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# CLT-Steel Composite Slimfloor Construction: An Investigation of Mechanical Behaviour Using Finite Element Software

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## ABSTRACT

A new method of constructing multi-storey buildings has been investigated using Cross-Laminated Timber (CLT) panels acting as floor slabs with a steel framed support structure. Conventionally, concrete slabs are used for floor systems but timber panels weigh approximately one third of the equivalent concrete slabs, and hence the superstructure mass is significantly reduced, smaller sections can be used and thus less steel is needed overall. Further economy can be gained if composite action that can be generated between the CLT flooring and supporting steel beams. This paper presents the findings of a detailed finite element study to examine the composite performance of steel-CLT beams taking into account the method and degree of shear interaction, the determination of a suitable effective to width to account for shear lag, the position of the CLT relative to the beam (i.e. above the beam as in conventional composite construction or within the beam depth to form a slimfloor type system) and the layup and orientation of the laminations in the floor units. After briefly outlining the advantages of using a CLT-steel hybrid system, this paper explains how a detailed numerical model which captures the key features of the system was developed and validated and then present the results of a parametric investigation into the generation of composite action in a CLT-steel hybrid system.

**Keywords:** timber, CLT, hybrid, composite

## 1 INTRODUCTION

A new method of constructing multi-storey buildings has been investigated. It involves using Cross-Laminated Timber (CLT) panels acting as floor slabs with a steel framed support structure.

In 2013, the overall floor area constructed in all multi storey buildings in the UK amounted to 8 million m<sup>2</sup>, 68.2% of which was steel frame construction [1]. Conventionally, concrete slabs are used for floor systems, either as precast units or cast in-situ on profiled metal formwork. Timber panels weigh approximately 1/3 of the equivalent concrete slabs, and hence the superstructure mass would be significantly reduced if CLT floors were used. With reduced floor mass, the superstructure itself requires smaller sections as the dead load is reduced, so less steel is needed overall, and also the foundations can be smaller and simpler. Research by Asiz & Smith has shown that CLT panels can be secured to steel with simple screw fasteners [2]<sup>2</sup>, so construction can be a quick, simple and a dry process. All these factors combine to make the proposed system potentially a much more efficient construction method, in terms of time, cost and environmental impact.

Using simple fasteners such as screws as the connection between floor panels, and provided the steel frame is bolted together rather than welded (as is typically the case in the UK), a CLT-steel hybrid frame offers great potential for designing for large scale material reuse. The concrete slabs in existing composite construction systems are very difficult to separate from the steel members, so reuse rates are low. Embracing deconstruction and reuse mitigates any increased initial material costs and fully exploits the environmental benefits.

The use of asymmetric beams would allow the panels to rest on the bottom flange and mostly within the depth of the beam in a slim-floor arrangement, see *Fig. 1*, thus allowing the system to

compete with popular RC flat slab construction. This configuration gives practical benefits, as the slab can provide a safe working surface immediately after fixing, and enjoys the benefits of established slim-floor methods. The flat soffit of slim-floor systems (devoid of downstand beams) gives easier service integration allowing a flexibility in how the space is used [3].

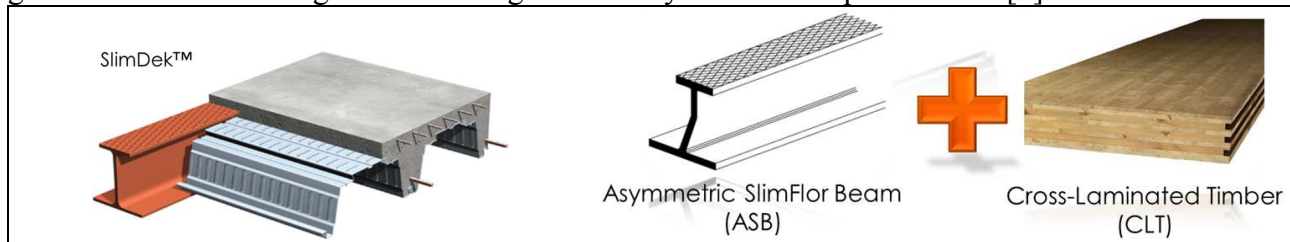


Fig. 1. Substitution of cross-laminated timber in a slim floor system

The first author has previously performed a comparative design study of CLT panels supported by ASBs against a conventional slim-floor system [4] and later extended this to a case study building designed in the proposed hybrid system, as well as in reinforced concrete and conventional composite construction [5]. The compared structures were designed to be functionally equivalent, and the material quantities for each design summarised and the environmental impacts calculated. Assessment of the embodied carbon and energy of the systems was made using Bath University's *Inventory of Carbon & Energy* [6], a well-known dataset with the CLT data coming from the EPD of a Canadian product, Nordic X-Lam [7], and showed a 50% saving in embodied carbon compared with a conventional steel-concrete solution.

In order to achieve the most efficient use of the materials involved, a degree of composite action between the steel and timber is sought. The research presented examines the potential of steel and timber working compositely and the most suitable methods of achieving this.

## 2 MODELLING CLT

### 2.1 Existing CLT models

The modelling of Cross-Laminated Timber can be split into two aspects – the modelling of the constituent timber, and the modelling of a CLT panel. The CLT handbook [8] has a useful collection of the main aspects relating to using CLT in construction, and includes sections on modelling CLT in bending and in-plane shear (lateral loading on walls). Unfortunately, the methods collated are not appropriate for this research project – bending is approximated using the shear analogy method which, whilst giving good predictions of the bending of a CLT element, cannot be used to develop an effective width or be easily modified to incorporate a stiffening steel beam. The CLT handbook guidelines are based on the findings of Blass and Fellmoser in 2004 [9] and Gliniorz, Mosalam & Natterer [10].

In the development of an innovative timber-steel hybrid floor system, Loss [11] has devised a means of modelling the influence of CLT panels on the composite section. Loss used the analysis software SAP with CLT panels represented by plate elements. For this, the gross properties of layered CLT were homogenised into its overall stiffnesses in the two planar axes. This simplification was possible because the CLT sits above the steel member with which it is acting compositely, thus the entire depth of the CLT is in compression. However, in the proposed slim-floor steel-CLT system described in this paper the laminated timber may be subject to tension as well as compression and a more detailed method of modelling the CLT is required.

### 2.2 FE software

FE modelling was performed using version 14.5 of ANSYS. This version includes the Workbench interface which makes setting up the 3D geometry of the problem less time consuming. It is possible to create user-defined materials, and it supports anisotropy and plasticity. ANSYS allows composite materials with different meshes to be assembled together, and also has a Crack Modelling feature that could be applied for investigating delamination, though the crack position must be manually input [12]. This software was chosen due to its availability at the University of

Sheffield, and its status as a commercial software meaning its core features will be common to the majority of software available to practising engineers.

### **2.3 Modelling failure**

The CLT panel is a composite made of a series of timber laths connected to one another by glue layers. Failure of the panel can arise in the timber, in the glue, or at the interface between the two. Similarly, the connectors (inter-panel or shear connectors) can fail in the timber, in the connector, or by a combination of the two through loss of interaction. Timber failure is a deeply complex topic due to the anisotropic nature of timber and the manner of failure. In compression (regardless of direction), the failure process is one of progressive crushing, and hence has a degree of plastic deformation and strain softening. The FE software, given appropriate parameters, can capture this fairly readily. In tension, failure is by sudden snapping of fibres, which is more akin to elements ceasing to exist. Whilst there are means of incorporating this into the CLT modelling philosophy (“Element Death”), this created instability in the system.

The research of Hong and Barrett [13] provided guidance on the modelling technique for addressing the connection between the steel and timber. Investigating FE modelling of nailed connections in wood, Hong and Barrett note that the localised crushing in nailed connections relates more to the contacting wood fibre performance than the overall compression behaviour of the wood. Hence it was necessary to have a distinct material property definition for the region immediately surrounding the connection points in order to pick up the localised behaviour. Then the more general material model developed was used for the majority of the panel.

## **3 VALIDATION**

### **3.1 In-plane shear response**

The most important behaviour to capture is the in-plane shear response of CLT. This parameter is critical in determining how much of the CLT contributes to the composite section, as it influences the effective width through the phenomenon of shear lag and therefore has a bearing on the degree of composite action that can be determined. There is limited experimental data available in the literature covering this aspect of CLT’s mechanical behaviour. Ashtari [14] faced a similar issue in seeking to use ANSYS to model the in-plane behaviour of CLT panels to assess the diaphragm capabilities of a CLT floor slab consisting of multiple panels. The mechanical properties used in her formulation were based on the properties derived by Gsell in 2007 [15]. However, these homogenised properties were derived through modal analysis of the panels and verified by out-of-plane bending tests alone, leaving the in-plane properties unverified by experimentation. As such, these research projects were unsuitable for use as sources of CLT panel property data or for verification in this project.

Direct testing of in-plane shear properties can be found in the research projects of Gubana [16] and of Bosl [17]. In Bosl’s PhD thesis, a series of tests on CLT were performed to derive test data to incorporate into a numerical formulation. One such test format was of CLT in planar shear. A square piece of CLT, was subject to diagonal tension in a custom made rig. Details of the CLT panel dimensions and the test rig were included, and the outputs are in the format of load-displacement (see *Fig. 2*). For simplicity, the panel was modelled using the global axes of ANSYS, so that the edges of the panel run parallel to the unit vectors of the global co-ordinate system. The tests performed by Bosl applied tension at 45° to the planar vertices of the panel. In order to apply the load in the same manner within ANSYS, the load had to be decomposed into the equivalent component vectors in the planar axes to define the vectors of loading through each timestep of the analysis. Fixed constraint was applied in one planar corner of the virtual panel (representing the fixity at the corner at the base of Bosl’s test setup), with the tension load applied in the corner diametrically opposite.

Timber Engineering STEP 1 [18] explains that for crosswise jointing with adhesives, as is the case in CLT panels, the adhesive needs to have consistent shear strength parallel to the grain and similar tensile strength in the transverse direction so that the glue will not be likely to fail before the timber.

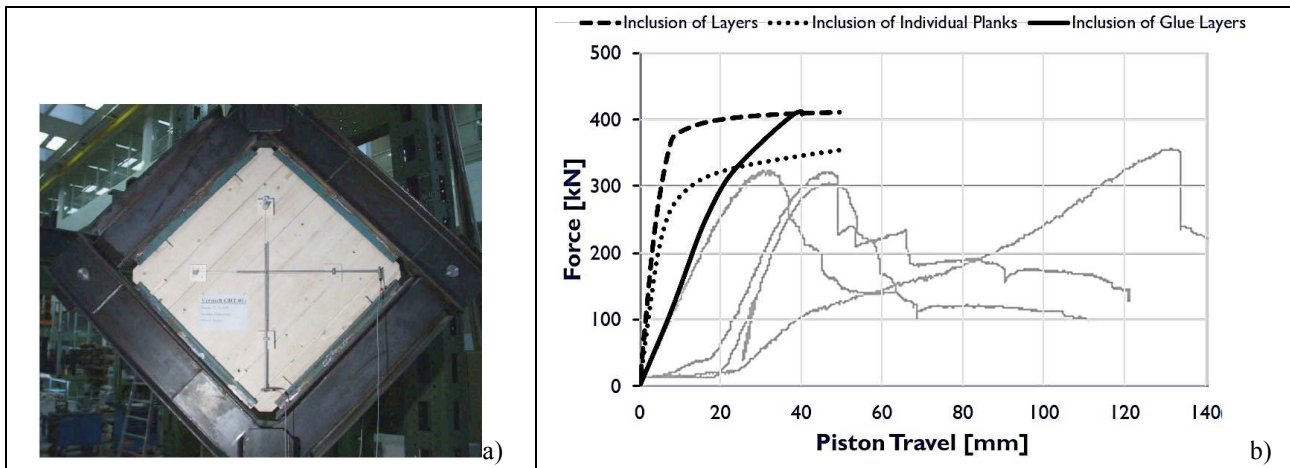


Fig. 2. a) Bosl's test rig [17] b) Shear force-displacement

It is noted, however, that due to the crosswise nature of the bonded connection, there is potential for the glue to be subject to the tensile forces that the timber can withstand in its *longitudinal* axis, which is many times stronger than that in the transverse direction. Glues in FE modelling are most often represented by a perfectly-plastic relationship once the yield stress has been exceeded. Whilst this is appropriate for determining failure of glues, this does not serve the purpose of demonstrating the flexibility that occurs in CLT panels as a result of the glue layers. For this reason, the glue layer included in the models was elastic. The glue used in the most modern CLT panels are of the polyurethane (PU) type. A technical report by Julia de Castro San Roman detailed mechanical properties of glues, including polyurethane, through physical tests [19]. As shown in Fig 2 (b) the FE model with individual laths and a glue layer resulted in close approximation of the panel initial stiffness, with the results of Bosl's testing shown in grey.

### 3.2 Out of plane bending

In addition to replicating the mechanical behaviour observed by Bosl, the new material model and modelling approach was applied to a test setup used by Hochreiner et al. [20]. In this work, a square panel of CLT, fixed on four sides was subjected to a central patch loading. The deflection at the centre was measured against the imposed load. In a setup such as this, the central deflection is linearly related to the imposed load. Therefore, one can infer that the onset of non-linearity in the load-deflection trace is the onset of plasticity (which is assumed as failure) in the system. In an FE model of the setup, matching the slope of the load-deflection curve and the onset of plasticity effectively captures the behaviour. Hochreiner's setup was modelled in ANSYS and the model was able to replicate the deflection slope (see Fig 3). It was not able to capture the post-yield behaviour though, as can be seen in Fig 3(b), where C35-5 and C18-1 refer to particular tests performed by Hochreiner.

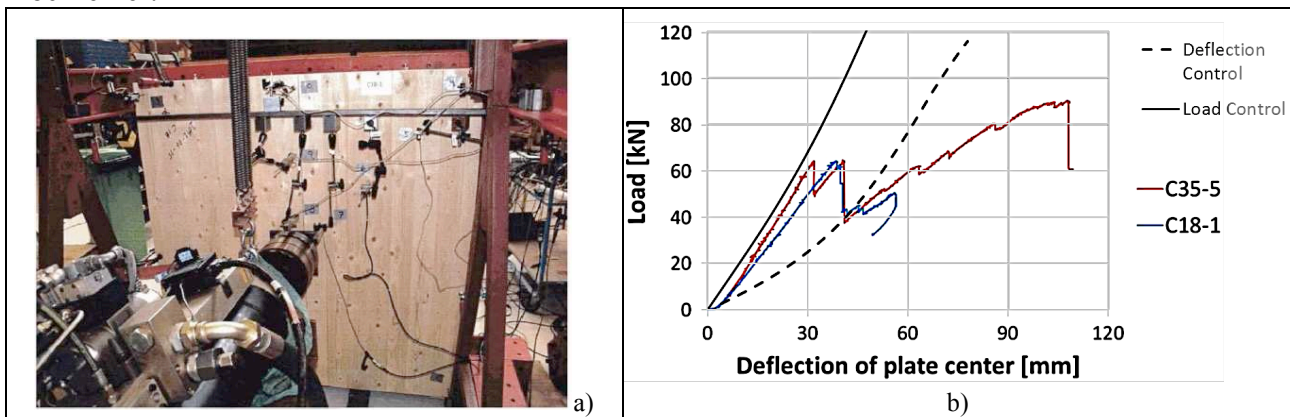


Fig. 3. a) Hochreiner's test [20] b) Load-deflection FEA of test results

Rather than demonstrating an overall plateau in the panel load-displacement curve, the load continues to increase with, in fact, an increase in gradient, even after plastic strain is detected in the timber in the model. This is most likely a result of redistribution of stresses into the glue as the timber in the model reaches its capacity – the glue layers do not have a yield point in the model, therefore will attract more and more load preventing capture of the collapse. Since in the CLT models any plastic deformation constitutes overall failure, over-estimation of the system capacity should not be an issue.

## 4 SHEAR CONNECTION

Due to the particular juxtaposition of CLT panel and steel beam in the slim floor system considered, the shear connection lies in an area of tension under gravity load. As timber fails in a brittle manner under tension shear connector behaviour is a critical factor. In preliminary investigations when the CLT was modelled to the detail of separate layers but not individual laths, it was not possible to determine composite benefit with a complete shear connection in ANSYS because of the orientation of the layers in CLT, with the loads of the shear connection in the tensile zone pulling apart the timber along one of the weaker radial or tangential axes. The beam and panel were modelled as bonded together in ANSYS, but because of the lack of ductility offered by a bonded connection and the large difference in Young's moduli, the deformation of the steel beam caused plasticity in the timber as it was forced to match the deformation. Although under bending the level of deformation in the steel resulted in stresses below the yield point of steel the equivalent deformation of the timber strained it beyond its yield point. Because timber tensile failure is a snapping of fibres, when ANSYS displays plastic strain in the model, this represents sudden and complete removal of all composite benefit when tensile stresses are being applied. Whilst the introduction of individual laths in the CLT models meant the CLT panel was now more resilient to the deformations dictated by the extension of the steel beam in the tensile zone (through increased separation of laths rather than stretching of timber), the experiences of inducing failure by too stiff a bond highlighted the influence the shear connection can have on the behaviour of a composite system in ways that may not be first envisaged. Similarly, this also highlighted the benefit to including the stress limits of the timber in the different axes and a means of reporting when they are exceeded.

Another factor to consider is the anisotropy of timber. At the level of the shear connection (underside of the slab), the timber is orientated with the grain direction (longitudinal axis) in the direction of span of the panel i.e. perpendicular to the beam's longitudinal axis. Bending of the beam causes tension and compression running along the beam length. Because of the relative orientations, the fixture of the CLT to the beam acts to pull the timber apart along its weaker axis resulting in plastic failure in the models very quickly.

### 4.1 Nailplates

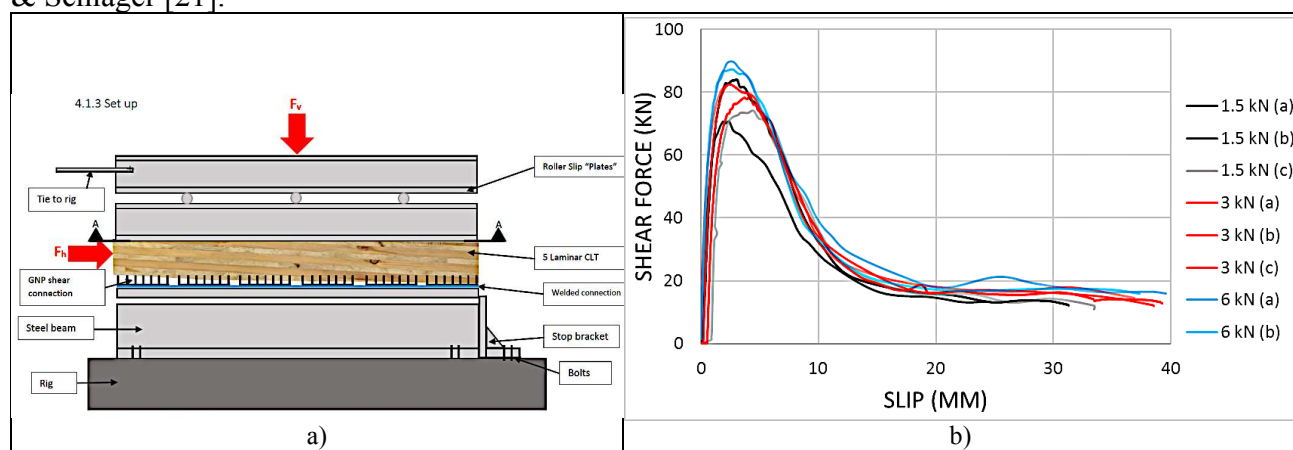
Nailplates are an established wood connector, and by positioning them on the bottom flange of the ASB, it was hypothesised that the timber slab could be dropped into place using a crane and the self-weight of the panel would cause the embedding of the nails into the timber matrix. It was further postulated that in forming a composite connection, the slip stiffness would be dependent on the embedment depth of the nails, and hence as more vertical load is applied, the embedment would increase, creating a stiffer connection in situations of higher load, where composite action is most helpful.

It is common practice to assess the behaviour of a shear connector via a slip test. In most cases this is arranged as either a single or double shear test (with two lines of connection for the double shear variant), and shear loading applied vertically, however a more representative and informative setup for this potential shear connector was established. One of the hypotheses of this part of the research was that in the proposed construction system, increased imposed load would drive the nails into the CLT, with the level of vertical load governing the embedment depth of the nails into the CLT, and hence the strength and stiffness of the joint. The presence of vertical loading is what induces the

curvature and combined action of the beam and slab that defines composite action. Therefore, it is more reflective of the service condition to apply shear loading together with vertical compression.

A small experimental test series was conducted [22] to determine the strength of nail plate joints in CLT, determine the slip stiffness to inform the level of composite interaction that can be determined, and to ascertain whether the failure would be ductile or brittle. Nail plates were welded to the outer face of the upper flange of small column sections (see *Fig. 4.(a)*) 4no. nail-plates per test sample. CLT panels (donated by KLH UK), were cut into sections of approx. 1m x 300mm, and placed onto the nail plates. To reflect the arrangement of elements in service, the bottom layer of the CLT was orientated orthogonal to the direction of shear load. The magnitude of preload was one of the parameters that changed, representing situations of different assumed panel spans.

The load-slip results are presented in *Fig. 4(b)*. It is difficult, and inadvisable to draw definitive conclusions from such a small sample set, particularly for tests involving timber elements. The traces show elastic behaviour up to between 45kN and 55kN, before the onset of a degree of non-reversible deformation marked by a reduction in the gradient of the curve. The connection joint trace shows a peak load of approximately 80kN (on average), after which pronounced strain softening occurs down to a plateau of plastic deformations at a lower load of approximately 15kN. The strain softening observed in every test conflicts directly with the experimental findings of Blass & Schlager [21].



*Fig. 4.* a) Test apparatus b) Load slip characteristics, grouped according to vertical pre-load

It was not possible to determine conclusively how the amount of pre-load affected the behaviour of the nailplate-CLT connection; a more expansive testing regime investigating this would be required. The tests performed in this study potentially show that the higher the pre-load, the higher the limit of elastic deformation. This could be explained by the tendency of the samples to try to deform by hogging longitudinally, and lifting away from the central nailplates. In such an occurrence, the embedment depth of the nails reduces, making that part of the joint less stiff and leading to non-linear behaviour. With higher preloads, there is greater limiting of this effect, allowing the linear deformations to continue to higher loads. Detail on the effects of pre-load can be found in [22].

## 4.2 Panel to panel connection

If CLT planks are used, the panel-to-panel connection in the regions of the CLT panels that are below the neutral axis is of concern. Whereas in conventional concrete-steel SlimFlor systems, the concrete below the neutral axis is ignored and assumed to have no tensile capacity, timber has significant capabilities under tension. Ignoring this would waste section depth and material, which impacts both the cost and environmental impact of the structure, and therefore the tensile zone of the CLT slabs must be engaged to generate composite action most successfully.

Models showed an improvement in reduction of deflections as a result of tying together the lower-most faces of the panels, and suggested a suitable connection must be found that, in connecting the panels together, transfers tensile forces across the joint and engages the tensile regions of the panels. This connection also plays an important role in ensuring the robustness of the structure and in transmitting loads to the lateral stiffening structures (cores/bracing etc.). Loss has been working

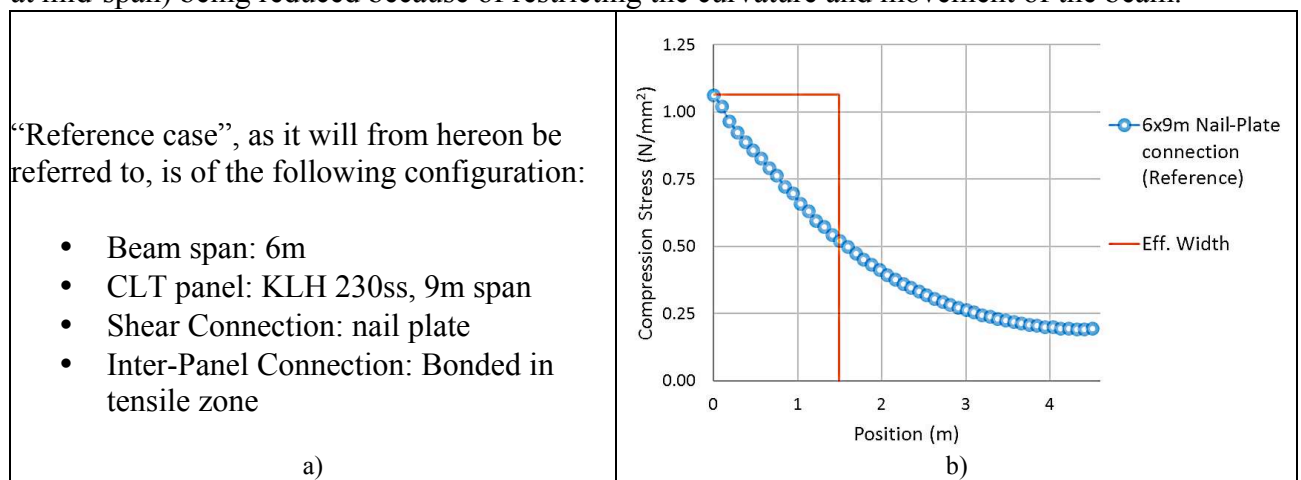
concurrently on the suitability of CLT as floor diaphragms in this way, and performed a programme of tests on a variety of inter-panel connectors [23]. An MSc project [24] investigated the performances of panel-to-panel connection types. Joints were constructed and subjected to bending and shear forces along the panel-panel interface. This sub-study aimed to assess the mechanical behaviour of two types of joint so their load responses could be incorporated into the overall system model. Butt joints and half-lapped joints were investigated due to their relative simplicity to create. To allow for load transfer across the interface, screws are used across the joint, and also to give some local reinforcement. The butt joint offers a generally stiffer connection between panels, with satisfactory ductility and a clearer transition from elastic to plastic behaviour, when compared to the half-lapped joints.

## 5 PARAMETRIC STUDY

### 5.1 Reference case

To provide a comparison point against which the effect of changes and variations in the system characteristics can be measured, a reference case setup was selected, and is outlined here. Each subsequent model created for the study of the effect of different parameters has the same characteristics as this case, except for the parameter under consideration. (The one exception to this is the study of slab position, where the shear connection was assumed as bonded instead of the nailplate, and compared to the reference case with a bonded shear connection.)

The response is sought against an imposed load of 2kN applied as an area load in the model of the configuration described in *Fig. 5(a)*. The beam ends were modelled as simply supported, with fixity in the horizontal plane of its mid-span (due to the symmetry constraints) and freedom to move longitudinally at the end. This was to prevent further stresses being induced or the deflection (taken at mid-span) being reduced because of restricting the curvature and movement of the beam.



*Fig. 5.* a) Drawing; b) Effective width

The output trace of stresses is shown in *Fig. 5(b)* as the blue circles. Also shown is the derived effective width. For this, case the effective width has been calculated as 3m (noting that the trace only displays one side of the beam). This result set will be used as a reference for comparison to determine the impact of various design changes. Deflection was found to be 12.39mm under the defined loading. This compares to 13.12mm that was found when the shear connection was removed and replaced with frictional contacts to represent the non-composite case. This represents a composite benefit of only 5.6% using a nail-plate shear connection rather than a non-composite connection.

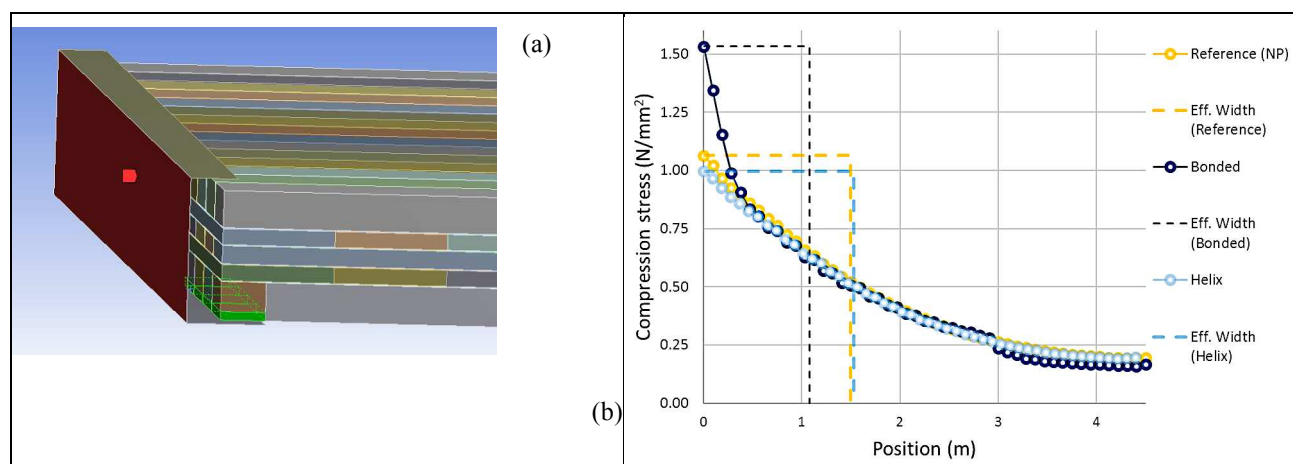
### 5.2 Influence of shear connection

The nail-plate connector material was implemented by incorporating an intermediate body between the CLT panel and the bottom flange of the ASB section, forming the reference case (see *Fig 6. (a)*). The connector was modelled as a solid body that runs along the length of the beam with a



width of 80mm (corresponding to the bearing width of precast panels on ASBs) and a height of 15mm, which is the approximate height of the nail plate's vertical dimension and embedment depth. For the ideal composite connection, the intermediate body was removed, and the bottom laths of the CLT panel allowed to continue and bear onto the ASB bottom flange directly. Contacts were then used within ANSYS to bond the CLT panel to the ASB, replicating a perfect composite connection. In this way, it is possible to both determine the success of the nailplate as a shear connection and approximate the potential composite benefit if the connection performance is maximised.

The trace of longitudinal compressive stress is shown in *Fig. 6(b)* and includes the derived effective widths for the reference case (nailplate, in yellow) against the idealised bonded shear connection. As can be seen by comparing the traces of the bonded and nail-plate connections, there is little difference in the generated stresses in the majority of the panel span. The difference lies in the peak stress induced above the panel-beam interface. With a stiffer connection, this increases the peak stress induced, and because this effect is limited to the zone close to the shear connection, this *reduces* the effective width (the width of panel that would capture an equivalent cumulative force if the longitudinal stress was constant and equal to the stress at the beam interface). In terms of deflection, compared to the reference case's 12.39mm (5.6% composite benefit), applying a perfect shear connection gives a deflection of 11.35mm, which constitutes a composite benefit of 13.5%.



*Fig. 6.* a) FE model shear connection b) Shear connection influence on effective width

### 5.3 Position of the slab

The position of the CLT floor slab, relative to the steel beam, has an effect on the effective width behaviour, and the impact on deflection. Whilst it is clear that having the slab above the beam will increase the composite 2<sup>nd</sup> Moment of Area (by moving more material away from the neutral axis) this will remove the practical benefits of using a slim-floor arrangement. For the purposes of this comparison, the beam modelled remained a 280ASB74, to give a direct comparison (*see Fig. 7(a)*). *Fig. 7(b)* shows the effective width is not significantly changed from the reference case. But for deflections the increase in composite section depth has, as expected, had a profound effect on the overall section stiffness – a deflection of 5.99mm is recorded (a composite enhancement of 54%) which compares to 5.6% for the reference case, and 13.5% for the equivalent slim-floor case with a bonded shear connection.

### 5.4 Panel to panel connection

The panel-to-panel connection had no discernible impact on the peak stress experienced by the CLT panel at midspan. However, the overall stress generated along the panel span was reduced when compared to the reference case, and subsequently the effective width was reduced. This behaviour demonstrates the reduction in stiffness of the overall flooring system through the use of a butt-joint compared to a perfect bond at the panel interface, and is reflected in a slight increase in beam deflection (12.51mm compared to the reference 12.39mm). The composite enhancement potential is reduced by inclusion of the inter-panel connection, an important factor to be aware of when modelling the behaviour.

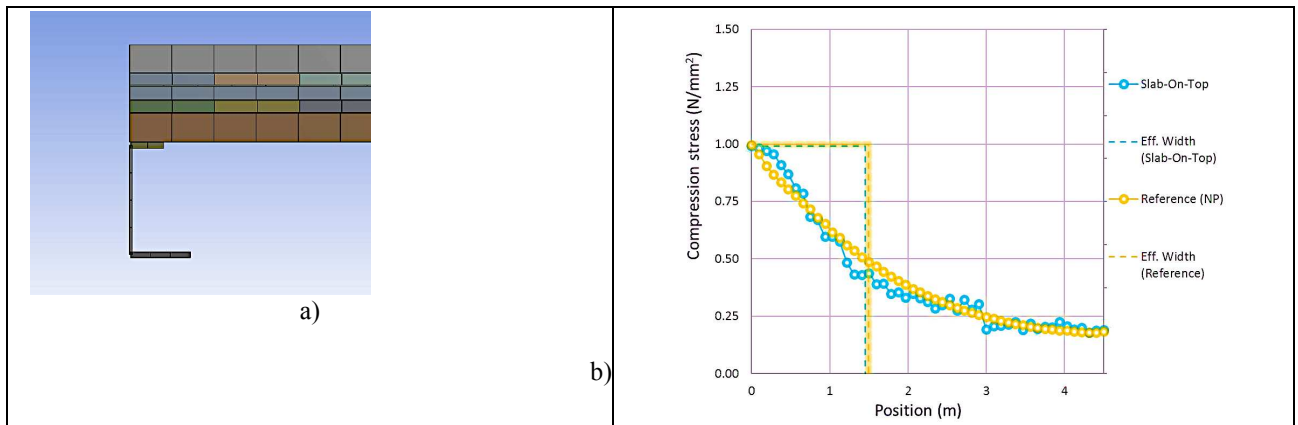


Fig. 7. a) FE model b) Slab position influence on effective width

### 5.5 Panel layup

The standard design lay-up of CLT panels maximises the capability of the panel to span in the direction orthogonal to the beam, but this means the amount of panel that can usefully contribute to the stiffness of a composite section is limited to the two thin, “even-numbered” layers which run parallel to the beam. The KLH 7ss230 panel, used in the reference analysis case, is notionally a 5 layer panel, however the outermost layers are in fact double thickness. The FE model was adapted so that instead of the face layers being of double thickness, the depth of the layers running parallel to the beam are doubled instead (see Fig 8(b)). The overall panel depth is maintained at 230mm deep. The inter-panel connection is maintained as bonded for the bottom two layers and frictional contacts for the upper layers.

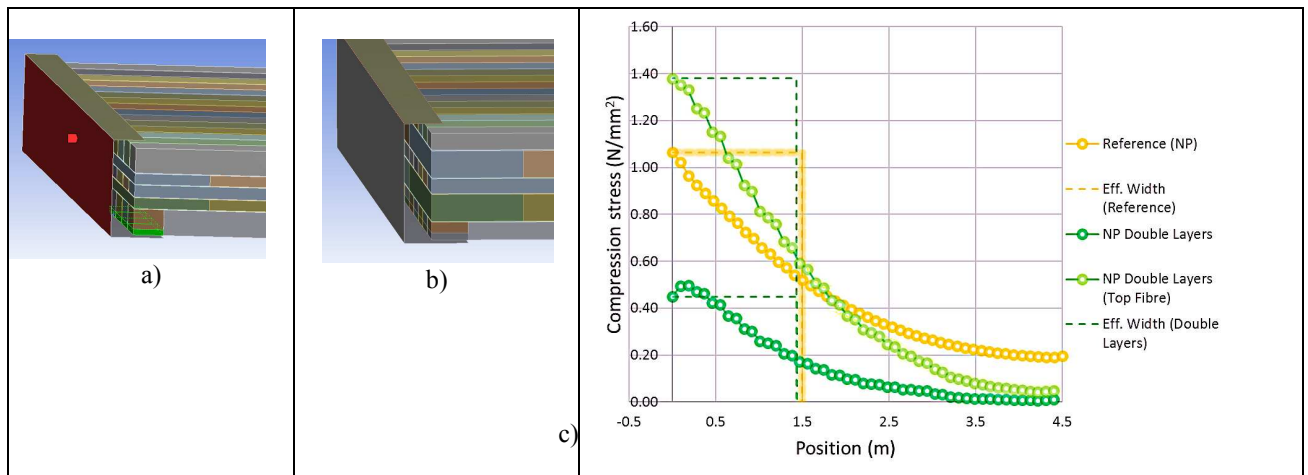


Fig. 8. a) FE standard panel b) FE modified panel c) Influence on effective width

The longitudinal compressive stress trace for the reference case and the revised panel design are presented in Fig. 8(c). The results show the effective width has not been changed through the change in panel layer dimensions and that the interplay of the shear connector stiffness and the longitudinal stiffness of the panel material has more bearing on the effective width than the build-up of the panel. The use of a designed panel increases the composite benefit from 5.6% to 12.6%. The implications of the results are that manipulation of panel layers can greatly improve deflection of the beam. What must be determined is the limit of reducing the layers that run along the panel span so that it can still span the required distances without breaching deflection limits. If necessary, it may be feasible to add a layer to the top of the panel with laths running parallel to the beam to create a similar effect without affecting the span capabilities of the CLT panel. Care must be taken with this as the extra layer will shift the neutral axis upwards and affect the 2<sup>nd</sup> moment of area generated by the layers resisting panel deflection. This area of the design shows promise for optimising the system for greatest performance.

## 6 ACKNOWLEDGMENT

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