A 30 LEVEL CROSS LAMINATED TIMBER BUILDING SYSTEM AND ANALYSIS OF THE EUROCODE DYNAMIC WIND LOADS

John Chapman¹, Thomas Reynolds², Richard Harris³

ABSTRACT: This paper looks at the possibility of using cross-laminated timber construction, which is commonly referred to as CLT, to support commercial buildings to thirty levels. A prototype building structure is proposed and analysed using an elastic analysis program. The main structural elements, including a central tube core, columns and beams, are made of CLT. The CLT panels are arranged to ensure structural efficiency with minimal material wastage. A building of this height has the potential to undergo significant vibration in response to turbulent wind loading. The Eurocode analysis procedure for along-wind response of structures is applied to the prototype building, and the effect of the vibration on the comfort of building occupants and the magnitude of the loads on the structure is considered. The paper concludes that the proposed structural system with CLT elements is likely to be suitable for buildings to thirty levels.

KEYWORDS: Instructions to authors, Proceedings, WCTE 2012

1 INTRODUCTION

A worldwide interest in multi-storey timber buildings is expected due to the environmental advantages of timber construction when compared to concrete and steel. Cross-laminated Timber, or CLT, is a recent development that glues and clamps timber planks in alternate layers to form large panels. The cross-laminating ensures reliable strength and stability. CLT construction has been used successfully for the nine storey Murray Grove Stadthaus building in East Central London which is regarded as the world’s tallest timber residential building. Waugh, Wells and Linegar write [1]:

‘In our paper, we will use our Stadthaus scheme to demonstrate that solid timber construction is a financially viable, environmentally sustainable and beautiful replacement for concrete and steel in high-density housing. Constructed entirely from cross-laminated timber from the first floor upward...The nine-storey building is the first of this height to construct load bearing walls, floors and cores entirely from timber.’

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Figure 1: Left image: Proposed structural system for 30 storey timber building with CLT circular core. 4no (of 16) frames shown for clarity.

In this paper a prototype building is proposed with CLT for the central core, floor beams, and columns. The columns are located on lines radiating out from the middle of the core giving the building an organic feel. The structural system, with appropriate wind forces applied for a UK city location, is elastically analysed and the results are reported below. The aim of the structural system is to arrange the CLT panels for optimum strength, stiffness, and economy. In comparison with conventional structural systems for tall buildings, which use concrete in floors and structural
cores, this structural form is relatively light weight. Timber also has a low stiffness to strength ratio compared with steel and reinforced concrete, meaning that a structure designed for a given failure load in timber is more flexible than the steel or concrete equivalent. The light weight and flexibility of this structural form potentially makes dynamic effects important, and the along-wind vibration due to turbulent wind loading has been assessed according to the Eurocode 1 [reference] analysis method.

**Figure 2**: Right image: Floor Structure Plan, A - central CLT core, B - RC hoop beam, C - floor beams, D - columns

2 PROPOSED 30 LEVEL CROSS LAMINATED TIMBER BUILDING SYSTEM

To investigate the structural system a 30 level prototype building was developed using an elastic analysis program. The main structural element is a central timber tube made of CLT panels. At each level, pairs of CLT floor beams radiate out from the centre of the building, as shown in figure 2. These beams are supported at their outer ends by vertical CLT columns and at their inner ends by the CLT tube core. The building is considered to be square in plan with 42m sides and with rounded corners. The secondary beams and columns around the building perimeter which support gravity loads are not considered in this paper. The vertical distance between adjacent floors is taken to be 3.2m, resulting in an overall building height of around 97m. The circular central core and vertical columns will be boundary conditions for the architecture. However, they result in considerably more open spaces than existing CLT multi-level buildings which rely on multiple shear walls. The tubular space inside the central core could be used for vertical circulation of people and services.

CLT panels are chosen from the KLH UK Engineering Brochure and these are summarised in Table 1 [2]. The 320mm measurement in Table 1 is an average total panel thickness; and the average thickness of the longitudinal laminates is 240mm. Timber wastage, at less than 5%, only occurs in the core panels due to door openings, floor beam slots, shear key cavities, and edge shaping.

<table>
<thead>
<tr>
<th>Location</th>
<th>Core</th>
<th>Columns</th>
<th>Beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>96</td>
<td>96</td>
<td>960</td>
</tr>
<tr>
<td>Length, (m)</td>
<td>16</td>
<td>16</td>
<td>16.5</td>
</tr>
<tr>
<td>Depth, (m)</td>
<td>3</td>
<td>3</td>
<td>0.5</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>320</td>
<td>320</td>
<td>320</td>
</tr>
</tbody>
</table>

For edge loads on the panels, the longitudinal laminates are assumed to act as a beam with a depth that is the full depth of the panel. This assumes that the laminates are edge glued. On-line information of the CLT manufacturers indicates that some factories do not edge glue their panels and others appear that they might edge glue panels. Alternatively, to achieve beams with depths that are effectively the panel depths, the panels could contain diagonal laminates at 45 degrees. These diagonal laminates would behave like web members of a truss. Another option is for the CLT members to be replaced by glulam elements.

**Figure 3**: Part elevation of central core (shown in one plane). E - RC hoop beam (drawn shaded), F - RC shear connector, G - horizontal butt joins between CLT panels, H - door opening. Arrows show how RC shear connectors transfer tension from CLT panel 'X' to adjacent panels 'Y' & 'Z'
2.1 CENTRAL CLT TUBE CORE

The proposed central CLT tube core has sixteen 3m wide facets, is 96m tall, and has an outer diameter of 15.2m. CLT is especially suited to the close fitting facets of the central core because it will remain stable. To aid the predictability of structural performance, the door openings in the central core are stacked on top of one another and occur at every 3rd or 4th facet as shown in Figure 3. Traditional wine barrels have transverse metal hoops which hold the longitudinal timber staves in place. For the 30 level CLT concept structure, the steel hoops are replaced by 1.2m deep by 400mm wide reinforced concrete beams that go around the CLT facets at each floor level.

To ensure that the panels of the 15.2m diameter central tube act in unity as one structural element, vertical shear forces need to be transferred between the vertical joints of adjacent panels. The solution shown in Figures 3 & 4 describes reinforced concrete shear keys between the core facets. Dry aggregate and reinforcing, that are both enclosed in an open weave bag, are placed in the shear key cavity. Boxing is then placed and cementitious grout is pumped into the key cavity. Pumped wet grout will ensure complete filling of the shear joint cavities and hence minimal joint slip. The floor beams are 0.75m deep and are CLT-concrete composite as shown in Figure 5. The pairs of CLT elements are 500mm deep * 320mm wide and the concrete floor topping thickness is increased above them to 250mm. The shear connection between the CLT elements and the RC topping is via a steel ‘C’ section that is glued and screwed to the top of the CLT. Gerber et al show that, when these steel elements are glued to the timber, the combined sections exhibit planar behaviour with negligible ‘slip’ between the timber and concrete layers [4]. The inner ends of the beams are supported by the CLT central core and then cantilever to support floor inside the core. The outer beam ends are fixed to the CLT columns. At the beam to column joints, the steel reinforcing in the thickened concrete slab is designed to yield at the maximum design moment to encourage energy absorption and, also, to prevent excessive bending moments being transferred into the columns.

2.2 CLT BEAMS AND COLUMNS

The floors span between the radial floor beams and are considered to be of timber-concrete composite construction. The concrete floor toppings, which are typically around 75mm thick and reinforced with steel mesh, provide useful diaphragm action in the floor planes. Timber-concrete floor arrangements have been developed in Europe and more recently in Australia and New Zealand [3].

There are sixteen 3m deep columns extending the full height of the building. For economy, they reduce in overall thickness from 480mm to 240mm with building height.

3 STRUCTURAL ANALYSIS

The structure was elastically analysed by the ‘Multiframe 4D’ program by Formation Design Systems [5]. The columns and central CLT core are fixed to the foundations. To compensate for reduced central core stiffness due to door openings through the core, the core modulus of elasticity was lessened from 12,000MPa, as suggested by KLH UK, to 10,400MPa. The floor beams are conservatively assumed to be pinned to the CLT central core.

3.1 LOADS ON PROTOTYPE BUILDING

Eurocode 1 is used for determining the building loads on the prototype building [6]. The building dead load, G, is taken as 2.5KN/m2 of floor area. Research at the University of Auckland has shown that a floor mass of at least this value is needed to achieve sufficient acoustic insulation between adjacent floor levels [7]. The floor live load, Q, is considered to be 3.0 KN/m2. The wind forces, W, on the building are based on a fundamental value of basic wind speed of 23m/s and a site altitude of 100m which is suitable for most large UK cities. The combined load cases for strength used in the elastic analysis are 1.35G+1.5Q+0.9W and 1.35G + 1.5W. W is the load used for assessing horizontal building sway.
Table 2: Member Properties and Critical Actions

<table>
<thead>
<tr>
<th>Member</th>
<th>Core</th>
<th>Columns</th>
<th>Beam</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>E (MPa)</td>
<td>10,400</td>
<td>12,000</td>
</tr>
<tr>
<td>BM Stress, $f_{max}$ (MPa)</td>
<td>23</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>Max BM, $M^*$ (kN.m)</td>
<td>187,135</td>
<td>432</td>
<td>514</td>
</tr>
<tr>
<td>BM Strength, $\phi M_n$ (kN.m)</td>
<td>1,154,316</td>
<td>12,420</td>
<td>1357</td>
</tr>
<tr>
<td>C Stress, $f_{c0}$ (MPa)</td>
<td>24</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Max C, $N^*$ (KN)</td>
<td>146,130</td>
<td>14,491</td>
<td>Negl.</td>
</tr>
<tr>
<td>C Strength, $\phi N_{nc}$ (KN)</td>
<td>328,033</td>
<td>25,920</td>
<td>9123</td>
</tr>
<tr>
<td>Factor of Safety</td>
<td>1.65</td>
<td>1.35</td>
<td>2.64</td>
</tr>
</tbody>
</table>

### 3.2 CRITICAL MEMBER ACTIONS

Table 2 presents the critical member actions for the combined load cases, the maximum allowable member actions based on a strength reduction factor ($\phi$) of 0.9, and factors of safety. As expected tension occurs in the beams but tension does not develop in the core or columns. This means that all joints are relatively simple and only transfer compression and shear. The Factors of Safety in the table are calculated using the formula $1/((M^*/\phi M_n) + (N^*/\phi N_{nc}))$. The central core and column factors of safety are reasonable as they are due to building forces that have been increased by around 35% and nominal member strengths that have been reduced by 10%. For example, in table 2, the factor of safety for the level 1 to 10 columns is 1.35. However, the factor of safety when the true loads and member strengths are used is around 2.0.

### 3.3 BUILDING SWAY

The top of the building moves 70mm horizontally under the wind forces, or 0.00073 times roof level height, 96m. The maximum inter-storey sway is 3.2mm, which is the inter-storey height * 0.001. This inter-storey deflection is 50% of the suggested maximum allowable value in AS/NZS1170:2002 [8]. However, there will be some additional inter-storey sway due to joint slippage that has not been accounted for in the elastic analysis. The timber member joints are all in direct compression which forms stiffer joints than those that rely on multiple screw or nail fixings.

![Figure 7: Vertical Section through Column](image.png)

A - central CLT core, B - RC hoop beam, C - CLT beam element, D - column, M - concrete slab thickening, 250mm deep * 1m wide, with reinforcing steel, P - reinforcing steel for negative beam moments, R - corbel. Mild steel channel not shown for clarity. Two arrows show how moments are transferred between the beam and column.

### 4 JOINTS

The proposed jointing systems in this paper have members butting together and do not rely on fixings like nails or bolts. Butt joints are less likely to have internal slip than joints with fixings. The beam to column connection is shown in figure 7.

#### 4.1 RC SHEAR KEYS IN CENTRAL CORE

Due to horizontal shear forces in the central core, vertical shear forces are generated between the CLT facets of the central core. The suggested method for transferring these vertical shear forces is via reinforced concrete shear keys as shown in figures 3 & 4. The maximum horizontal shear that occurs in the central core is 7,135KN, and the factor of safety of the shear keys is 2.4. In theory, no tension will exist in the columns or central core. If tension stresses did arise in the core, then a means would be necessary to transfer them where CLT panels butt join to their neighbouring panel below. However, the RC shear keys will transfer panel tension stresses around these butt joints by transferring them through adjacent panels to the left and right as illustrated.
using arrows on Figure 3. The maximum capacity of the shear keys to transfer tension from a panel to its neighbouring panel below is 6,900KN. Fortunately, tension stresses are most likely to occur near the Y-Y axis. Thus, they will not reduce the ability of the shear keys transferring vertical shear forces which occur around the X-X axis.

4.2 ALTERNATIVE ‘ZIGZAG’ SHEAR TRANSFER SYSTEM FOR CENTRAL CORE

Figure 8: Elevation of alternative shear connector for central core. K - CLT central core panel, W – structural grout between CLT panels, 25mm approx. thick.

If for the RC shear key described above, small gaps occur between the concrete and the wood, surprisingly large increases in horizontal deflection of the building during horizontal load events may result. These gaps could arise for a variety of reasons, such as concrete shrinkage, different rates of thermal expansion of the concrete and timber, creep, and varying moisture conditions. An alternative ‘zigzag’ shear transfer system for the vertical joints of the central core panels is shown on Figures 8 & 9. The sides of the CLT panels are shaped to form ‘teeth’ and they mesh with the ‘teeth’ of the adjacent panels. The approximately 25mm wide gap between the panels is filled with a high strength but low shrinkage grout. This system would require more CLT panels for the central core as the width of each panel effectively reduces from 3m to around 2.6m. However, it is simpler and less likely to lead to joint slip.

4.3 CLT BASE CONNECTION

A proposal for connecting the base of the central core to the foundations is illustrated in Figure 10. Horizontal shear is transferred to the foundations via RC keys. Core tension, if it exists, is supported by large diameter steel bars between the foundations and the 1st floor hoop beam. These bars are located on the line of the middle of the hoop beam and are around 200mm beyond the outside face of the central core. Additional keys would be needed to transfer tension between the core and the first floor hoop beam.

Figure 10: Part elevation of central core base connection. B - reinforced concrete hoop beam (drawn shaded), E - reinforced concrete shear connector, R – additional key to transfer tension from CLT tube to 1st floor RC hoop beam, S – large dia. steel bars to transfer tension from 1st floor RC hoop beam to foundation, T – RC key to transfer CLT horizontal shear to foundation, U – RC foundation, V – RC piles (if required)

5 WIND INDUCED VIBRATION

Through dynamic loading and aeroelastic effects, the wind can induce vibration in structures in a variety of ways. Eurocode 1 [citation] lists the following ones:

- Along-wind vibration due to turbulence
- Across-wind vibration due to vortex shedding
- Galloping
- Divergence
- Flutter
Kawai [citation] suggests that the critical wind velocity for vortex shedding can be within the design wind velocity for buildings with an aspect ratio greater than 8, and Eurocode 1 requires a check to be carried out if the aspect ratio is greater than 5. The prototype building has an aspect ratio of approximately 2.3. The likelihood of the wind reaching the critical velocity for vortex shedding is further reduced by the relatively light weight of the structure, resulting in a high natural frequency. With a natural frequency of 0.4Hz and a square plan 42m by 42m, the critical wind velocity for vortex shedding is 140m/s, which is far higher than would be experienced by the building. The geometry of the structure means that it is not susceptible to galloping, divergence or flutter.

5.1 ALONG-WIND VIBRATION

The primary form of wind-induced vibration in this form of structure is along wind vibration due to turbulence. This is a resonant response of the structure which has consequences both for the serviceability of the structure, in that building occupants may perceive and complain about the movement, and for the ultimate strength behaviour of the structure, as the vibration adds to the peak loads experienced by the building. Under this form of load, the mass of the building generally tends to reduce the amplitude of vibration, so a light-weight structure experiences more severe vibration.

It is considered that the analysis procedure specified in Eurocode 1 [6] for assessment of along-wind vibration is appropriate for this structural form. The procedure is based on a method first proposed by Davenport [9], which represents the structure as a slender, line-like vibrating object.

5.1.1 Dynamic Properties of Structure

The response of the building is assumed to be dominated by the response in the fundamental mode of vibration, and the shape of this fundamental mode is chosen based on the form of the lateral load resisting system. In determining the shape of the fundamental mode in this case, the lateral loads are considered to be resisted primarily by the vertically cantilevering core.

The building is taken to be in an urban location, with basic wind speed and altitude as described in Section 3.2. Following guidance in Eurocode 0 [10], the mass of the building for dynamic calculation is calculated based on the sum of the minimum permanent actions on the building, that is, its self weight. This gives an average density inside the building envelope of 117kg/m³.

The frequency of the fundamental mode of vibration of the building with this mass is estimated from the elastic model as 0.4Hz. This frequency will be reduced by any slip in the shear connections in the core, and increased by any outrigger action in the radial beams and columns. It is therefore considered appropriate to assess the sensitivity of the vibration response to natural frequency over a range of frequencies centred on 0.4Hz.

Damping in building structures is normally estimated based on experimental measurements of buildings previously completed in that form, such as those presented by Satake [11] for steel and reinforced concrete buildings.

Eurocode 1 gives guidance on the magnitude of damping to assume in various forms of structure and structural materials. No guidance is given for timber building structures, however, and the fact that this is a new form of structure makes it inappropriate to use experimental evidence from other structures as a basis for damping estimation. A sensitivity analysis for damping has therefore been carried out over a wide range of values.

5.1.2 Vibration Acceptability Criteria

Vibration limits are specified by, amongst other standards and guidance documents, ISO 10137 [12]. The magnitude of vibration which is considered acceptable varies depending on the type of occupancy of the building. This document suggests that a peak acceleration of up to $60 \times 10^{-3} \text{m/s}^2$ at a frequency of 0.4Hz is suitable for residential buildings, and that up to $90 \times 10^{-3} \text{m/s}^2$ is acceptable a general office building at the same frequency. Both limits are indicated in the results of the analysis, although the stated use of the prototype structure is a commercial building.

5.1.3 Magnitude of Vibration

The variation of peak acceleration with damping and natural frequency is shown in Figure 9. This is the acceleration at the top of the building. The results show that for most levels of damping and stiffness the magnitude of vibration falls below the vibration threshold for general office occupancy.

![Figure 9: Sensitivity analysis for acceleration and comparison with vibration limits](image-url)

The results highlight the importance of achieving minimal slip in the shear connectors in the core, to maintain sufficient stiffness and therefore a sufficiently high natural frequency to achieve an acceptable level of wind-induced vibration.

Figure 10 shows the variation of peak acceleration with damping at the predicted natural frequency of the structure for zero slip. It shows that the peak acceleration is suitable for general office use at all the levels of damping above a logarithmic decrement of 0.06, and that a logarithmic decrement of damping of 0.14 or greater is required to achieve an acceptable peak acceleration for residential occupancy.
The serviceability of the building is therefore dependent on achieving the necessary values of stiffness and damping in the structure, to ensure that the vibrations are not of a magnitude that they might be perceived by the building occupants. It is noted that in a lightweight, flexible building of this sort, it may well be more efficient to use special devices to enhance the damping in the structure, rather than to add material to the structure to increase stiffness, which may be detrimental to other aspects of the structural performance. Smith and Willford [13] propose a supplementary damping system for a tall building which could be applied in this case. By stiffening the radial structure at certain floors, some outrigger action could be developed between the core and the CLT columns, and viscous dampers could be incorporated into the connections.

5.1.4 Dynamic Forces

The amplification of the static wind load is practically represented in Eurocode 1 by \( c_s c_d \), the variation of which is shown in Figure 11 for the prototype structure. The dynamic factor \( c_d \) is a multiplying factor to allow for the inertial forces induced in vibration, and \( c_s \) reduces the forces to account for the lack of correlation of wind forces over the face of the structure.

The magnification of the static forces is only 4% at the extremes of natural frequency and damping considered in the sensitivity analysis. Considering only the cases where the peak acceleration meets the criteria for office use, there is no magnification of forces above those already considered in Section 3.2.

6 CONCLUSIONS

This paper proposes a cross-laminated timber, CLT, structural system to support commercial buildings to thirty levels. The main structural element is a strong central timber tube core. Timber beams and columns, that assist the tube core in frame action, are located on rays from the centre of the core. Except for the RC ‘hoop’ beams, floor toppings, and foundations, all the main structural elements are made of CLT. CLT has reliable strength and stability and is a marvellous new timber building product. The proposed prototype building has considerably more open spaces than existing CLT multi-level buildings which rely on multiple shear walls. The CLT panels are arranged with timber wastage less than 5%, and the inter-panel jointing is designed to be simple and have minimal slip. An elastic analysis indicates that the main structural members and associated jointing have reasonable factors of safety. Also, the analysis shows that suitable inter-storey deflections are achieved during major wind events.

This dynamic analysis highlights the fact that there is a lack of information on the damping of building structures in timber, and particularly on the damping of CLT buildings. This means that the magnitude of vibration in the prototype building cannot be accurately predicted, and for that reason a sensitivity study has been carried out here, which shows that damping is a very important design consideration in this case. An experimental study into the dynamic properties of completed CLT buildings would provide valuable information for the design for vibration of taller structures using CLT.

The investigation into wind-induced vibration of the prototype structure shows that significant accelerations are induced by turbulent wind loading, but that they are likely to be within the limits recommended in design guidance for a commercial building. It is also possible that the building could meet the requirements for residential occupancy, especially if special damping devices were incorporated in design.

This paper concludes that CLT construction may be suitable for buildings to 30 levels.

REFERENCES


[7] Chapman, John, Dodd, Dr George 'Improved sound insulated floors for a 6 storey timber building system', New Zealand Acoustics, volume 22, 2009 / #4, pp20 – 26


