DALSTON LANE - THE WORLD’S TALLEST CLT BUILDING

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ABSTRACT: When completed in 2016, Dalston Lane will be the tallest platform CLT structure in the world. In this paper the motivation and drivers for the project are considered, as is the design process that resulted in a highly coordinated fabrication model. Details of the numerous design challenges investigated by the design team, including stability, fire and connections, are outlined, alongside testing of floor plate stiffeners with Cambridge University. It is concluded that to construct taller buildings using the same design principles it will be necessary to undertake further engineering design and research.

KEYWORDS: Dalston Lane, United Kingdom, Cross Laminated Timber, Tall, Timber, Ramboll, CLT

1 INTRODUCTION

Dalston Lane is a 10 storey cross laminated timber (CLT) residential development in London, UK, due to be completed in 2016. At 33.8m, once completed it will be the tallest platform timber structure in the world. Using approximately 4000m³ of CLT, it will also be one of the largest.

Ramboll UK (RUK) worked as part of an experienced design team, including Waugh Thistleton Architects (WTA), Pringuer-James Consulting Engineers (PJCE), XC02 Energy and CLT contractor B&K Structures (BKS) to deliver the landmark project for developer Regal Homes.

Timber buildings have been steadily increasing in size over the past few years, from The Stadthaus in London [1] (also designed by WTA), completed in 2008 at 29.75m, to the Treet building in Norway [2], at 49m when complete. This latest engineering achievement is the result of over a decade of expertise in timber design developed at Ramboll, working in partnership with Cambridge University (CU).

2 CONTEXT

2.1 Hackney and cross laminated timber

CLT is a mass wood panel system that uses timber to form load-bearing solid wall, floor and roof panels. Individual boards are glued in multiple layers of alternating direction to form large single panels, typically up to 3x16m, and 50-300mm thick (Figure 2). Structural openings, such as doors and windows, are incorporated within the panels. It offers numerous benefits over traditional timber frame systems, including higher capacities, improved fire resistance, and greater dimensional stability.

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The London borough of Hackney has had a ‘wood first’ planning policy since 2012 [3]. It promotes the use of wood construction over more traditional materials such as concrete or steel, primarily due to its sustainable credentials. This policy has put the borough at the forefront of timber construction in the UK.

2.2 UK usage

In 2013, the UK Government published its ‘Construction 2025’ strategy, outlining its aspirations for the industry in 2025 [4]. Some of the ambitious targets set out include reducing costs by 33%, delivery time by 50%, emissions by 50%, and increasing exports by 50%.

CLT has many advantages over traditional construction materials and methodologies which address some of these targets. Most notably, the erection time of CLT structures is significantly lower, and the material can be sustainably produced, potentially even carbon negative when sequestration is considered [5].

As a result, CLT usage in the UK has been steadily increasing over the last decade. CLT supplier data [6] suggests that the volume of CLT supplied to the UK over the last 5 years nearly doubled –fuelled by a buoyant education and residential market. Our experience suggests the trend in the UK is for large scale developments to use significant volumes of CLT in one development, as opposed to the European model of more, smaller scale developments.

Over the past few years some large CLT developments have been completed (Figure 3), such as the award winning Ramboll projects of Open Academy and City Academy in Norwich, and the William Perkin High School in London. All of these have pushed the boundaries of design and were the largest developments of their kind in the UK at the time of completion. In total there are approximately 500 CLT projects in the UK, to date Ramboll have worked on over 40 of these projects in various sectors.

2.3 Tall timber

High rise timber of up to 12 storeys has become a necessity in many inner city sites where land values are high. The industry in the UK is slowly pushing the boundaries of this —starting from the Stadthaus in 2008, through to Bridport House in 2012 and now Dalston Lane. All of these have responded to various external influences and drivers to opt for timber construction.

3 DESIGN DEVELOPMENT

3.1 Brief & Site Constraints

In 2014, Regal Homes commissioned the design of a residential development on the recently purchased site at 67a-71 Dalston Lane. WTa developed the concept design, with the rest of the design team including RUK as the specialist CLT engineer.

The brief was to provide 121 residential units, with an additional 3450m² of non-residential space. This is primarily composed of offices, with some allowance for retail and restaurants.

3.1.1 Architectural strategy

The site is bordered by roads on the north and east, and adjacent buildings to the south and west. The south boundary is directly adjacent to an existing block of flats (Figure 4).

The adjacent buildings to the south drove the initial massing exercise, with the building split into two blocks and joined by a low rise office. The southern half of the building is kept to just 5 storeys, referencing the adjacent structures and maximising natural light on the south facades.
3.1.2 Construction weight

The site is located over the underground rail lines of High Speed 1 and the proposed exclusion zone for Crossrail 2. Strict foundation load limits were imposed on the scheme as a result of this, and the design team explored timber as a lightweight frame option. An early stage comparative study indicated that the weight of an equivalent concrete frame would be around 4x that of the timber scheme.

These weight savings mean that an additional three storeys were included on the building, resulting in an additional 15 flats in the scheme (Figure 5). The option of expanding further was also considered. An allowance was made in the structural design to add additional storeys to the shorter blocks as part of a future expansion. This could potentially allow for 20 additional flats to be added at a later date.

Figure 5: Comparison of sizes using timber or concrete

The use of timber in this context was clearly beneficial, as it allowed the developer to maximise their usage of the space within the constraints imposed.

3.2 Structural philosophy

The timber structure has been designed to provide an efficient layout with regards to m³ timber / m² floor area. The floors are kept to a maximum span of ~5m, and panels are designed to be continuous over wall supports.

Floors span in one or two directions, and are supported by bearing onto the CLT walls on the level below. A repetitive floor plan arrangement allows these walls to stack at each level, giving a direct vertical load path onto the first floor concrete podium. In a few locations on the upper floors the layouts change to facilitate larger penthouse flats. Where required, steel beams transfer out the vertical load to adjacent walls.

On the ground floor the building usage is commercial, with more flexible spaces and higher ceilings. This meant a first floor concrete podium slab was adopted (Figure 6), with columns to ground floor. As piles would clash with the rail tunnels, a raft foundation was adopted, to evenly spread the load at a high level.

Figure 6: Structural materials strategy

Horizontal wind loading is transferred through diaphragm action of the floor and roof plates. The CLT wall panels resist axial, racking and bending loads – providing lateral stability and transferring horizontal loads into the first floor slab. The analysis for Dalston Lane is based on stability being provided through a number of distributed internal walls (Figure 7), and typically not the facade due to the higher void to wall ratio.

Figure 7: Arrangement of internal stability walls in red

The solid panel construction is extremely robust and versatile. All panel/panel joints are half-lapped and screwed together, and all floor and roof panels are directly fixed to load bearing walls (Figure 8), to form a fully tied and robust structure.

The structure is classified as “Consequence Class 2B” under UK Building Regulations [7]. To comply with the requirements, effective horizontal and vertical ties are provided throughout by the screws and nailed bracket connections.
3.3 Stability connection design

Each stability wall is designed to cantilever from the level below. Lateral loads are transmitted through the floor plate as shear into the head of a shear wall. This results in a moment being generated at the base of the panel, and a resultant tensile and compressive force.

The compression component is transmitted in bearing through the floor plate into the wall below (discussed further in section 3.3.4). Off the shelf high capacity nailed brackets take the tensile component through a bolt in the floor, into brackets below the floor plate and back into the wall on the level below (Figure 9).

At the ground floor, a similar configuration of shear connectors and uplift brackets are fixed to holding down bolts, cast into the concrete base (Figure 10). This strategy of using simple details to solve complex issues minimised the requirement for bespoke fabricated details.

3.3.1 Lateral stiffness

The combined behaviour and stiffness of the walls and connection system is fundamental to the lateral stability of the structure. When a series of these cantilevered walls are built from a concrete podium, as in Dalston Lane, the lateral deflection of the system is a result of four main components (Figure 11); tensile connection elongation, bending and shear deformations in the panel, and transfer podium deflections.

For Dalston Lane, deflection due to connection spring stiffness was found to be the largest component at approximately 50% of the combined deflection, and necessitated the following approach.

3.3.2 Analysis model

The ‘traditional’ approach to analysing this wall-connection system is to use an elastic triangular stress distribution beneath the wall (Figure 12), and design the connections accordingly.
This approach implicitly assumes that the neutral axis of the system is at the centre of the wall. However this does not account for the differing stiffness of the panel in compression and the connectors in tension. When this is considered the neutral axis shifts to approximately 1/10th of the way along the wall. This is consistent with testing of stability walls [8], which indicates that panels tend to ‘rock’ about the end point.

Accounting for this effect provides a longer lever arm over which the connectors act. One can also consider that in the ultimate limit state, some plasticity in the metal connectors can be expected. Rather than the triangular stress distribution, redistribution of forces into adjacent connectors to form a rectangular stress block is assumed (Figure 13). By adapting the panel-connection system model to allow for these effects, a more realistic estimate of the anticipated wall movement can be obtained, and connections designed more efficiently.

Rothobraas WHT 620 brackets were typically used throughout on the main stability walls. The manufacturer has tested their behaviour under tensile loading [9], and provides an empirical value for the stiffness K_{ser}, as well as verification of the ductile behaviour in the ultimate limit state (Figure 14).

3.3.3 Sway deflections
With the connections efficiently designed at ULS, the overall stiffness of each wall system can be fed back into the global stability analysis model. This enabled accurate assessments of sway deflection to be undertaken. It was identified in several locations that stiffness of the walls needed to be increased; wall thicknesses were therefore increased and additional connectors included, stiffening the system.

3.3.4 Grout plug reinforcement
When utilising CLT in platform construction, an important consideration is the deformations that can occur at the wall-floor-wall interface due to perpendicular to grain crushing of the floor panels and long term creep. These settlement deformations can have implications on façade behaviour.

On lower rise structures this movement can be estimated and allowances made in finishes and cladding to accommodate it. However, for Dalston Lane the magnitude of the predicted movements was such that an alternative strategy was required.

The solution developed saw floor plates stiffened and strengthened locally with factory routed holes filled with non-shrink grout on site, as shown in Figure 9. This allowed for the direct transfer of compression forces between wall panels, thereby reducing overall vertical deformations of the structure.

A series of mock-ups of the construction details were developed to assess the practicalities of manufacturing and assembling the proposed details. The team worked closely with CLT fabricator Binderholz to determine the size, shape and tolerance of penetrations that could be cut in the factory. A full size component of a connection was then built, to better understand the process and develop appropriate on site quality control procedures for its implementation.
Experimental testing was also undertaken with the University of Cambridge [10] on full size specimens comparing the load/displacement performance of both un-strengthened and strengthened arrangements. The testing was displacement controlled, as the behaviour of the samples under load was unknown.

In the un-strengthened testing, the samples behaved elastically up to an average applied load of 883kN (736kN/m). At this point large splits formed (Figure 15), and further loading resulted in large displacements with some increase in capacity, with samples exhibiting ductile failure.

![Figure 15: Floor panel load displacement testing (un-strengthened test shown)](image)

In the strengthened testing, the average failure load was 1805kN. At this point some samples failed with large pieces of grout flaking off. As displacement and loading continued beyond this point, local buckling of the wall occurred in some locations directly above the grout (Figure 16). No strength gain beyond failure was observed, making this a brittle failure.

![Figure 16: Strengthened testing at failure (buckling and grout crushing shown)](image)

The results verified the design proposals, showing that the instantaneous deformations under loading were considerably reduced in the strengthened test sections. The loads at which significant deformations occurred, due to failure of the grout blocks, were double that of the un-strengthened sections (Figure 17).

![Figure 17: Load/displacement plot for un-strengthened (1) and strengthened (2) samples](image)

### 3.4 Fire strategy

Unlike some other countries, the UK fire regulations do not impose an upper limit on the number of storeys that can be built using a purely timber construction. The CLT structure is designed to achieve a 90 minute fire rating as per Building Regulations Part B2 [12], which is primarily achieved through a combination of two layers of Type F plasterboard, and allowing the CLT panels to char for around 41 minutes once this has fallen away.

The use of five layer panels means that even after a 90 minute fire, three layers of timber typically remain, and the walls have sufficient structural capacity for the reduced loading in the fire design case (Figure 18).

![Figure 18: CLT wall charring in fire](image)

### 3.5 Design for Manufacture and Assembly

Modern Building Information Modelling (BIM) processes and an understanding of the manufacturing process enabled a single 3D structural Revit model to be developed from scheme design to construction (Figure 1). A coordinated design development approach with the architect and MEP engineer enabled all the door, window and MEP penetrations to be modelled in advance in the structural model.

This model was then handed over to the CLT subcontractor at construction. The CLT fabricator, Binderholz, used an IFC export of the Revit model to develop the panel splitting and cutting patterns. This data was then fed directly into the CNC routers to fabricate panels. Meanwhile, BKS took the connection designs
and added further detail to the existing model, with every bracket, screw and nail modelled and set out.

The panel fabrication model and connection models were then federated. On site, BKS used this highly detailed asset to order the exact number of connectors required. Interrogation of the 3D model makes it straightforward to set out fixings, and quickly understand the details in any given location.

4 CONCLUSIONS & LESSONS LEARNT

4.1 On site challenges
A number of unique challenges were encountered during the construction works.

4.1.1 Grout installation
The grout pocket detail (Figure 19), while structurally adequate, required special care on site. The grout must be flush with the top of the slab to guarantee a good contact with the CLT walls, and to ensure load transfer across the floor plate without deflection. On site it was found that ‘non-shrink’ grout still tends to shrink when applied in thicknesses of 140mm or greater, possibly due to absorption of moisture by the timber. A two pass approach was adopted, where the routed hole is filled with grout, allowed to cure, and then a second final layer applied, which is trowelled off for a level flush finish.

*Figure 19: Grout pocket installed beneath wall*

The cutting of the holes for grout into the panels also introduced some problems. Depending on the size and location of pockets, the routed holes often interfaced with panel half laps, butt joints, or other panel edges. This resulted in openings cut into the panel very close to edges, leaving only a thin strip of timber at the edge of the panel, which often became damaged in transport or erection. Closing up the gaps was then necessary before the fluid grout could be poured in.

4.1.2 Erection tolerances
While CLT is factory cut to a precision of ±2mm, additional tolerances need to be made on site to allow for erection. Slight misalignments of panels, uneven bearing surfaces and practical limits on the accuracy of measuring position and verticality tend to accumulate and can result in the panels not fitting together exactly as modelled. One result of this is that floor levels may creep upwards, and over a number of levels the accumulated growth can be appreciable.

This effect was identified on Dalston Lane, and to overcome it wall panel heights at each level were reduced by small amount during machining to offset the vertical deviation. Close monitoring of floor levels was essential to identify the issue and track the adjusted levels.

4.1.3 Holding down bolt detail
As discussed in section 3.3, nailed brackets transfer uplift forces into holding down bolts cast into the concrete podium slab (Figure 20). Cardboard cones allow for a reasonable level of tolerance in the holding down bolt position after being cast, however a large number were required (typically arranged at 300mm spacing along both sides of the stability wall in the highest loaded areas).

Accuracy with the depth and position of anchors was poor, in several cases necessitating redesign of connections using post-fixed anchors, or drilling of slab holes and fitting through bolts. Whilst this is not a timber specific problem, it is worth noting that post fixed anchor details avoid many of these issues, despite not typically provide the same uplift capacity. Robust quality assurance procedures are always necessary, particularly at material interfaces.

*Figure 20: Cast in holding down anchors before wall erected*

4.2 Lessons learnt

4.2.1 Stiffening detail
The grouted panel stiffening detail was complicated to design and implement on site, which might make alternatives more suitable for similar structures.

The effectiveness of this detail at transferring compressive loads through the floor plate is largely dependent on the area of grout in contact with the end grain lamina of the wall panels above and below. When
routing the grout openings, fabrication and erection tolerances must be considered to avoid the loss of grout down the side of the wall panel. This means that in practice holes must be undersized, and the external lamina of CLT is not fully utilised (Figure 21).

![Figure 21: Underutilised bearing capacity of CLT panels due to tolerance and erection limits on grout plug size](image)

In addition, the grout takes up a large amount of the zone traditionally used for screw connections from floor to wall panels. This means that the uplift and shear connection has to be achieved using fewer high capacity screws or additional brackets.

Future projects might consider the use of alternative details, such as castellated wall panels [11], factory installed hardwood plugs, or stub steel sections. Each of these come with their own advantages and disadvantages when evaluated holistically.

4.3 Whole building analysis models

The method of analysing individual shear walls in a building is somewhat conservative. While the development and implementation of the partially plastic shear wall analysis method discussed in section 3.3.2 yields improvement over simpler methods, there are still other effects that could be considered as beneficial to the stability system of the structure. These include:

- The coupling effect of lintels between shear walls and on façade walls, with a whole façade tending to act as a moment frame
- The additional stiffening of return walls in stability systems, creating partially effective T or I sections on plan
- The stiffening effect of other materials, such as the masonry façade

Research into the possible impact of these effects could result in a more efficient design. In practice, applying this to a building the size of Dalston Lane or larger can only be achieved using finite element modelling methods. Procedures to correctly define and apply the non-linear parameters of partial fixity at each junction in a model are crucial to the accuracy of the final results.

4.4 How can we go taller?

4.4.1 Fire

For a taller building with habitable floor levels above 30m, UK building regulations [12] stipulate that the structure must achieve a 120 minute fire rating. In a CLT structure this becomes much more onerous, with any walls under 160mm thick typically being governed by fire design, reducing the economic viability of this structural system. Further research into the protection of mass timber walls in fire, for example through the use of alternative fire boarding, could improve the cost-effectiveness of timber structures greater than 30m in height.

4.4.2 Stability

The methods and experiences described in this paper to design the stability system of Dalston Lane have demonstrated that CLT platform construction is nearing the limit of its applicability for tall timber structures. It may be possible to build a similar structure, one or two storeys taller, perhaps by more detailed consideration of the effects described in section 4.3. However taller timber structures will require alternative structural systems to provide adequate resistance to wind loading. Structural solutions could include:

- Glulam exoskeleton bracing systems [2]
- CLT shear walls coupled to a core [13]
- Hybrid systems (e.g. concrete cores with CLT walls)

4.5 Conclusion

The use of CLT on the project was highly successful, addressing numerous site constraints and maximising usage of the space. The team overcame a number of unique design challenges. Important lessons were learnt, both in the design and construction stages, which can inform future projects of a similar nature. With further work and research, the authors are optimistic that the industry can continue to grow and develop, pushing the boundaries of timber construction to new heights.
ACKNOWLEDGEMENT
We would like to acknowledge the invaluable input of Ben Price of BKS, and Dave Lomax and Kieran Walker of WTa.

REFERENCES