CHALLENGES AND SOLUTIONS IN THE DESIGN OF A 10 STOREY CLT BUILDING

D. Moroder[1], T. Smith[2], F. Sarti[3], J. Armstrong[4], B. Young[5] & A. Buchanan[6]

[1]Structural Engineer, PTL | Structural Consultants, d.moroder@ptlnz.com

[2]General Manager, PTL | Structural Consultants, t.smith@ptlnz.com

[3]Structural Engineer, PTL | Structural Consultants, f.sarti@ptlnz.com

[4]Structural Engineer, Taylor Thomson Whitting, jane.armstrong@ttw.com.au

[5]Director, Taylor Thomson Whitting, barry.young@ttw.com.au

[6]Principal, PTL | Structural Timber Consultants, a.buchanan@ptlnz.com

This paper was originally published for the 2017 SESOC Conference in Wellington.

ABSTRACT

Australia has joined the world in pushing the boundaries of timber construction. In recent years several tall all timber buildings have demonstrated the significant advantages of mass timber construction, especially in multi-unit, multi-level residential buildings. Recent modifications to the National Construction Code of Australia have provided a deemed-to-satisfy pathway for timber structures of up to 25 meters (or 8 stories), further accelerating the adoption of tall timber buildings.

As these buildings become more commonplace, it is natural that architects and clients begin to push the boundaries of structural form. This leads to increasing spans, irregularities and openings. One such structure is a ten storey aged care facility currently under construction in Sydney, Australia. The entire structural system is composed of engineered wood, predominantly of Cross Laminated Timber (CLT), with the exception of the steel framed balconies, sitting on a two storey reinforced concrete base. The architectural design required large open spaces and expansive glazed façades. Economies gained from importing the CLT panels from Europe limited the design team in possible panel dimensions.

These challenges meant that innovative solutions and design approaches were required, supported by learnings from past projects, both in Australia and around the world. This paper provides an overview of the design challenges and discusses a range of solutions to issues including the incompatibility of 'standard' European brackets with Australasian architectural layouts, modelling of limited size wall panels, analysis of diaphragm response and problems of floor dynamics.

A collaborative design team ensured that these challenges were overcome, enabling what will be one of Australia's most exciting mass timber buildings.

1 INTRODUCTION

Engineered timber construction has a long history in New Zealand, but the number of outstanding buildings made of timber remains low, mostly due to the competition with other building materials like steel and concrete and the perceived higher risk of using a less common construction material. The amount of timber construction has increased since the Structural Timber Innovation Company (STIC) developed new technology and guidelines for the design of Timber Concrete Composite floors (TCC), prefabricated solid timber floors, QuickConnect portal frames, timber rivets and the Pres-Lam system of post-tensioned structural timber technology. The investment of some well-established engineered timber manufacturers in CNC machines and a new Cross Laminated Timber (CLT) manufacturer in Nelson has further increased the uptake of timber buildings. This has led to a better general education of the industry in using timber and has also sparked the investment in a new large-scale CLT manufacturing plant in Australia.

The Australian construction market has followed this development, and has used New Zealand as prime example of how to increase the use of engineered timber. The recent construction of the Forte Building and the Docklands Library in Melbourne, as well as the International House office building in Sydney [1] have attracted considerable interest in engineered timber construction. The Australian timber industry and WoodSolutions [2], developed an industry initiative which successfully lobbied for a change in the National Construction Code of Australia to allow a deemed-to-satisfy pathway for timber structures of up to 25 meters (8 stories) tall [3]. Since this code change and documented cost savings in the order of 15% [4], the number of multi-storey residential and commercial buildings in design or under construction has skyrocketed, with a number of larger developers leading the charge. Australia has now overtaken New Zealand in terms of new timber buildings and has attracted a great number of European engineered timber manufacturers, keen to deliver off the shelf products or entire projects to Australasia.

This rapidly increasing interest in mid-rise timber construction brings challenges to the designers and contractors, as it requires a novel approach to designing and constructing multi-storey timber buildings. Even though guidance and past experience from Europe can be used, different architectural detailing, transport limitations and bold structural layouts create some challenges which architects, engineers, manufacturers and builders need to overcome.

This paper describes the challenges encountered and the solutions provided for the design of the 10 storey CLT structure in Sydney shown in Figure 1. Designed by Jackson Teece Architects and Taylor Thomson Whitting Sydney in collaboration with PTL Structural Consultants, the building will host approximately 67 luxury aged care residential units constructed from over 3,250m^3 of CLT panels. The building also houses retail spaces on the ground floor and a conference centre over two floors, as well as basement parking. The CLT panels and glulam beams are supplied from Binderholz in Austria and are being delivered to the manufacturing plant of the main contractor Strongbuild in Sydney, where machining and pre-assembling is carried out before delivery to site and erection of the panels.

This will be one of Australia's most exciting timber buildings and one of the largest timber building in the Southern hemisphere.

2 STRUCTURAL FORM

The ten storey structure shown in Figure 2 consists of underground car parking and a first storey concrete

Figure 1: *Artist's impression of the new 10 storey timber structure*

podium for retail. All other storeys are built in timber, mainly with CLT wall and floor panels and glulam lintels over door openings. All panels were initially planned and designed to be shipped as 1.25m wide panel segments, which could easily fit standard container sizes. Wall panels were to be pre-assembled off-site with nailed splices in a dedicated warehouse. Due to the time and cost of the large number of fasteners required, it was later decided that full sized wall panels should be shipped in open containers directly from Europe to the building site.

In contrast to other structures built in CLT, normally comprising of single residential dwellings and only more recently multi-storey multi-residential buildings, the building has a relatively irregular and large plan with an area of 632m^2 per floor. The long floor spans push the limit to satisfy stringent serviceability performance requirements. Because of the large openings along the perimeter of the structure, the typical box-like behaviour common to other tall CLT buildings could not be relied upon, and the floor diaphragms were required to carry the loads back into the internal walls. The height and width of the building attracts very large wind loads, generating large uplift and shear demands in the individual wall panels. Seismic loads were considered in the design, but were not governing.

Although high acoustic and fire performance levels need to be achieved in the building, these have little impact on the structure itself. To guarantee 60 minutes fire rating and to minimize the spread of flame, all walls are lined with fire resistant plasterboard on both sides. CLT partition walls have stud framing fixed on one side with resilient fixings allowing an air cavity for acoustic separation. The cavity is filled with insulation and lined with an additional layer of plasterboard.

To reduce the impact of sound transmission, 40 mm

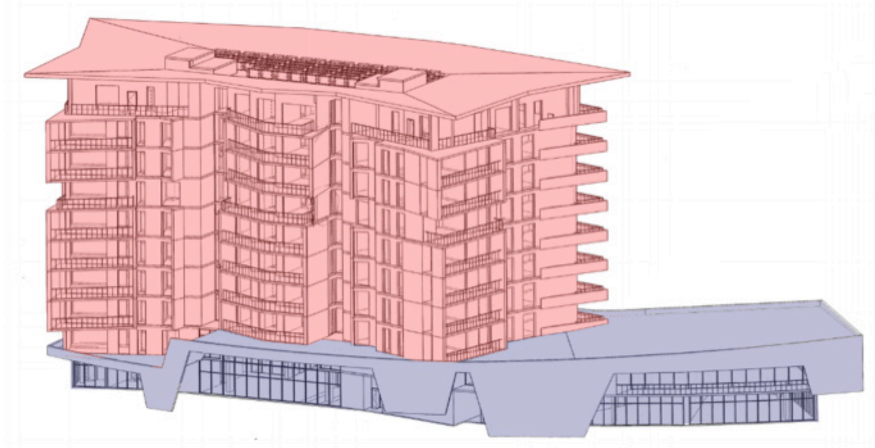


Figure 2: *3D sketch of the structure. Concrete podium is shown in blue, CLT structure in red.*

of concrete screed is placed directly on the CLT floor panels, which is then covered with a soundproofing acoustic underlay under the floor finish. To fire rate the floor, a layer of fire resistant plasterboard is fixed directly under the CLT panels. A suspended ceiling with plasterboard and lightweight insulation further improves the acoustic performance of the floor assembly. Due to the presence of the heavy floor and the stud framing on the walls, no additional measures were required to diminish flanking noise between partition walls and the floor. Rubber bearings are provided under the walls along the lift shafts and stairwells to attenuate transfer of flanking noise.

2.1 Gravity load resisting system

All gravity elements in the structure above the concrete podium are CLT wall and floor panels, with structural steel framing to the balconies. These are designed to resist a dead load of 2 kPa including the 40 mm screed and a live load of 2 kPa, except for the corridors where a load of 4 kPa is needed. All the floor panels act as simply supported or continuous beams over two bays, seated directly on top of the wall panels, which act as linear columns. Because of the large floor area and the varying floor spans shown in Figure 3, the panel thickness was varied as statically required. Even though the resulting steps are hidden by the suspended ceiling, the connection detail between the floor and wall panels had to be designed carefully in order to transfer all forces accordingly and to avoid splitting due to notches in the panels as will be discussed in Section 3.4.2.

Over most door openings glulam lintels transfer the load via direct bearing contact to the adjacent wall panels (see Figure 4 centre). This additional load in the walls needs to be accounted for and will be discussed later in Section 3.4.4. For smaller penetrations like windows, the openings are cut out directly from the panels (see Figure 4, right).

2.2 Lateral load resisting system

The structure is considered to be of Importance Level 3, with a return period of 1000 years for wind and earthquake events. Situated in wind region A2 and with a terrain category 2 (open terrain with no more than two obstructions up to 5 meters per hectare), the regional wind speed at the ultimate limit state is 46 m/s as per AS/NZS 1170.2 [5]. Even though not governing, the earthquake load is calculated for a probability factor k_p=1.3 and a hazard factor Z of 0.08 for a site sub-soil class C. Due to the height of the structure this requires a design according to the earthquake design category 3 as per AS1170.4 [6]. The structure was analysed with a modal analysis with a S_p/μ ratio of 0.38.

All vertical walls in the structure are designed to act as bracing walls in order to guarantee the drift limit of H/500 under serviceability wind loads and to resist the ultimate wind loads. The use of platform frame construction (i.e. the wall panels are interrupted at each floor) required the design of hold downs and shear splices in the walls at each storey. The interruption of the wall panels, together with the perpendicular-to-grain compression properties of the floor panels and the wall segmentation at every floor required increased analysis of the wall response. This was done to accurately determine the forces in the hold downs and shear brackets, as well as to define the fastener spacings in the panel splices. The modelling approach to account for all the sources of flexibility is discussed in 3.1.

During the design, it became evident that the use of multiple 1.25 m wall panels with splices would

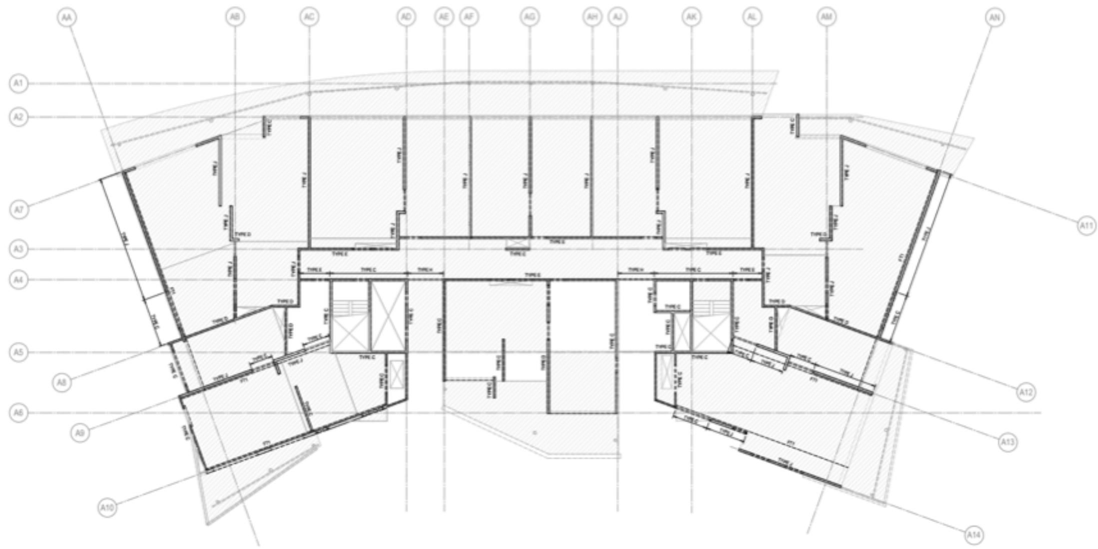


Figure 3: *Plan view of a typical level*

require very small fastener spacings which were not cost effective when compared to importing wider panels (in open-top containers). Figure 4 shows the layout of the typical wall design with decreasing wall thickness along the building height, and the number of hold down brackets (nominated TCN) and shear brackets (nominated WBO). Where penetrations were required, the wall was either divided into two independent bracing walls (Figure 4, centre) or was analysed as a wall with openings (Figure 4, right). In the latter case the stress concentrations around the openings had to be verified in order to guarantee the bracing action. Furthermore, the behaviour of these configurations under lateral loads was considered to be more like frames, whose displacements were not compatible with typical wall deformations, creating significant transfer forces in the floor diaphragms. Both design issues are discussed in Section 3.2.

3 DESIGN CHALLENGES AND SOLUTIONS

3.1 Modelling of the shear walls

Since all of the CLT panels were initially intended to be imported as relatively narrow strips, their splicing details needed to be considered during the design. Due to the building height and the high horizontal wind loads, the wall panels and their respective connections needed to be modelled in such a way to account for all sources of flexibility, and to predict the fastener demand. A spring model was developed, which allowed the equivalent wall properties to be

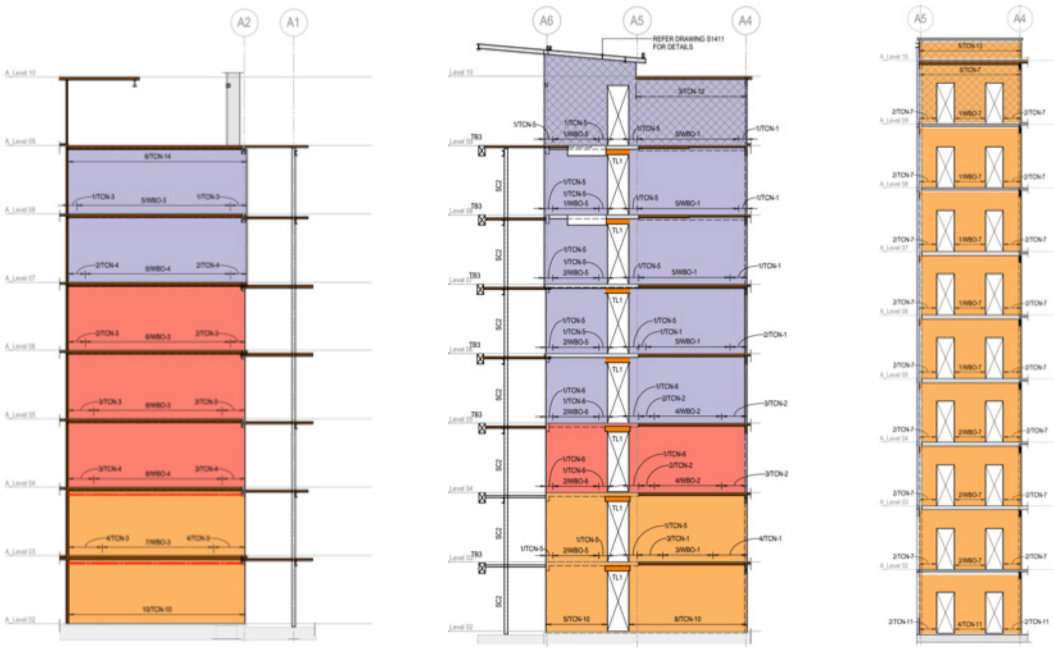
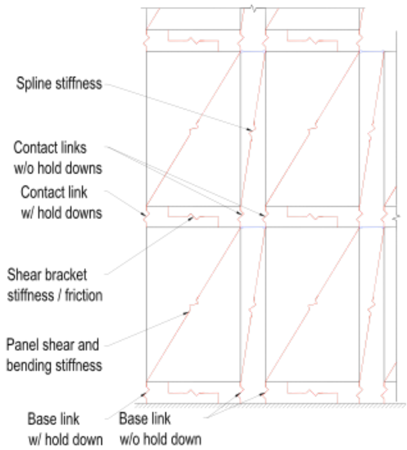
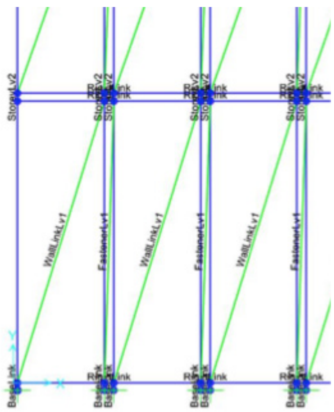


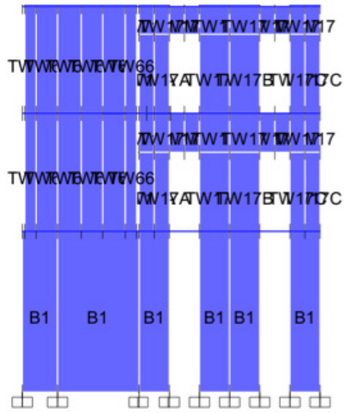
Figure 4: *Typical CLT walls with fasteners (different colours indicate different wall thicknesses)*



a) Schematic spring model



b) Spring model of a typical wall



c) ETABS model with equivalent wall properties

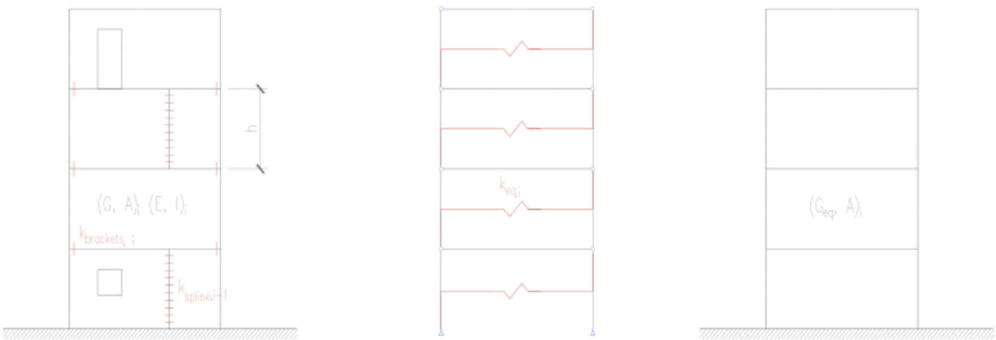
Figure 5: *Models for CLT wall analysis*

determined for use in Etabs [7].

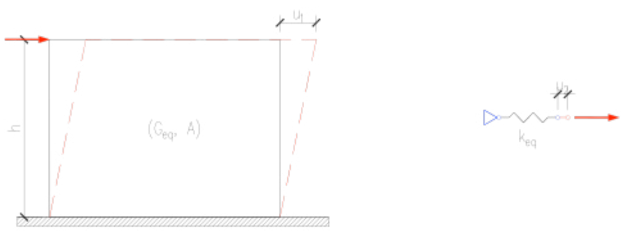
Figure 5(a) shows the spring model, accounting for all sources of flexibility in a typical wall: shear and bending of the CLT panels, shear brackets, panel splice and hold-down/compression connections of the panels through the floors. Each panel splice was modelled with contact links (compression perpendicular to grain in the floor panels); and with and without hold downs (tension) for external and internal panel connections respectively. The connections of the panels to the concrete floor/walls were normally more rigid and were specifically accounted for. Stiffness properties of proprietary brackets were made available by the supplier. The compression stiffness of the floor panels was determined assuming a stress spreading of 30degrees [8]. Although friction between the wall and floor panels could be taken into account [9], in order to resist the serviceability wind loads, it was decided to neglect friction and to only rely on the shear brackets.

Once all stiffness values were determined, a linear push-over analysis of the spring model was used to determine an equivalent horizontal stiffness k_{eq} for each level. From this, the equivalent material properties for the shell elements in Etabs could be determined by the deformation analogy as shown in Figure 6a. The equivalent shear stiffness was determined by Equation 1, based on the model in Figure 6b.

Once all wall properties were inserted into the 3D model, the building deflections under serviceability conditions and the shear force distribution in the



a) Wall with properties, equvialent horizontal spring and equivalent shear stiffness of panel



b) Analogy between shear deflection in a CLT wall and elongation of the equivalent spring

Figure 6: *Equivalent wall model*

building could be assessed. By applying the storey shears on the spring model, the number of required fasteners could be calculated. Since the equivalent panel shear stiffness was based on the chosen wall thickness and layup as well as the fastener spacing, a small number of iterations were required to analyse and design the structure.

$$G_{eq} = \frac{k_{eq} h}{A} \tag{1}$$

As mentioned above, value engineering determined that importation of large finished wall panels in open top containers was more economical than the original individual spliced wall panel option, simplifying the analysis of the wall elements. Figure 7 shows a typical wall layout with hold down and shear brackets for the full size wall panels.

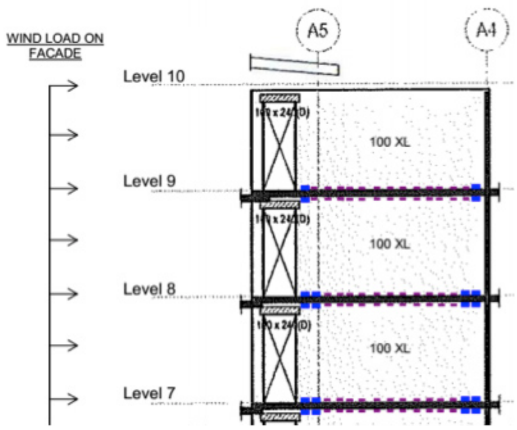


Figure 7: *Wall layout with single panel*

3.2 Dual system

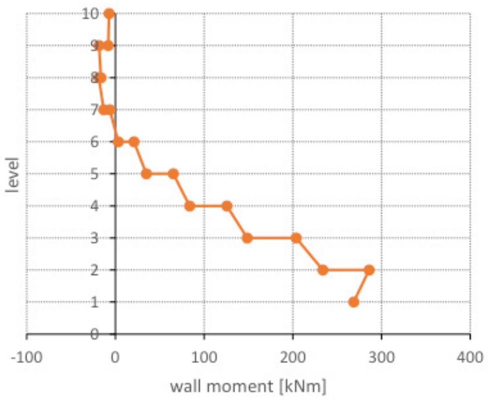
By analysing the wall shears and moments from the 3D model, as shown in Figure 8a, it became evident that the structure was not responding as a purely cantilevered structure The observed negative moment at the top stories is typical of dual structures, normally wall and frame structures, where displacement incompatibilities create large transfer forces [10]. In addition, the coupling effect of the floors contributes to resisting the lateral loads. In CLT buildings it is not common to design the floors to resist these moments therefore the moment connection between the floor and wall panels was decoupled.

The moment distribution shown in Figure 8a can be explained by the fact that some bracing walls have openings cut out, changing the behaviour of the remaining wall panel from a typical cantilever to a frame. Because the panel with openings is connected to adjacent panels without openings, their different deformation patterns cause displacement incompatibilities. This leads to the typical 'fighting effect' creating the negative moment in the higher storeys and transfer forces in the diaphragm.

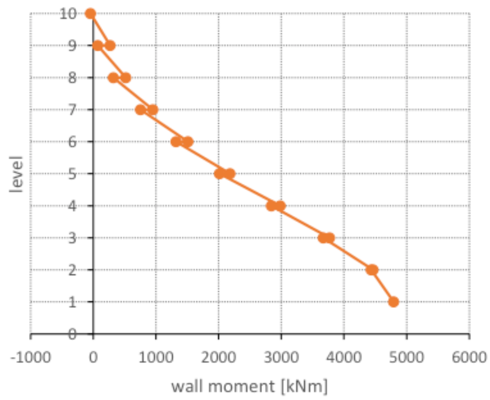
It became essential to check the stress concentrations around the openings of the shear walls. This could be easily done by the 'cut section' command in Etabs, which provides the integration of the stresses along an ideal section. These values were then verified against the axial, shear and moment capacity of the remaining CLT panel. In addition, diaphragm panels and fasteners need to resist large transfer forces, which are problematic for the panel splices, as they are only designed for shear along the panel edges. Because of the larger resultant force in the panel splices and the increased edge distances, it was decided to use steel ties to transfer these diaphragm forces as will be discussed in the next section.

3.3 Diaphragm design

Since the floor diaphragms are built from spliced floor panels, and with the absence of walls along

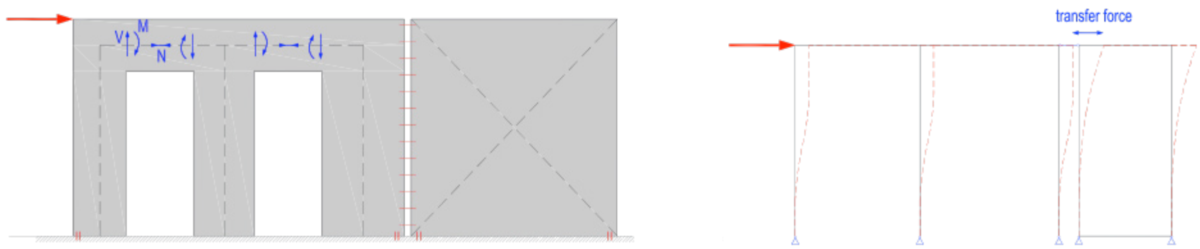


a) Short wall showing the typical moment distribution found in dual structures



a) Long wall

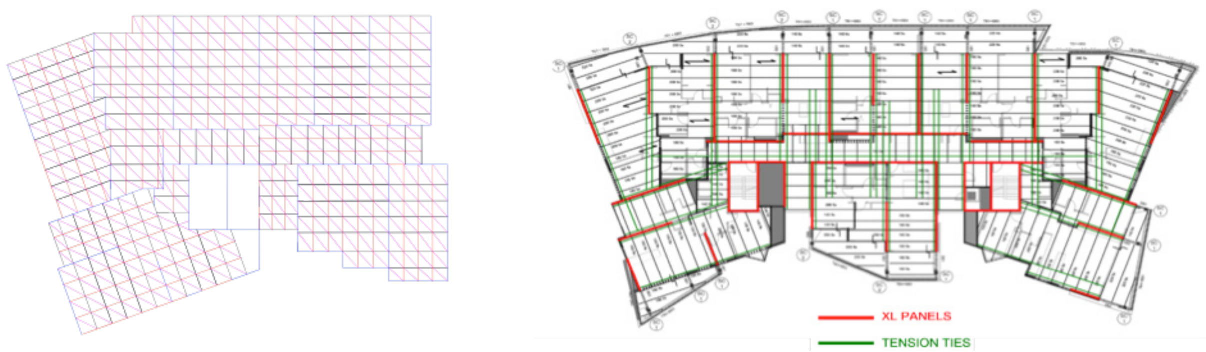
Figure 8: *Moments along two typical walls*



a) Internal forces in wall around openings   b) Dual system

Figure 9: *Wall with openings coupled with a solid wall. Forces in wall around openings and dual system analogy*

the perimeter of the building, it was paramount to carefully analyse the load paths in the diaphragms. The diaphragms were modelled as shell elements with a reduced shear stiffness to account for the panel splices. This equivalent shear stiffness was calculated based on the Equivalent Truss Method [11]. This approach was sufficient to assess the diaphragm stiffness and therefore the force distribution into the walls. Because of the presence of transfer forces, and the incapacity of standard panel splice connections to transfer forces perpendicular to the panel edges, the diaphragm was also analysed with the Equivalent Truss Method as shown in Figure 10(a). With this approach tension ties in the form of multi-braces and nail-on plates could be verified. These elements as shown in Figure 10(b) were used to create the tension chords and strut beams at re-entrant corners and around openings. The ties are also used to connect portions of the diaphragm which would not have any direct tension connection due to the discontinuity of the walls (i.e. the portion of the diaphragms on either side of the corridor).



a) Equivalent Truss Model (only the left part of the diaphragm is shown)   b) Preliminary position of diaphragm panels and tension ties

Figure 10: *Shear walls. floor panels and diaphragm ties*

3.4 Connection detailing

3.4.1 Brackets

CLT structures were first built in Europe about 20 years ago and standard details have become available over the last decade. This has led to wide availability of proprietary brackets and hold downs on the market. Most of these are conceived for the use in typical European floor and wall assemblies with floating floors, service cavities and large amounts of insulation. Most buildings in Australasia do not allow for thick floor assemblies and wall cavities under the plasterboard linings. The typical hold down shown in Figure 11(a) could therefore not be used in this job, as it would have been incompatible with the architectural details. The problem was overcome by using a newly available bracket from Rothoblaas shown in Figure 11(b), which can carry the large axial load with the use of a special washer. The additional advantage of this bracket was that stiffness values are available and special rubber pads can be combined with the brackets to reduce flanking noise.

a)   b)

Figure 11: *Standard hold down bracket WHT and Titan N with washer [12]*

3.4.2 Change in panel thickness

Large cost savings were achieved by changing the

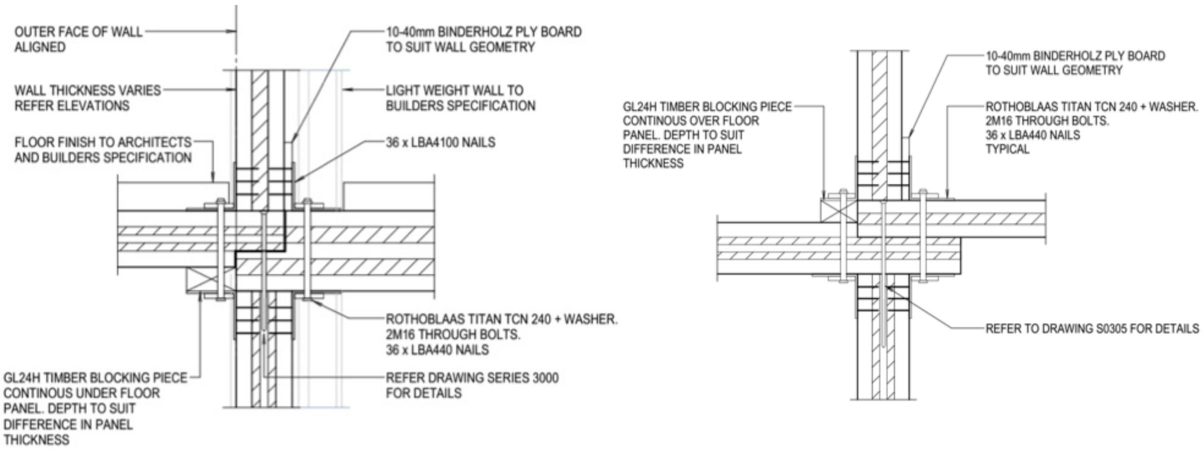


Figure 12: *Typical hold down connections with varying floor and wall panel thicknesses*

panel thickness over shorter floor spans or by reducing the wall thickness along the building height. However, these had to be accounted for in the connection detailing, as gravity forces still need to be transferred into the walls and the shear and axial loads from the upper walls need to be transferred into the lower walls. Depending on the relative thicknesses, different solutions were adopted in the building as shown in Figure 12.

Where floor panels are notched underneath (Figure 12 left), the shear verification had to consider the low splitting resistance of timber. This verification is further penalized by the presence of the cross layers, reducing the effective height as shown in Figure 13. Although this verification was satisfied in most cases, a limited number of panels required extra reinforcement, which was provided by the bolts connecting the shear brackets above and underneath the floor panel. Spacers in form of battens or boards were required to connect the shear brackets between the different panels. Special care was required as fasteners through these spacing elements had lower strength and stiffness capacities when they were not rigidly fixed to the CLT panel [13].

Figure 13: *Verification of notches in CLT panels [14]*

3.4.3 Low strength of floor panels perpendicular to grain

Timber has a relatively low strength in compression-perpendicular-to-grain, and transferring large axial forces from one storey to the other through the floor panels can quickly reach this limit. For a small number

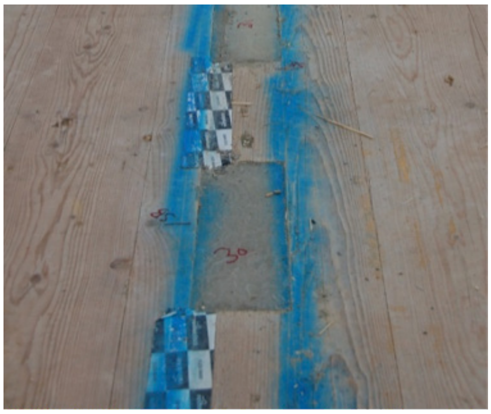


Figure 14: *Notches in floor panels filled with concrete, from another job*

of highly loaded walls, an alternative load path for gravity loads was required. To bypass the floor panels, notches are cut in correspondence of the wall panels similarly as shown in Figure 14. The notches, filled with glulam sections, provide a direct load path between the axial layers of the wall panels. Care was needed to assure that the remaining floor panel had enough capacity to transfer the gravity loads into the supporting wall panels.

3.4.4 Distribution of lintel forces

Although CLT panels have a relatively high axial capacity when they act as linear supports for the floor panels, the introduction of concentrated forces over several stories can quickly reach the wall's capacity. Since the floor geometry is identical for all storeys, the sum of all forces introduced from the lintels above the door openings was too high to be resisted by the small strip of wall under the lintels. To overcome this, the stress spread capacity of the CLT panels was taken advantage of, assuming a 30 degree stress distribution angle, so that the force spread through the cross layers diminishes after a quarter of the panel height [9] as shown in Figure 15.

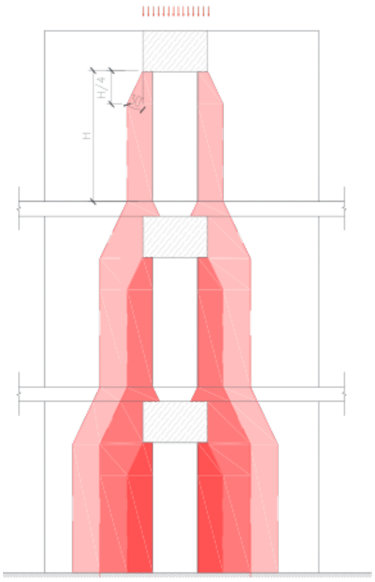


Figure 15: *Distribution of vertical forces from lintels*

The force can however be spread again at each floor panel and in the wall panel underneath. With this approach the lintel reaction over 9 storeys is resisted by the increased resisting area at each storey.

3.5 Floor vibrations

Most of simplified analysis methods for predicting the dynamic behaviour of floors do not capture the vibration performance of lightweight, low damped systems, therefore the vibration analysis as outlined in Smith et al. 2009 [15] was used. The vibration performance of the floor was measured in terms of weighted root-mean-square acceleration and compared against acceleration limits in accordance to BS 6472 [16] and ISO 10137 [17]. Even though these references provide limiting values, it is important to keep in mind that vibration problems are about human perception, and annoyance thresholds can vary, so the limiting values are based on probability considerations.

The assessment of the vibration performance of the flooring system was carried out with a finite element modal analysis of a typical apartment unit floor. The modal analysis results were then used to evaluate the steady-state and transient responses of the floor in accordance with Smith et al. 2009.

The vibration analysis results for a typical unit are shown in Figure 16. The plan view shows the distribution of the R-value which is the ratio between the weighted root-mean-square acceleration and the human perception threshold acceleration. The limiting values of the response factor, R, are based on Table 5 of BS 6472 and correspond to a "low probability of adverse comment" with up to three occurrences. Limit values of 30 and 20 were assumed for daytime and night time, respectively. A very limited area in the apartment and an area of the balcony, as shown in Figure 16, were found to be above these limits and some design adjustments were necessary in order to obtain a 'lower probability of adverse comment' from the users, by using thicker floor panels or a stiffer beam under the outer edge of the balcony. Some early photos of the building under construction are shown in Figure 17.

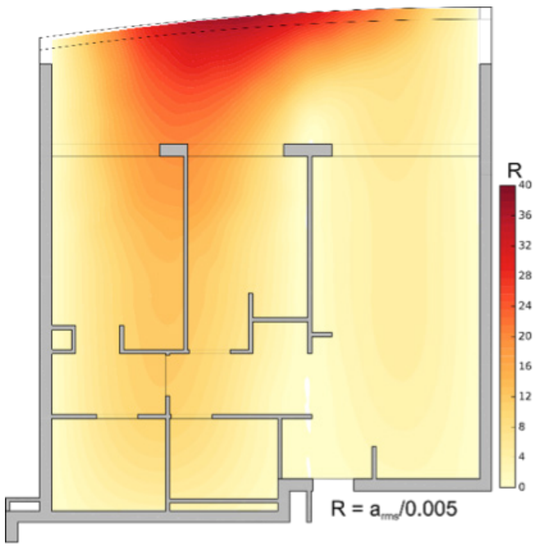


Figure 16: *R values indicating likely vibration perception for a single apartment unit*

4 CONCLUSIONS

This paper has presented an overview of the structural challenges and solutions during the design of a tall CLT building currently under construction in Sydney. The ten storey structure was an all timber CLT structure, with structural steel balconies, on a concrete podium.

In contrast to typical CLT structures such as single storey residential dwellings and small multi-storey multi-residential buildings, this building has an irregular and very large floor plan, requiring additional care in the design of both the gravity and lateral load resisting systems.

Understanding of the system displacements is crucial in the design of any structure, and paramount in the design of a tall timber building. For this reason, computer modelling was used to understand load paths and interactions within the structure. The modelling of individual fasteners or connections consumes significant computational cost and time, so equivalent springs were used to calibrate cantilever members within the 3D Etabs model. An equivalent truss model was used to understand floor load paths and to ensure diaphragm actions.

During the analysis, it was noted that dual system action, common to hybrid reinforced concrete structures, was being displayed by the numerical model. Although this assisted the structural performance, careful design consideration was required to ensure the stresses created by this action could be resisted by the structural members. For example, while it is possible to transfer stress where doors are cut within the CLT panel, it is difficult to create the required moment connection across glue laminated lintels.

Significant material savings were made by altering the thicknesses of panels, both up the height of the building and across the floor plate, which created an additional challenge during detailing with notches and cuts being required to enable the use of hold down brackets. Additional design consideration was also required around door lintels where the cumulative introduction of concentrated loading up the 10 storey building placed significant compressive load on the CLT panels.

As with any lightweight flooring solution, the use of long span CLT floor requires rigorous analysis to identify potential vibration issues. Finite element analysis was performed to check performance against code levels.

Close collaboration between the members of the Trans-Tasman design team was key in ensuring the successful design of what will be one of Australia's most exciting timber buildings.

Figure 17: *The ten storey CLT structure under construction*

5 REFERENCES

[1] T. Butler. International House Sydney. International Holzbau Forum. Garmisch-Partenkirchen, Germany, 2016.

[2] WoodSolutions. www.woodsolutions.com.au, 2017.

[3] WoodSolutions. 2016 National Construction Code Change. timberUPdate(1), 2016.

[4] FWPA. Final Report for Commercial Building Costing Cases Studies – Traditional Design versus Timber Project. PNA 308-1213. Forest & Wood Products Australia Limited, Melbourne, Australia, 2015.

[5] Standards New Zealand, and Standards Australia. AS/NZS 1170.2:2002 Structural Design Actions Part 2: Wind Actions, Wellington, New Zealand, 2002.

[6] Standards Australia. AS 1170.4 - Structural Design Actions Part 4: Earthquake Actions in Australia, Sydney, Australia, 2007.

[7] CSI. ETABS 2016: Integrated analysis, design and drafting of building systems. CSI Computers and Structures Inc., Berkeley, CA, USA, 2016.

[8] H.J. Blass, and R. Görlacher. Compression perpendicular to the grain. World Conference on Timber Engineering. Lahti, Finland, 2004.

[9] M. Wallner-Novak, J. Koppelhuber, and K. Pock. Cross Laminated Timber Structural Design - Basic design and engineering principles according to Eurocode. proHolz Austria. Vienna, Austria, 2014.

[10] T. Paulay, and M.J.N. Priestley. Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley & Sons, Inc, 1992.

[11] D. Moroder, T. Smith, S. Pampanin, and A.H. Buchanan. An equivalent truss method for the analysis of timber diaphragms. Pacific Conference on Earthquake Engineering. Sydney, Australia, 2015.

[12] Rothoblaas. Wood Connectors and Timber Plates, Cortaccia, Italy, 2015.

[13] H.J. Blass, and B. Laskewitz. Load-carrying capacity of joints with dowel-type fasteners and interlayers. CIB-W18 Meeting. Delft, The Netherlands, 2000.

[14] KLH. The new ETA-06/0138 for KLH solid wood slabs - Approval and comment, Katsch an der Mur, Austria, 2013.

[15] A.L. Smith, S.J. Hicks, and P.J. Devine. Design of Floors for Vibrations. The Steel Construction Institute, 2009.

[16] British Standards Institution. BS 6472-1:2008 Guide to evaluation of human exposure to vibration in buildings. Vibration sources other than blasting, 2008.

[17] International Organization for Standardization. ISO 10137:2007 Bases for design of structures - Serviceability of buildings and walkways against vibrations, 2007.