey timber buildings is expected due to the environmental advantages of timber construction when compared to concrete and steel. Timber buildings when compared to the equivalent in concrete and steel use about 40% the energy during manufacture and are carbon neutral [1]. In Europe, during the last few years, cross-laminated timber (CLT) panels for both walls and floors have proven to be a viable form of construction. The panels are made of layers of lumber at right angles to each other [2]. The CLT system relies on walls as the main elements for supporting both vertical and horizontal loads and has been used for residential apartments up to nine storeys [3]. For commercial buildings, reliance on walls for supporting vertical gravity loads means a relatively large number of internal walls and this restricts flexibility of floor space. This paper looks into three timber based systems for resisting lateral loads for buildings to six storeys that will allow more 'open' floor spaces. In this paper, the three proposed lateral load resisting systems (LLRS) are termed 'frame', 'circular core', and 'shear walls' (Figures 1A, 1B & 1C).

Only low stresses occur in the proposed three lateral load resisting systems and they can be made with timber below 'structural grade' which is more economical. A fourth type of lateral load resisting system, 'continuous frame', could be considered. However, researchers at the University of Canterbury, in Christchurch, New Zealand, are making a major effort to look into this

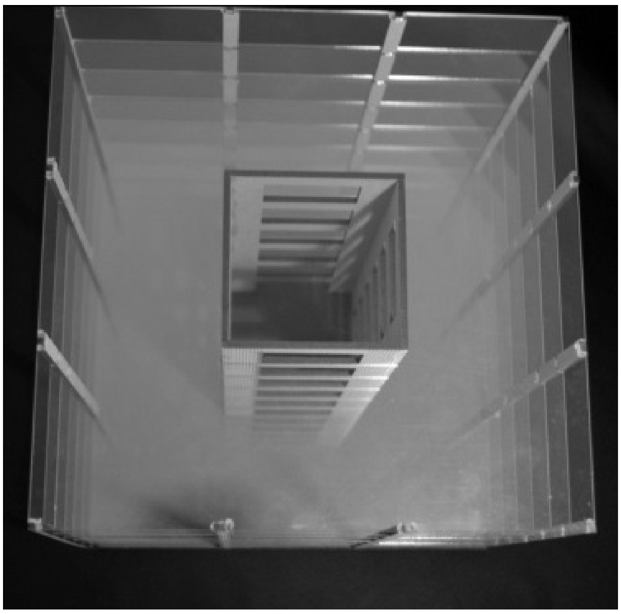


Figure 1A. Model, 'frame' LLRS

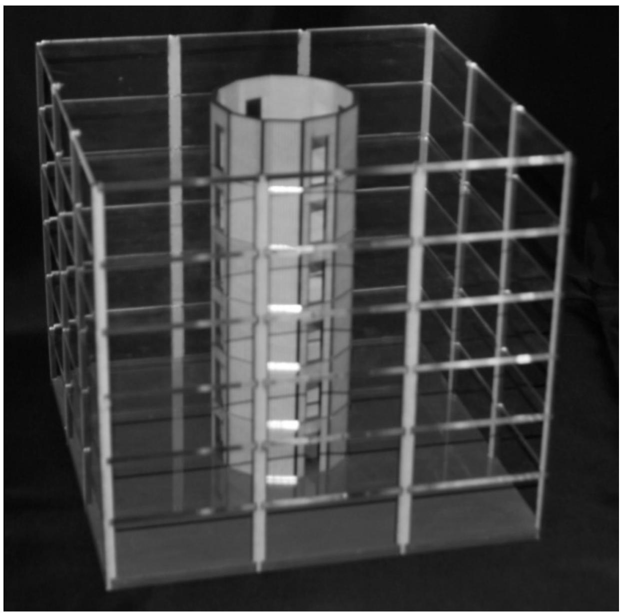


Figure 1B. Model, 'tube core' LLRS

Figure 1C. Model, 'shear walls' LLRS

possibility using laminated veneer lumber (LVL) and have constructed a prototype [4]. Commercial building structures need to support vertical gravity loads due to building self weight, people etc; and horizontal loads due to wind and earthquake events. For a six storey building with columns spaced 8.4 m apart, supporting vertical gravity loads is relatively straight forward. Timber floor beams, and timber box columns can be efficiently made with LVL. However, the internal bending moments, that a lateral load resisting system for a six storey building must support (up to 10,800 kNm), are much greater than those supported by the floor beams (520 kNm) and columns (minimal moments). These lateral load resisting system bending moments are large for timber construction to date and this paper looks at ways to achieve these structures. The paper is an overview of possible lateral load resisting systems from the viewpoints of structure, architecture and economics. Its intention is to contribute to the general discussion on the topic of multi-level timber commercial buildings.

2.0 CRITERIA, ASSUMPTIONS AND CONDITIONS FOR THE STUDY

2.1 BUILDING GEOMETRY

The floor plans are considered to be three bays long by three bays wide, with the bay spacings at 8.4 m in both directions. Allowing for external wall cladding etc, the resulting overall building widths are around 26 m. The height between floor levels is taken to be 3.25 m, resulting in an overall height for a six storey building of approximately 20 m. All of the three proposed LLRSs require timber panels to extend vertically for the full building height, i.e. to be made with a length of 20 m. For CLT construction, two panels would need to be joined together, as they are made to a maximum length of around 16 m.

2.2 WOOD FORM AND QUALITY FOR THE LLRS PANEL ELEMENTS

The members of the lateral load resisting systems are considered to be CLT. CLT is only presently available in a few European countries, but is more stable than glulam and more economical than LVL. CLT has proven to be dimensionally stable for large inter-connecting timber members; whereas there is a question mark over glulam – 'will it tend to move and split under secondary stresses, due to conditions such as temperature and humidity changes?' A building is presently being built for the Nelson Marlborough Institute of Technology in Nelson, New Zealand, that uses LVL sheets which are glued together to form shear walls [5]. However, it does not seem likely that currently in New Zealand there is the capability of pressing large LVL panels together with a pressure of between 0.55 mPa and 1.0 mPa, which is the pressure range recommended for both glulam members [6] and CLT panels [7].

The timber for the proposed lateral load resisting systems in this paper is considered to be a softwood, say pine or spruce, with a grading which is generally just below structural quality. The modulus of elasticity (E) is taken as 7000 mPa. This value is based on the E, for New Zealand grown *Pinus radiata* members in the lowest structural grade, G8, being 8000 mPa [8]. The shear modulus (G) is taken as 470 mPa based on published values [6]. Slipping within joints will take place under horizontal loadings. A proportion of the elastic deflections are used to take account of this effect and these are shown in Table 1.

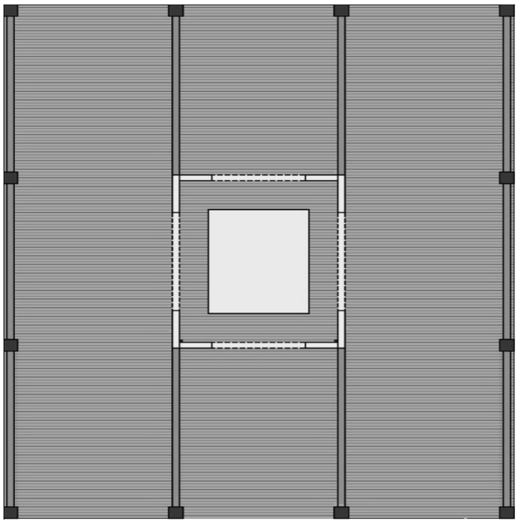
The CLT panels for the lateral load resisting system members are considered to have a 7 layer arrangement and a total thickness of 284 mm. There are 4 longitudinal layers (in the direction of the length of the panel) of 42.5 mm thickness; and 3 transverse layers of 38 mm thickness.

2.3 DESIGN LOADS & SERVICEABILITY CRITERIA

The lateral load resisting systems are designed for a static ultimate horizontal wind load (Wu) of 2 kPa, this being a typical horizontal wind loading for a commercial building in an inner city setting. According to the relevant New Zealand code, NZS1170, the associated serviceability horizontal wind load (Ws) is 1.35 kPa [9]. For towns that have low earthquake risk, the lateral load resisting systems in a design earthquake event would experience similar stresses to those produced by a wind load of around 2 kPa.

The floor construction and associated beams have not been specified, because the emphasis of this paper is on lateral load resisting systems. A typical floor layout is shown in Figure 2. The floor construction spans across a bay width and is supported by floor beams.

At the University of Auckland, the Acoustic Testing Service, in association with Scion, has had the experience of building and testing around 27 timber

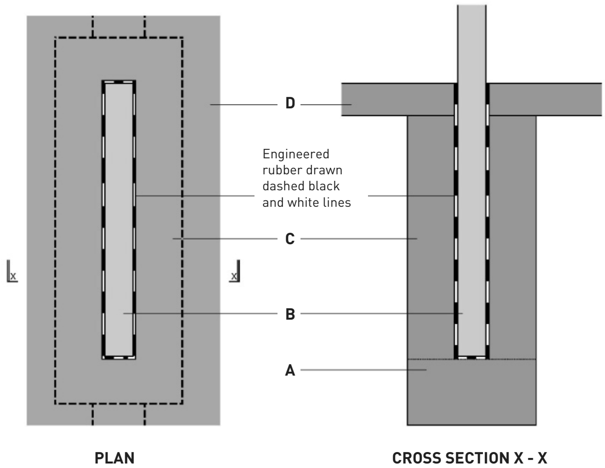


Plan Notes:

This plan is for the 'frames' LLRS with four frames (drawn light colour), creating a central core. Inside the frames is drawn a lift/stair shaft (white rectangle at centre) surrounded by four narrow floor areas. The floor joists are shown spanning across the page into floor beams (direction up page). The beams are supported either by the frames or timber columns at external walls.

Perhaps the lift/stair shaft can be arranged so the remaining space within the core can be used for toilets and common facilities.

Figure 2. Typical floor plan (diagrammatic)



Order of Construction

A. Pour reinforced concrete lower foundation, with reinforcing bars protruding to link to the side foundations.

B. Place timber column, with engineering rubber attached.

C. Place side and end reinforced concrete foundations.

D. Pour reinforced concrete slab.

Figure 3. Socket Foundation (diagrammatic)

floors for acoustic performance. A finding of this study was that floor mass is important for resisting inter-storey airborne sound transference. The dead load for the most recently developed floor was 2.5 kPa, and this dead load was used for the stress analyses of the proposed three systems [10].

The maximum allowable inter-storey drift, under serviceability wind loads, is taken as 0.002 times storey height, which calculates out to be 6.5 mm for the assumed 3.25 m inter-floor height [9]. An elastic analysis of each LLRS was carried out in the 'Multiframe' structural analysis program [11].

2.4 FOUNDATIONS FOR THE LATERAL LOAD RESISTING SYSTEMS

All three systems require fixity where the columns meet the foundations, to ensure that the deflection criteria are met.

It is advantageous if a building structure is designed to return to its original position after an earthquake event. The obvious advantages are that the lateral load resisting system can continue to resist further ground shaking and that costs due to damage to the building will be minimised.

A simple solution to maintain the original location of lateral load resisting systems after earthquake events might be to sit the timber columns in rubber, or perhaps plastic, lined reinforced concrete 'socket' foundations (Figure 3). The bottom and side parts of the reinforced concrete 'socket' foundation are all well connected together with reinforcing bars, but the reinforced concrete is not connected to the timber panels. There is no uplift on the columns and the 'socket' foundations only need to support translation and base rotation of the panel. Yielding in rubber linings will absorb earthquake energy. The reinforced concrete foundation can easily be arranged to be 'waterproof' to keep the timber dry. Leak-proof reinforced concrete construction is often required for swimming pools and basements below ground. Also, the seams of the rubber layer could be welded together like a swimming pool liner to resist water ingress. The 'socket' foundations would be linked into the adjacent footings to ensure base overturning is resisted.

Other systems that have been recently under development to maintain lateral load resisting systems in their original positions after earthquake events, include the University of Canterbury initiative for LVL continuous frames that '....combines un-bonded, post-tensioning and additional energy dissipaters, providing a recentering capability after the earthquake, while greatly reducing the structural damage.' [4]

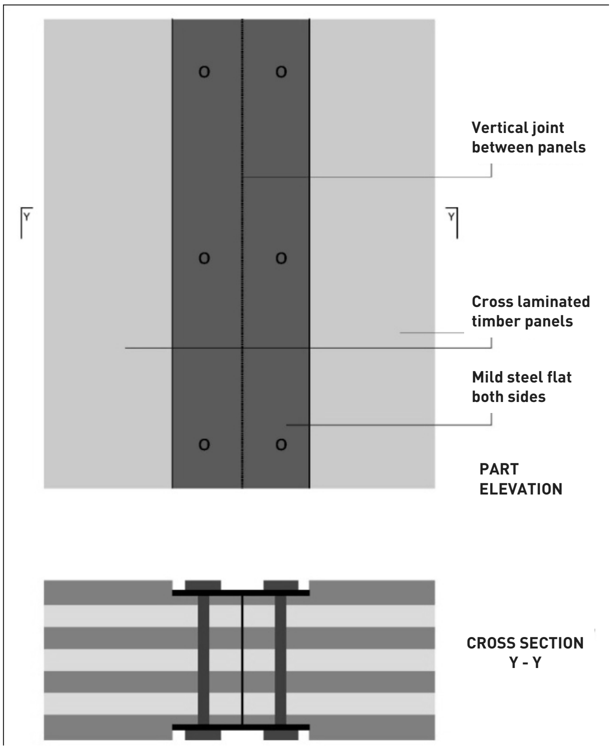


Figure 4. **Panel shear joint, with m.s. flat plates. (diagrammatic)**

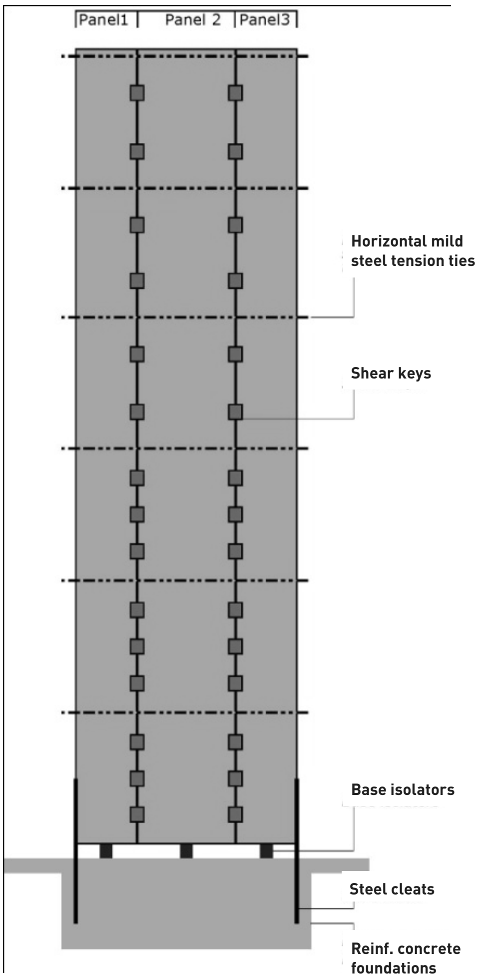


Figure 5. **Elevation of 'shear wall' LLRS (diagrammatic)**

2.5 SHEAR TRANSFER BETWEEN PANELS

One way to transfer the shear between the panels is to fix steel flats on both sides of the vertical joint (Figure 4). Jointing for shear transfer between the large timber panels could possibly be achieved by rectangular or square shear keys (Figure 5). The shear keys could be made in a variety of ways, with steel, timber or possibly reinforced concrete. Timber shear keys could be made with CLT but with the laminate directions being on the diagonals so the laminates act in direct compression. To minimise slipping within the shear joints leading to excessive deflections, packing should be placed between the sides of the keys and the panel timber. There are tension forces between the large timber panels, associated with the shear keys. These can be resisted using horizontal steel rods within the floor construction space at each floor level (Figure 5).

3.0 DESCRIPTIONS OF THE THREE LATERAL LOAD RESISTING SYSTEMS

3.1 'FRAME' LATERAL LOAD RESISTING SYSTEM

The 'frame' lateral load resisting system comprises four frames which are arranged to form a vertical rectangular shaft (Figure 1A). Each frame has two columns with beams at all floor levels, plus a beam at roof level. The sizes used for the frame members are chosen to ensure that the inter-storey deflections are less than 0.002 times inter-storey height. It is envisaged that the lift(s) and perhaps the stairs would be located within the 'shaft' space that is defined by the four frames.

Between the columns and beams at each floor level, jointing is required for transferring bending moments and shear forces. The maximum moment in the column to beam joint is approximately 450 kNm. The associated tension and compression forces to be transferred between the columns and beams will be approximately 500 kN.

The frames are analysed as if they work independently of each other. However, the outer edge of each frame column is connected to the column of a frame at right angles to it. As a consequence, each column acts, to some extent, as a combined 'L' section. Thus, the columns are stiffer than assumed in the elastic analysis, resulting in less deflection than calculated. To compensate for this stiffness increase which is unaccounted for in the structural analysis, the slippage in the joints is considered as zero.

3.2 'CIRCULAR CORE' LATERAL LOAD RESISTING SYSTEM

The 'circular core' is made of large vertical timber panels that, like staves of a barrel, are assembled together to form a vertical circular tube (Figure 1B). The 'tube' diameter is taken as 8.4 m to fit in with the bay spacings and also, to provide an internal void that is architecturally useful for landings and stairs. Door openings would be needed in the panels at each floor level for people moving between the inside and outside of the 'circular core'. In Figure 1B, the 'circular tube' form is considered as twelve vertical panels that are around 2.2 m wide. The large vertical panels are joined together to transfer the shear forces between them; and the tension forces between the panels can be resisted at each floor level within the space of the floor constructions, similar to the steel bands around a barrel.

The deflections due to joint slippage and door openings in panels, as included in Table 1, are considered to be equivalent to the deflections deduced by the elastic analysis.

3.3 'SHEAR WALLS' LATERAL LOAD RESISTING SYSTEM

The 'shear walls' lateral load resisting system is illustrated with a shear wall in each external wall that is 8.4 m long to fit in with the bay spacings (Figure 1C). Each shear wall would probably comprise of 3 panels of 2.8 m width that are shear jointed to form one monolithic panel. Like the tube above, the deflections due to joint slippage and openings are considered to be equivalent to the elastic analysis deflections.

4.0 LLRS STRESSES AND INTER-STOREY DEFLECTIONS

The resulting stresses of the lateral load resisting systems are summarized in Table 1. The maximum resulting stresses for bending moments and axial forces are low for the 'tube core' and 'shear walls' arrangements. For the 'frame' system, the resulting bending plus axial stresses for the panels appear relatively high for typical CLT construction. The maximum stresses in the columns are 16.1 mPa, but this reduces with height and becomes 7.0 mPa by level 3. The ends of the beams have relatively high stresses at all floor levels.

However, as these high beam stresses are at the ends of the beams, they will not actually occur. This is because steel cleats are screwed to the ends of the beams to transfer the beam to column bending moments. The true maximum beam stress will be where the steel cleats commence on the beams, at about 1 m from the ends of the beams, and will amount to approximately 7.6 mPa. If steel cleats are used at the base of the columns, the maximum column stresses will also be considerably reduced and will be at a maximum at the level of the top of the cleats. Thus, for the 'tube core' and 'shear walls' systems, the resulting bending plus axial stresses are within the range typically found in CLT construction and timber below 'structural grade', which is more economical, can be used extensively in them. The stresses in the 'frame' system are slightly higher and more structural wood will be needed in the CLT members.

The 'tube' deflections are minimal with the maximum inter-storey deflection being 1.6 mm for the design serviceability wind load. Also, the maximum stresses, which occur at the base of the tube, are small at around

Table 1. Lateral Load Resisting Systems - Panel Sizes, Stresses, & Inter-storey Deflections.

LLRS Type	**Frames**		**Tube Core**	**Shear Walls**
Panel Type	Column	Beam	Vertical	Vertical
Panel Length *(m)*	20	6.6	20	20
Panel Width *(m)*	1.8	1.04	2.2	2.8
Panel Thickness *(mm)*	284	284	284	284
No of panels per building	8	24	12	12
Max Compression Stress [1.] *(mPa)*	16.1	14.6	2.6	4.1
Max Tension Stress [1.] *(mPa)*	5.1	14.0	0.0	1.4
Max. Shear Stress *(N/sq.mm)*	0.9	1.3	0.4	0.34
Shear Stress Between Laminates [2.] *(mPa)*	1.3	1.9	0.9	0.9
Max Inter-storey Deflection, Elastic *(mm)*	6.3	-	0.65	1.4
Max Inter-storey Deflection, Joint Slippage *(mm)*	0	-	0.65	1.4
Max Inter-storey Deflection, Total *(mm)*	6.3	-	1.3	2.8

1. **combined bending and axial stresses due to critical load case, 1.2G+Qu+Wu**
2. **maximum shear stress in glue line between longitudinal laminates.**

2.6 mPa. The proposed 'tube' system will be able to withstand considerably larger wind loads than those which are assumed to be acting for a typical inner city environment. For the same wind loads as used in the analyses, the 'tube' structure can extend to nine storeys, when the maximum inter-storey deflection (elastic plus joint slippage) becomes 6 mm and the maximum stress is 5.7 mPa. The 'tube core' form is structurally efficient.

The 'shear walls' system stresses and inter-storey deflections are also minimal, and the walls could support larger wind forces than considered by the analyses. For the same wind loads as used in the analyses, the 'shear wall' system could be extended to eight storeys where the inter-storey deflections, elastic plus joint slippage, reaches a maximum of 6.3 mm and maximum stress is 6.5 mPa.

5.0 VOLUMES OF TIMBER AND COSTS

Because the resulting stresses in the three proposed lateral load resisting systems are relatively low, timber that is considered just below structural grade can be used extensively in their panel elements. This will save cost. For example, structural grade *Pinus radiata* lumber in New Zealand with boron preservative treatment costs around USD 400/m^3 and non-structural grade radiata lumber costs around USD200/m^3.

The volumes of timber needed for each of the three systems is shown in Table 2 with an approximate cost per m^2 (of floor area) for their construction. There is little information for this type of manufacturing with New Zealand grown *Pinus radiata*, and the costings below have been estimated using the New Zealand Building Economist [12]. The motive of building developers is currently profit and it is important for timber commercial building systems to be as cost effective as possible if they hope to compete with those of concrete and steel. The cost of timber in place per m^3 is highest for the 'frame ' system because more structural grade timber is needed and there are 48 joints between the beams and columns which are needed to transfer both bending moments and shear forces. The in-place timber cost per m^3 for the 'shear walls' system is considered the least because the timber can be largely below structural grade and the number of shear joints required is less than for the 'tube core' system. It appears that the lateral load resisting systems costs, including surfaces finishes, would be around USD120/m^2 (NZD160/m^2). The finishing cost per m^2 of wall area is taken as USD40/m^2 (NZD55/m^2) due to 16 mm fire resistant plasterboard and three paint coats.

6.0 FORM & ARCHITECTURE

The three lateral load resisting systems are described in this paper in their most basic forms. The 'frame' system is shown with four frames forming a central core. These frames could be located elsewhere in the building, like one within each external wall. The 'tube core' system relies on all parts working together and the form cannot be modified, except to vary the diameter and replace the circular shape with an ellipse. Like the inside of a castle keep, the inside of the tube core could be kept relatively empty and, perhaps with a glass roof and planting at bottom floor level, could feel like a quiet sanctuary and be useful in occasional situations. The proposed 'shear walls' system requires an 8.4 m long wall on each external wall. These walls are an impediment to light entering the building and could appear bland. Limited penetrations could pierce the walls; or the two walls in each direction could be replaced by a greater number of

Table 2. Lateral Load Resisting Systems, Timber Volumes and Approximate Costs / m^2.

LLRS Type	Frames	Tube Core	Shear Walls
Timber Volume *(m^3)*	129	150	191
Timber Cost in Place, *(USD/ m^3)*	3000	2600	2200
Timber Total Cost *(USD)*	385,700	389,900	419,900
Building Area *(m^2)*	4056	4056	4056
Timber Cost / m^2 of floor area *(USD)*	95	96	104
Surface area of LLRS members *(m^2)*	1090	1190	1480
Finishing Cost / m^2 *(USD)*	40	40	40
Finishing Total Cost *(USD)*	43,500	47,700	59,200
Finishing Cost / m^2 of floor area *(USD)*	11	12	15
LLRS Cost / m^2 of floor area *(USD)*	106	108	119

shorter walls. For example, eight walls of 5.2 m length would be suitable. Also, frames in one direction and shear walls in the other could be used.

All three of the proposed lateral load resisting systems have limitations with respect to architecture. The most flexible arrangement would be a 'frame' system on each external wall. It would leave the floor areas free except for internal columns. Also, windows can be placed within the frame construction allowing light to enter the building. For frames on external walls, the beams could be deeper extending up to the underside of the windows of the storey above. These deeper beams would help frame stiffness.

7.0 CONCLUSIONS

A worldwide interest in multi-storey timber buildings is expected due to the environmental advantages of timber construction when compared to concrete and steel. This paper looks into three timber based systems for resisting lateral loads, on buildings to six storeys, in an attempt to achieve 'open' floor spaces. The lateral load resisting systems are termed 'frame', 'circular core', and 'shear walls'. They have been designed to withstand a typical wind load for an inner city environment. Only relatively low stresses occur in the proposed three systems and they can be built with timber that is mainly below 'structural grade' which is more economical.

Reinforced concrete 'socket' foundations may be a simple solution in assisting lateral load resisting systems' columns to maintain their original locations after earthquake events. 'Socket' foundations prevent the bases from overturning, but ensure a degree of energy absorption due to yielding in the column base tiedowns and in the 'engineering rubber' lining between the column and 'socket'. The 'circular core' form is an efficient system and would be suitable for buildings that are taller than six storeys, perhaps useful to nine stories.

Architecturally, the most flexible arrangement would be a 'frame' system on each external wall. It would leave the floor areas free except for internal columns; and windows can be placed within the frame construction allowing light to enter the building.

The paper is an overview of possible lateral load resisting systems from the viewpoints of structure, architecture and economics. Assumptions have been made, such as the deflections due to joint slippage being around the same as the elastic deflections, and these assumptions will at some stage be researched and made more accurate. The paper's intention is to contribute to the general discussion on the topic of timber multi-level commercial buildings.

GLOSSARY

- *Carbon neutral,* wood decay or burning will return the same amount of carbon to the atmosphere that was originally absorbed during the growth process.
- *E, modulus of elasticity,* the ratio of longitudinal stress to the longitudinal strain.
- *G, shear modulus,* the ratio of shear stress to the shear strain.
- *Glulam,* timber product composed of several layers of timber laminates, of alternating vertical and horizontal orientation, glued together.
- *Inter-storey drift,* is the difference between adjacent horizontal floor displacements during wind or earthquake events.
- *kN, kilonewton,* a force, 1 kN is approximately 102 kg.
- *kPa, kilopascal,* unit of distributed load, 1 kPa = 1 kN/m^2.
- *LVL or Laminated veneer lumber,* product that uses multiple layers of thin wood that are glued together under pressure
- *mPa, megapascal,* unit of stress, 1 mPa = 1 N/mm^2.

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FEEDBACK REQUEST

The writer welcomes any feedback, at jb.chapman@auckland.ac.nz, on the above ideas or assertions which would be most helpful for further progress in understanding timber commercial buildings.

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