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# Experimental study of Moment sharing in Multi-Joist Timber-Concrete Composite floors From Zero Load Up To Failure

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## ABSTRACT:

The critical T-sections of multi-joist timber concrete composite (TCC) floors must be designed at ultimate for support shear force and midspan moment, both of which are influenced by transverse sharing, but to different extents. Prior experimental work has investigated only support reaction sharing and only up to serviceability loads. The present experimental study builds on that status quo by quantifying also moment sharing, via strain gauge layouts at quarter-span and mid-span, along with reaction sharing via load cells at the supports of a multi-joist TCC specimen, over the entire load range up to failure. Use of steel mesh connectors bonded into hardwood laminated veneer lumber joists, and near geometric resemblance to a real building TCC floor recently built in London, were novel features of the specimen. The results show that midspan moment and reaction sharing both vary nonlinearly with load, but in distinctly different ways from each other (with up to almost 20% difference observed between them), in the progression between the uncracked, cracked and connection ductility regimes. In this approach reliable assessment of moment sharing depends on the quality of the recorded strains. Accordingly, the strain data were shown to be of high quality by converting these data to internal stress resultants that were then found to satisfy longitudinal equilibrium. It is concluded that this strain gauge layout is useful for future work aimed at building a database of transverse sharing of moments in TCCs.

**KEYWORDS:** timber concrete composite; indeterminacy; experiment; transverse distribution; hardwood; beech; LVL

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1 **1 Introduction**

2 A timber-concrete composite (TCC) member comprises a timber joist or panel shear-connected to an  
3 overhead concrete slab. The connectors resist separation and slip between the timber and the concrete  
4 and generate shear forces along the interface, by these means significantly increasing the strength and  
5 stiffness of the floor relative to the case where the slab simply rests on the joist. TCC floors in buildings  
6 typically comprise concrete slabs continuous across multiple joists, an example of which is shown in  
7 Figure 1(a), and therefore there will inevitably be load sharing between individual composite T-  
8 sections, such as the one highlighted in Figure 1(a). In this paper, load sharing refers to two key  
9 behaviours, namely the sharing of reaction between individual joists at each support and the sharing of  
10 bending moment between individual composite T-sections at any section away from the supports and  
11 particularly at mid-span.

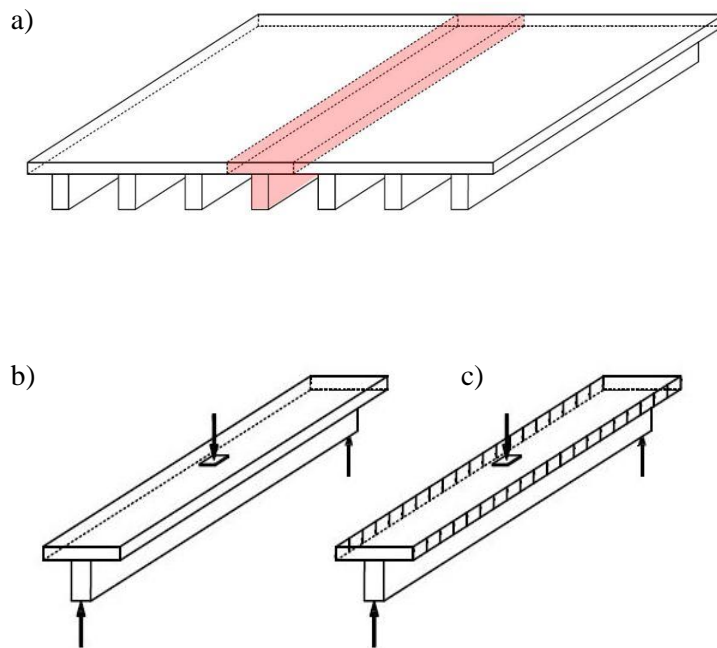


Figure 1 - (a) TCC floor with a single slab spanning multiple timber joists, (b) isolated T-section, (c) isolated T-section with corresponding shear forces along the interfaces

12

13 Now, it is convenient and sensible to isolate a single T-section for design against critical moment and  
14 shear force, as shown in Figure 1(b). What occurs in reality however, is shown in Figure 1(c), with  
15 vertical shear forces present along the imaginary vertical cut faces in the slab between T-sections. Due

16 to these **vertical** shear forces, in general moment sharing between T-sections at any section away from  
17 the supports will not mimic the corresponding support reaction sharing.

18 Notwithstanding this disparity between moment and reaction sharings, the approach in Figure 1(b)  
19 would be justified if the lower bound theorem of plasticity applied, which in itself is crucially dependent  
20 on sufficient ductility in the structural system. However TCCs in reality often show a behaviour which  
21 is brittle or of limited ductility. Hence any disparities between support reaction and midspan moment  
22 sharing cannot be overlooked and instead must be investigated as a crucial issue in their own right. The  
23 outcome of such an investigation can lead to even more effective use of the Gamma Method in Annex  
24 B of Eurocode 5 [1] as a design tool for TCC floors.

25 Research to date, e.g. [2-8], has focused predominantly on single-joist, single span, simply supported  
26 TCCs, which are internally indeterminate due to slab-joist slip. Another study [9] experimentally  
27 investigated the non-linear load responses up to failure of multi-span continuous, single T-section TCCs  
28 which, in the longitudinal direction, were both internally and externally indeterminate. In practice, TCC  
29 floors to date are largely simply supported with slab continuity in the transverse direction over multiple  
30 timber joists. It is the associated transverse indeterminacy which introduces the above challenge of  
31 quantifying load sharing, which has been investigated only in few previous experimental studies,  
32 including those by Monteiro [10] and by Kieslich and Holschemacher [11] who focused on support  
33 reaction sharing in the elastic regime of TCC behaviour.

34 In advancing these studies into the domain of midspan moment sharing from zero load to failure, two  
35 key considerations must be taken into account, namely the dimensional proportions and composition of  
36 the multi-joist TCC experimental specimens. To those ends a study of previous experimental TCC  
37 research shows that the span:depth ratio of the test specimens varied from 6.4 to 22.8, with the majority  
38 in the 13 – 18 range and with the spans in the 2m – 10m range. The average joist depth was 270mm,  
39 with slab thicknesses in the 30mm – 150mm range and an average of 65 mm. Joist spacing (or concrete  
40 slab width in single-joist specimens) ranged from 300mm to 2000mm with an average of 740mm [3-  
41 8,9,12-17,25].

42 Moreover the engineered timbers used previously have typically been softwood glulam, commonly  
43 spruce, with other studies using hardwoods [9,18], cross-laminated timber (CLT) [19] or laminated  
44 veneer lumber (LVL) [20]. The slabs have largely been of normal concrete, with a small number of  
45 studies using lightweight [21], cork-aggregate [22] concrete, limecrete [23], and fibreglass-strengthened  
46 concrete [24]. The most common timber-concrete shear connectors have been dowels, rods, or inclined  
47 screws affixed into the timber, with concrete cast around them [3, 5, 9, 25-27]. They are characterised  
48 by low slip stiffness and low ultimate strength, but good ductility. Glued connections [28] and notches  
49 cut into the timber joist [24-26] are significantly stiffer, yet suffer from lack of ductility. Expanded steel  
50 mesh and plates [29-33] have been used largely in softwoods and exhibit high stiffness compared to  
51 other connection types, with a good shear force-slip ductility plateau and high strength.

52 In cognisance of the above studies, the investigation reported in the remainder of this paper has  
53 incorporated a number of novelties to enable in-depth investigation of load sharing in multi-joist TCC  
54 floors. The key innovations embedded in this investigation were:

- 55 • The combination of *steel mesh connectors* and *laminated veneer hardwood* joists in a large-scale  
56 experimental multi-joist TCC specimen.
- 57 • Experimental quantification, from zero load to failure, of *moment* sharing across T-sections in this  
58 multi-joist specimen, based on implementation of an ad-hoc strain gauge layout on the specimen.
- 59 • Understanding the extent to which the nonlinear variations, with load, of moment and support  
60 reaction sharing are distinct from each other across the *entire loading range* up to failure.
- 61 • Demonstrating that the recorded strains which underpin the above assessments are reliable, by  
62 showing that the internal stress resultants based on these strains very nearly satisfy equilibrium.

63 To those ends, the following sections of this paper describe the details and testing of the large-scale  
64 multi-joist timber-concrete composite floor specimen used to investigate the above points. Also  
65 described are the double shear specimens fabricated to enable characterisation of the longitudinal shear  
66 behaviour of the connections. The test results are then presented and interpreted to provide new insight  
67 into moment and reaction sharing as nonlinear functions of load up to failure.

## 68 **2 Materials and Methods**

69 The details of the present specimen were partly influenced by the design of TCC floors constructed in  
70 multiple storeys of a new building in London in 2018 over 6m spans, using beech LVL joists spaced at  
71 950mm centres, a 95mm thick concrete slab, and high shear stiffness coach screw connectors. The  
72 present specimen was of similar specifications, namely a 4.9m span, beech LVL joists at 700mm  
73 centres, an 80mm deep slab and expanded steel mesh shear connectors of very high stiffness, high  
74 strength and good ductility as shown in Section 3.5.1. Also, the use of hardwood enabled ductile  
75 connection behaviour to become manifest before final failure by flexural fracture of the timber.

76 In what follows, the design and fabrication of the experimental specimens is addressed, alongside the  
77 creation of a testing regime that addresses the key objectives, namely to:

- 78 • Understand the nonlinear variations, with load up to failure, of moment sharing and reaction sharing.
- 79 • Determine the effectiveness of the steel mesh plate connector in multi-joist TCC floor specimens.

80 A novel instrumentation layout was implemented in the specimen to enable inference of moment sharing  
81 at quarter-span and mid-span. That this moment sharing varied with load, as will be seen in Section 3.1,  
82 required the testing regime to record data right up to failure of the floor specimen, something which has  
83 not been previously pursued in the experimental domain for a near full-scale specimen. Due to  
84 limitations of space, logistics, and time, one full scale specimen was fabricated and was comprehensively  
85 instrumented.

86

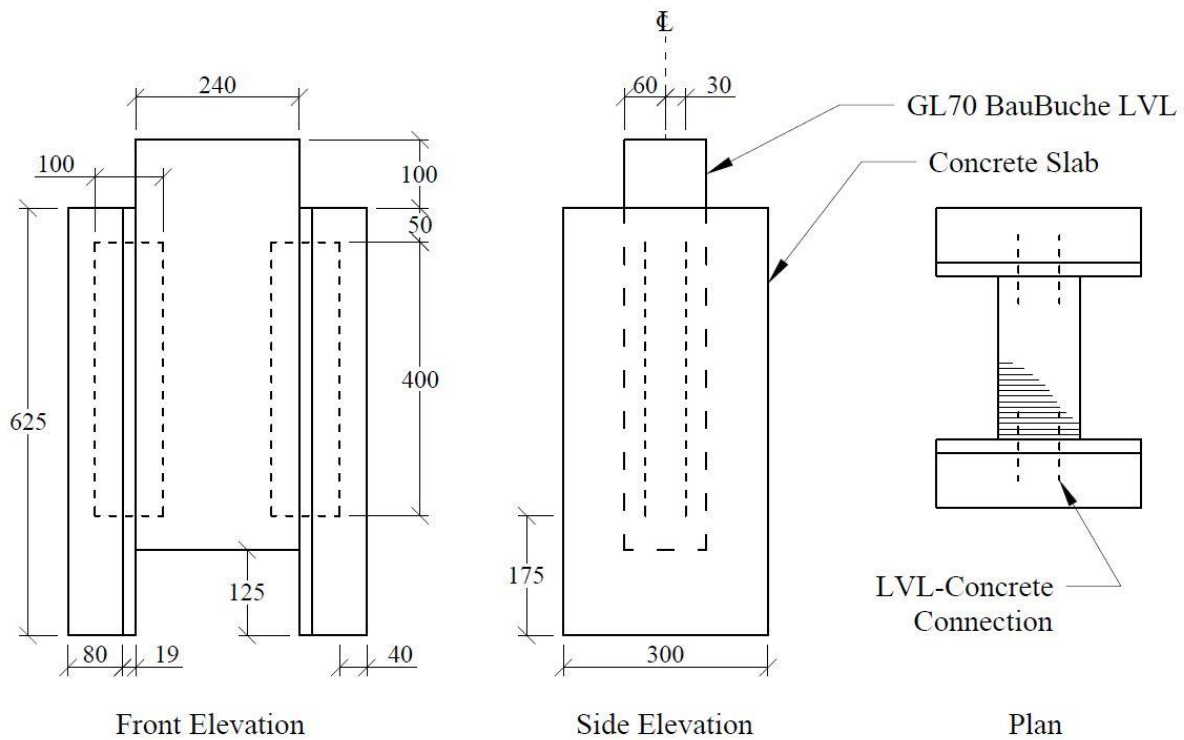
### 87 **2.1 Double shear push-out tests**

88 Double-shear tests were performed in order to define the stiffness, strength, and ductility of the steel  
89 mesh-based connections. As shown in Figure 2, each connection specimen comprised a 120 x 240 x  
90 600mm GL70 type S LVL joist connected on each side to a 625 x 300 x 80mm RC32/40 concrete slab  
91 with a layer of A193 steel reinforcing mesh (7mm diameter at 200mm square centres) at mid-depth to  
92 serve as anti-crack reinforcement during curing of the concrete.

93 Between the timber joist and the concrete slab a 19mm thick timber interlayer, which comprised thin  
 94 layers of GL70 timber around a plywood core, was placed to serve as permanent formwork, thus further  
 95 mimicking site practice. Tests to date on other TCC connections have found that the presence of an  
 96 interlayer leads to significant reductions of slip stiffness and longitudinal shear strength. The interlayer  
 97 was expected to be beneficial in the multi-joist floor tests due to its contribution to bending strength,  
 98 but its impact on connection behaviour was less clear. Visual forensics were employed after the test to  
 99 determine whether the interlayer acted as a soft medium that allowed the connector to fail within that  
 100 interlayer, or whether it provided sufficient restraint to prevent buckling of the connector.

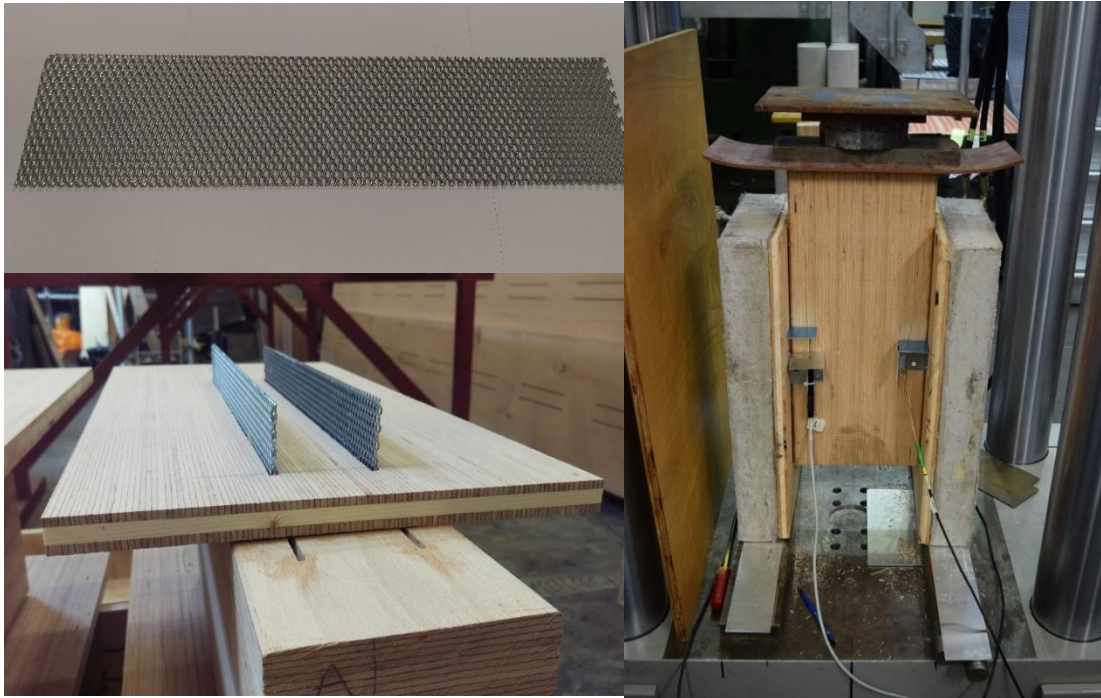
101 One steel mesh panel, 400mm x 100mm in elevation and fabricated from 10-09G pre-galvanised  
 102 expanded steel mesh, is shown in Figure 3(a). Each TCC connector comprised two such panels, inserted  
 103 side by side into each of two preformed grooves in the timber joist, as shown in Figure 3(b). Therefore,  
 104 each connection in the main TCC multi-joist specimen contained four steel mesh plates.

105



106

107 *Figure 2 - Dimensions of the push-out double shear specimens*



109

110 *Figure 3 – (a) mesh connectors, (b) connectors inserted into LVL joist, (c) double-shear specimen ready for testing.*

111 The specimens were tested in a DARTEC 600 kN testing machine, according to the procedure as defined  
 112 in BS EN 26891 [34]. Force was recorded by the machine’s internal load cell, while timber - concrete  
 113 slip was recorded by four linear potentiometers aligned with the centres of the connectors, adjacent to  
 114 each concrete slab on both sides of the joist. The two readings for each connector were averaged to  
 115 allow for possible unsymmetrical behaviour. Figure 3(c) shows a specimen ready for testing.

116

## 117 **2.2 Full-scale specimen**

118 The aim of the large experimental specimen was, from zero load to failure, to enable understanding of  
 119 the midspan moment and support reaction sharing capabilities of TCC floors, alongside determining  
 120 the effectiveness of steel mesh plate connectors bonded into hardwood LVL joists in such floors. In  
 121 order to enable inference of the moment sharing, it was crucial to devise an appropriate strain gauge  
 122 layout. As shown in Figure 4, the presence of strain gauges at 3 distinct levels within the timber joist



123 allowed the assumption of a through-depth linear strain distribution to be checked and also enabled  
 124 quantification of the joist's curvature at the gauged section. In order to check curvature compatibility  
 125 between the concrete slab and the timber joists, strain gauge readings were also required from two  
 126 further distinct levels within the slab. However, due to the presence of the formwork interlayer and to  
 127 the small size of the steel mesh reinforcing bars this was not possible and only one longitudinally  
 128 oriented gauge was placed on top of the slab.

129

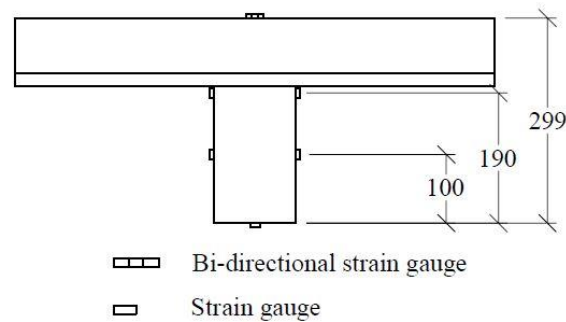
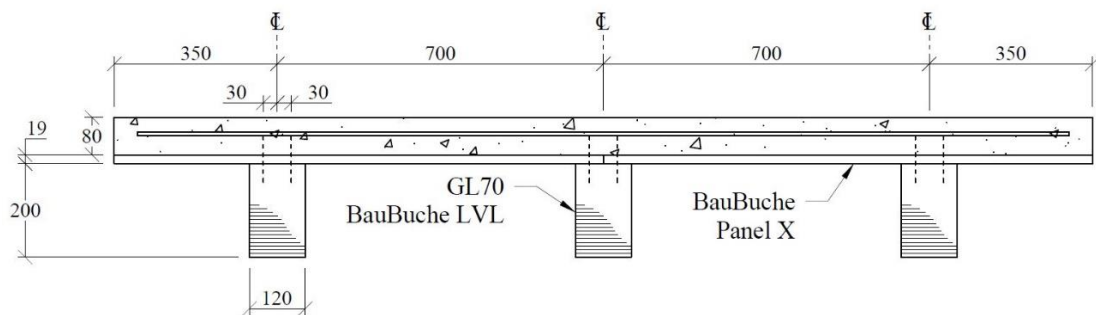


Figure 4 - Through-depth strain gauge layout on TCC specimen

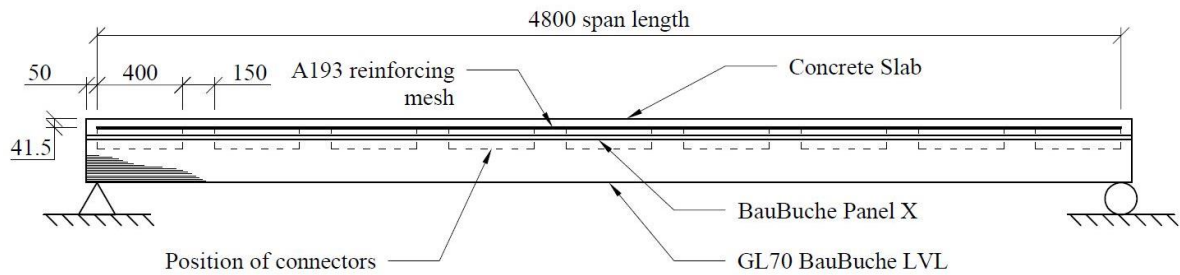
130 The specimen consisted of three 120 x 200 section x 4900mm long GL70 type S LVL joists, spaced at  
 131 700mm centres, with slab overhangs such that each individual 'T-section' also had a slab flange width  
 132 of 700mm. A 19mm thick interlayer (BauBuche Panel X) separated the joists from an 80 deep x 2100  
 133 wide x 4900mm long concrete slab, to the same steel mesh plate connector specification as for the  
 134 double shear tests. Again, an A193 steel reinforcing mesh (7mm diameter, 200mm square spacing) was  
 135 positioned at the mid-depth of the slab to prevent shrinkage cracks. A cross-section and elevation of the  
 136 specimen are shown in Figure 5. The span:depth ratio of 16.3 sits comfortably within the spectrum of  
 137 span:depth ratios of previous research TCC specimens as discussed earlier in the introduction, and the  
 138 dimensions of both the joists and slab fall within the respective previous ranges presented also.

139 A conservative layout of connectors was chosen which, it was estimated, would allow the structure to  
 140 take significant load, whilst still exhibiting ductility towards failure. The suitability of the connector  
 141 layout was determined using a longitudinal shear analysis and estimates of the strength and stiffness of  
 142 the connectors, which were then verified through the above-described double shear tests: a simple FE  
 143 grillage model of the structure (formed of line-beam elements representing the three individual T-

144 sections, connected by transverse elements representing the concrete slab) was made, and a load applied  
 145 under linear-elastic conditions. The shear forces from this model were converted to longitudinal shear  
 146 at the interface via the equation  $q = VA\bar{y} / I$  and compared with the shear capacity of the connectors.  
 147 From this analysis it was estimated that the connectors would begin to yield at approximately 90kN,  
 148 less than the 190kN required to reach the bending capacity of the timber joist, and therefore enabling  
 149 both ductile behaviour and high load capacity.



150



151

152 *Figure 5 – (a) Cross-section, and (b) side elevation of full-scale experimental specimen*

153 Due to space restrictions, this specimen was fabricated in-situ. A temporary formwork was constructed  
 154 above the LVL joists using the interlayer as a base (Figure 6(a)), into which the concrete was then cast  
 155 and compacted. The concrete was left to cure for 40 days before testing began. Figure 6(b) shows the  
 156 freshly cast concrete slab.

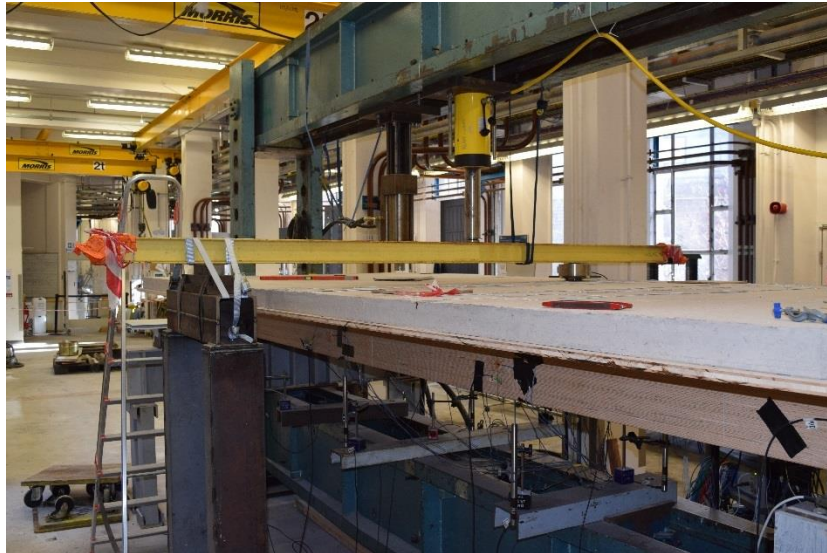


Figure 6 - Formwork for three-joint specimen (a) before concrete pour, and (b) following the concrete pour

157 A testing regime was devised which would enable repeat loading of the structure at distinct locations to  
158 explore the load sharing capabilities of the specimen, without initially affecting the linear-elastic  
159 behaviour of the structure or connectors. This required all loading to be conducted within service level  
160 loads to prevent plastic deformation of the structure. A 20kN load was estimated as being appropriate  
161 for these uncracked stage explorations. Load was always applied onto the slab, first directly above the  
162 middle joist and then directly above one of the off-centre (edge) joists.

163 The load was applied using a servo-hydraulic actuator, and specimen response was recorded. For the  
164 lower concentrated loads within serviceability limits, a 50 x 50mm steel plate on a rubber pad was  
165 positioned between the load cell and the surface of the concrete slab, as recommended in Eurocode 1,  
166 Section 6.3.1.2(5) [35]. For the load-to-failure test, this was increased to a 200mm x 200mm plate to  
167 inhibit local crushing or punching through of the concrete.

168 Due to limitations of the testing frame, the actuator could only be positioned directly above the centre-  
169 line of the specimen and, therefore, could not directly load the slab over the edge joists. Hence a transfer  
170 structure was designed to enable loading of the off-centre joists, see Figure 7. The actuator applied load  
171 to a transversely oriented I-section, which in turn transferred the load to the edge joist.



172

173 *Figure 7 - Transfer beam structure for off-centre loading*

174

### 175 2.2.1 *Instrumentation*

176 The mechanical behaviour of the specimen was monitored using a comprehensive set of instrumentation  
177 shown in Figure 8. As stated earlier a key aim of the research was to determine moment sharing across  
178 the specimen, and this was achieved by experimentally deriving a through-depth strain distribution, and  
179 subsequently converting those strains through the section to stresses, forces, and then moments. In order  
180 to be able to determine these strain distributions, a specific layout of strain gauges was required. The  
181 underpinning principles of this layout are as follows:

- 182 • Three gauges placed at distinct levels through the depth of timber joist allowed the calculation  
183 of curvature in the joist. In practice, as seen in Figure 4, five gauges were used in order to allow  
184 for any asymmetry in the joists, by averaging out the two values at each of the upper two gauged  
185 levels within the joist section, to get strain values for the centreline of the joist.
- 186 • Bi-axial gauges placed on top of the slab, directly above the corresponding gauges on the joists,  
187 enabled determination of the longitudinal and traverse strains there, and also investigation of  
188 shear lag effects at the top of the slab. The steel rebars of the A193 mesh were of too small  
189 diameter and were too smooth (giving rebar-concrete slip) to allow strain recordings that were  
190 meaningful in the context of the slab, while the formwork interlayer prevented reliable

191 placement of gauges at the soffit of the cured slab. Also note that gauges placed through the  
192 depth of the slab's exposed vertical edges would have led to strain readings not applicable  
193 across the width of the slab.

194 • The above limitations meant that it was not possible to place gauges at multiple levels through  
195 the slab. Consequently, an assumption of full slab-to-joist curvature compatibility was used  
196 along with the single longitudinal strain at the top of the slab, to enable determination of the  
197 entire through-depth longitudinal strain distribution in the slab. Once the strains had been  
198 converted to stresses and ultimately moments, the validity of this curvature compatibility  
199 assumption was checked by comparing the sum of the moments in each T-section with those  
200 deduced from equilibrium requirements (see Figure 10 and Figure 13).

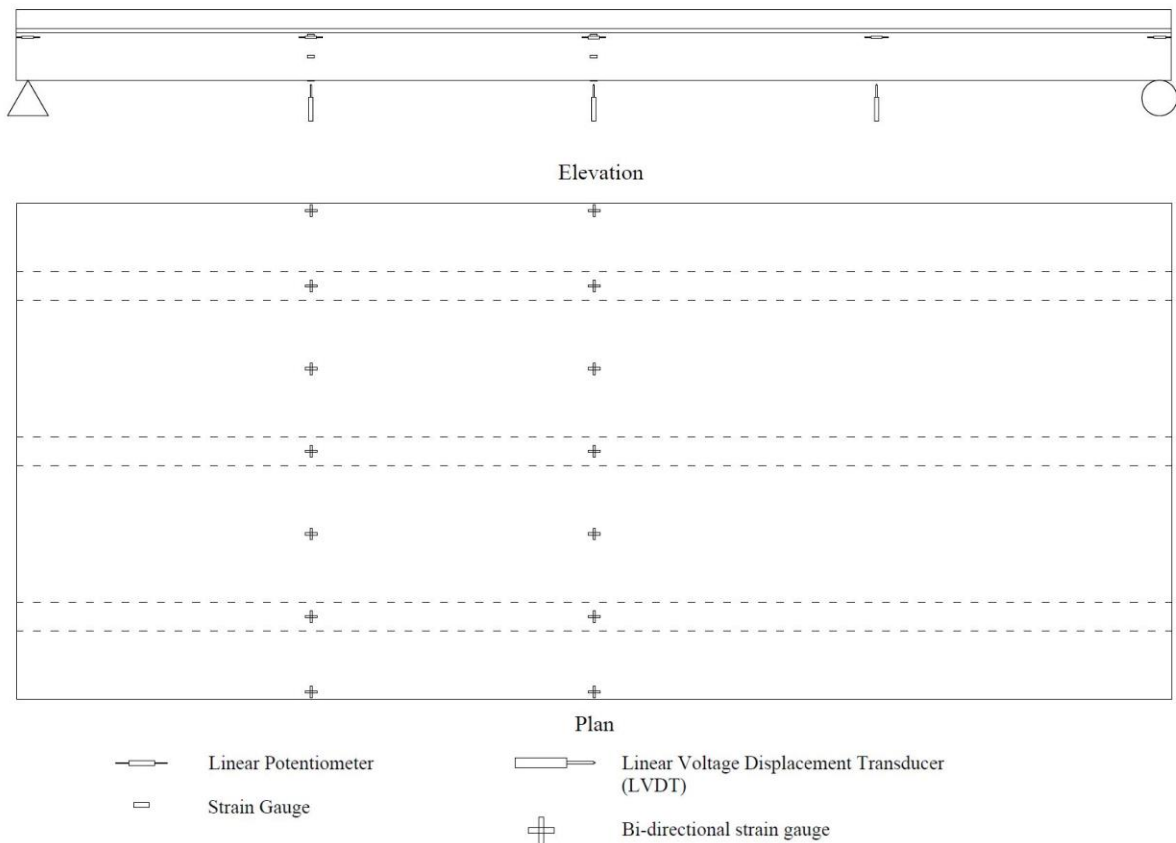
201 • The double shear connection test results were used to determine where through the depth of the  
202 interlayer that the slip strain should be applied.

203 All strain gauges were of the electrical resistance type. In order to achieve the above objectives, these  
204 gauges were strategically placed at six distinct locations, namely at mid-span and at one quarter-span  
205 location of each joist. At each location the timber joist was instrumented with five gauges as described  
206 previously, and directly above a 90° bi-axial gauge was placed on the concrete slab's top surface (Figure  
207 4). Four more longitudinally oriented gauges were placed on the top surface of the slab at mid- and  
208 quarter-spans, halfway between joists and also at the edges of the slab, in order to better determine the  
209 transverse distribution of longitudinal strains (Figure 8).

210 In addition to this ad-hoc and important strain gauge layout, a load cell was positioned at each support  
211 of each joist, six in total, to evaluate the distribution of support reactions across the width of the TCC  
212 specimen. An additional load cell was placed beneath the forcing actuator to enable conduct of  
213 equilibrium verification checks. Linear variable displacement transducers (LVDTs) were used to  
214 measure displacement at quarter- and mid-span locations along the specimen under each joist, and slip  
215 measurements were taken using linear potentiometers placed between the timber joists and timber  
216 interlayer. Five slip gauges were used per joist, spaced equally along the length, at both ends, mid-span  
217 and at quarter-spans. Whilst these types of instrumentation have been used in previous research on

218 multi-joist TCCs, the focus in these prior studies has been on behaviour in the linear regime, whereas  
 219 the research reported on in the present paper uses the instrumentation to capture data well into the non-  
 220 linear regime up to failure.

221



222 *Figure 8 - Instrumentation used on the specimen (not to scale). Instrumentation on the Elevation is per-joist.*

223

### 224 3 Results and Discussion

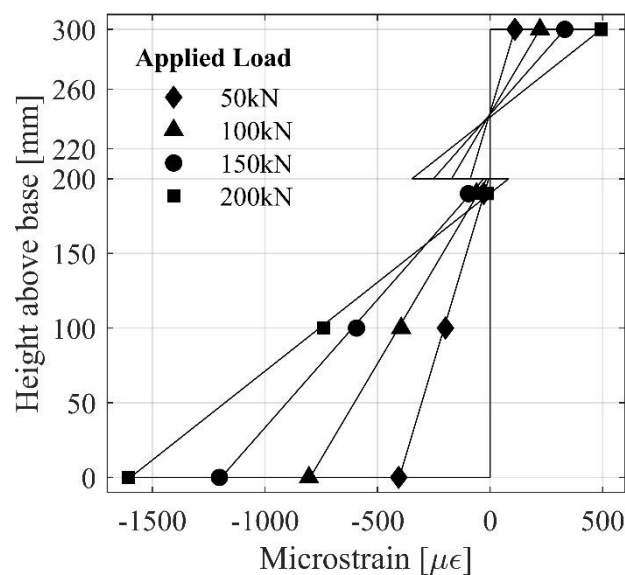
225 The following section describes the following:

- 226 • The disparity between moment sharing and reaction sharing profiles across the specimen at  
 227 service loads and up to failure.
- 228 • The behaviour of the specimens during the wet concrete loading stage
- 229 • Global structural behaviour at service loading and up to failure
- 230 • Structural characterisation of the shear connection and further analysis of its behaviour

231 **3.1 Load sharing profiles comparison**

232 Throughout the remainder of this paper joists J1, J2 and J3 refer to an edge joist, the middle joist and  
233 the other edge joist respectively. Likewise M2M, R1, M3Q, etc refer to the moment at midspan for the  
234 T-section including joist J2, to the support reaction for the T-section including joist J1, to the moment  
235 at quarter-span for the T-section including joist J3, etc.

236 To calculate the bending moments in the TCC specimen, the strain gauges that were placed strategically  
237 through the depth of the specimen were utilised as part of a Multi-Layer Analysis (MLA) tool developed  
238 for this purpose. Figure 9 shows the through-depth strain distributions recorded at mid-span of the TCC  
239 T-section including joist J3 for different applied loads. Compressive strains are positive. The markers  
240 represent the strains as recorded via the gauges, and the through-depth distributions are obtained as  
241 described in Section 2.2.1. The location of slip is assumed to be at the interlayer-joist interface (see  
242 Section 3.5.1), and therefore the strain profile is continuous throughout the slab and interlayer, again  
243 assuming curvature compatibility between the two components.



244

245 *Figure 9 - Through-depth strain plot for TCC section based on joist J3 at mid-span*

246

247 In the MLA approach the cross-section of the specimen was divided into 1mm thick layers. A sensitivity  
248 study showed little change in section force for layers of lower thickness. The through-depth strain  
249 distribution was then used to apply a strain value to each layer. These layer strains were converted to  
250 layer stresses, then layer forces, the values of which and the moments about a fixed point of which were  
251 algebraically summed to give the gauged TCC section axial forces and moments respectively. The  
252 bending moments were converted into a percentage share per T-section, and then plotted on the same  
253 axes as the support reaction share per T-section for comparison. This process was carried out for every  
254 load increment during each test, so that moments could be plotted against applied load. The plots also  
255 include an "expected value" which was derived from equilibrium requirements.

256 Three cases were assessed when running these MLA analyses, as follows:

- 257 • Case 1: No structural contribution from the timber interlayer, nor from any concrete in tension  
258 (concrete cracked for MLA layers in tension).
- 259 • Case 2: No structural contribution from the timber interlayer, nor from the concrete only  
260 when exceeding the experimental tensile capacity of 3MPa (concrete cracked for MLA layers  
261 in tension exceeding 3MPa).
- 262 • Case 3: Full structural contribution from the timber interlayer, and from the concrete up to  
263 and including the experimental tensile capacity of 3MPa (concrete cracked for MLA layers in  
264 tension exceeding 3MPa)).

265 The first case is a commonly used assumption for finite element modelling, and the third an optimistic  
266 assumption of obtaining the maximum structural contribution from the constituent materials, with the  
267 second providing an intermediate value.

268 The presence of the steel reinforcing mesh was not taken into account in the model for the following  
269 reasons:

- 270 • As seen in Figure 9, and verified for all other analysis cases, the strain at the mid-depth of the  
271 slab is in compression, and therefore the additional benefit provided by the reinforcement is  
272 minimal in compression concrete.

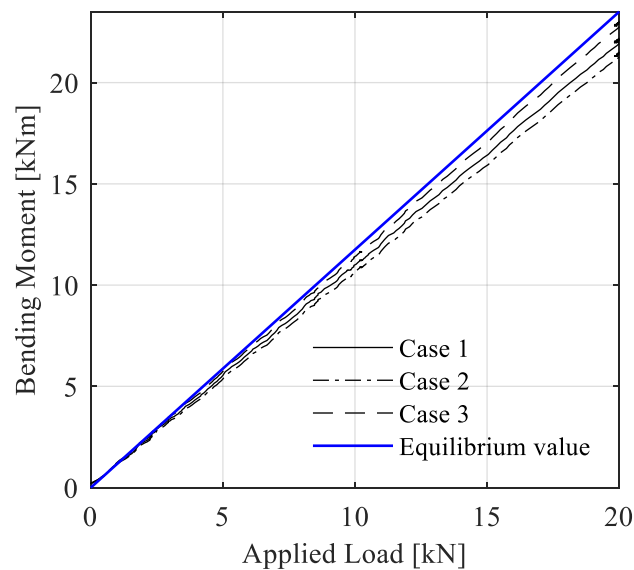


- The position of the neutral axis remains almost constant during testing, and therefore it is not expected that steel reinforcement at the mid-depth of the slab could be expected to be within cracked concrete before failure.
- The area reinforcement is less than 200mm<sup>2</sup> per metre width of slab and so its contribution in any case is minimal.

An assessment of potential tension stiffening of the concrete slab due to the tensile capacity of the underlying timber interlayer was also conducted. The value of  $\alpha_1$  was set as 0.5, and values of  $\alpha_2$  between 5 and 25 chosen – noted as appropriate by Kaklauskas and Ghaboussi [36]. The analysis showed that the effect of tension stiffening provided no determinable benefit in the results compared to those that are presented in this paper.

### 3.1.1 Results at service load

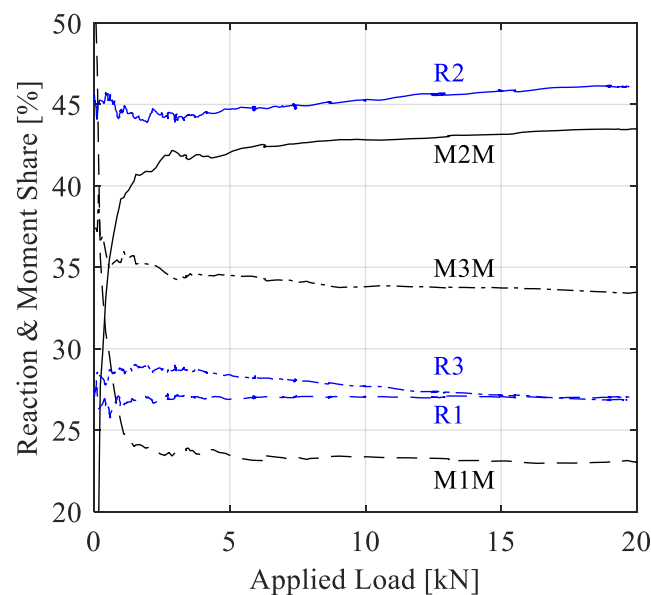
Figure 10 shows bending moment against applied load for the load applied directly on joist J3 at mid-span. The results show a linear trend which follows the expected value closely, deviating at 20kN by 7.9% for case 1, 5% for case 2, and 1.4% for case 3. Therefore, for the results presented for service loads, case 3 will be used in subsequent analysis.



288

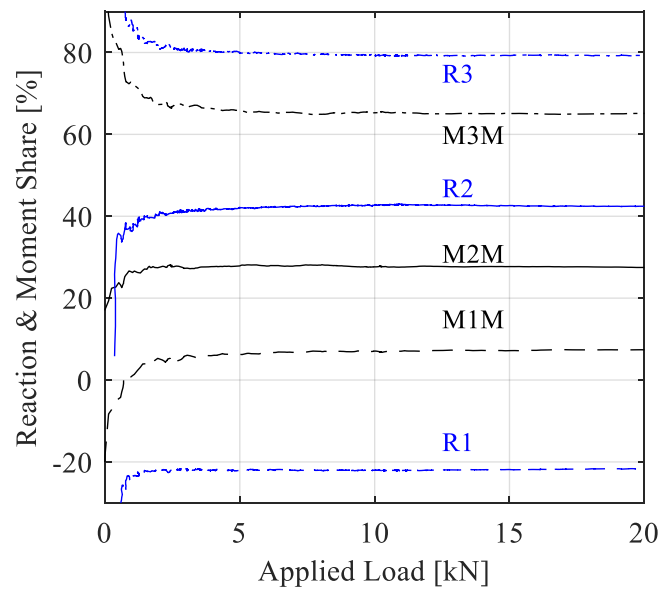
289 *Figure 10 - Bending moment vs applied load at mid-span for joist J3 loaded up to 20kN*

290 Figure 11 and Figure 12 compare the support reaction and midspan moment shares for the T-beam  
 291 corresponding to each joist, as calculated using the MLA. In Figure 11, the support reaction and moment  
 292 share for the crucial T-beam of the directly loaded joist J2 are similar, to within 4% of each other  
 293 between 3 and 20kN, indicating that use of the two values interchangeably would be appropriate for  
 294 design of this joist under this specific external load. However Figure 12 shows that when one of the  
 295 outer joists is loaded there is a distinct difference between support reaction sharing and moment sharing.  
 296 Indeed it is seen that in this situation, the bending moment share for the T-beam of joist J3 is about 15%  
 297 less than that of the support reaction. Otherwise note that the results show a distinctly linear response  
 298 to load (after a small period of the structure settling). This suggests that at service loads the structure  
 299 experiences little nonlinearity in either support reaction or moment distributions, an important  
 300 consideration for SLS design.



301

302 *Figure 11 - Comparison of support reaction and mid-span moment share for joist J2 loaded at 20kN. R(X) signals summed*  
 303 *support Reaction of joist X, M(X)M signals bending Moment share of joist X at Mid-span.*



304

305 *Figure 12 - Comparison of support reaction and mid-span moment share for joist J3 loaded at 20kN. R(X) signals summed*  
 306 *support Reaction of joist X, M(X)M signals bending Moment share of joist X at Mid-span.*

307

### 308 3.1.2 Failure load test data reliability checks and moment sharing assessment

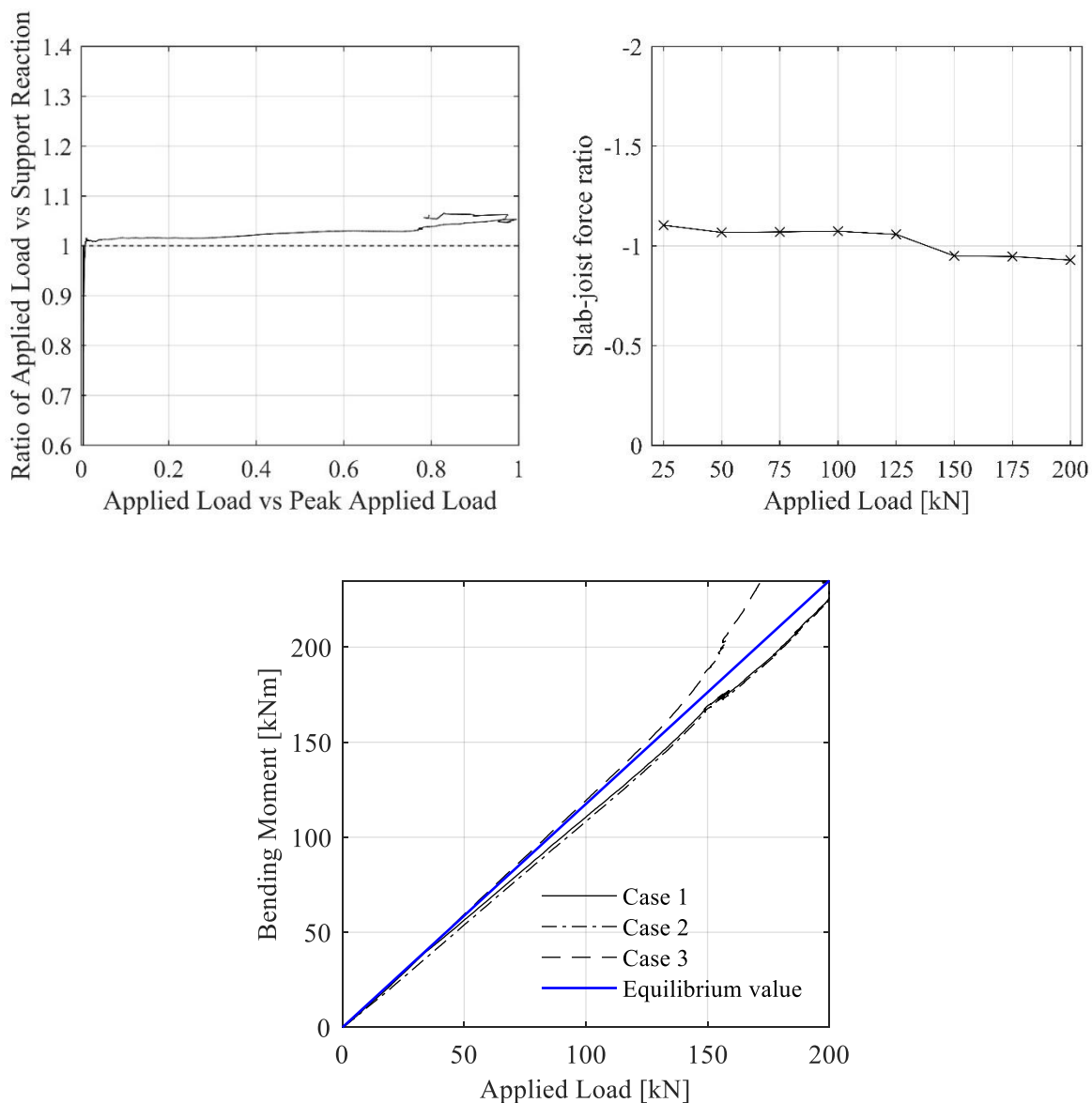
309

310 Figure 13 shows the three sets of equilibrium checks made on the specimen based on the failure test  
 311 data. Key comments are as follows, namely:

- 312 • Figure 13(a) shows that the support reactions summed up almost to the applied load throughout  
 313 the test, giving confidence in the load cell readings which underpin this plot.
- 314 • Figure 13(b) shows that, up to quite high applied loading, the slab section axial force at midspan  
 315 very nearly equates to the summed axial forces from the three joists at the same gauged section.  
 316 All of these axial forces have been obtained via the MLA approach and so are based directly  
 317 on the recorded midspan strains. This relationship between the slab and joist axial forces is  
 318 almost fully consistent with the longitudinal equilibrium requirement, an observation which  
 319 vouches for the high quality of the recorded strain data.
- 320 • In Figure 13(c) the bending moments obtained via the MLA method, using each of the three  
 321 cases set out in section 3.1, and also the moment required by equilibrium, are all plotted against

322 applied load. The results show that all three MLA cases follow the equilibrium curve closely  
 323 up to well beyond the onset of non-linearity. Indeed for cases 1 and 2 the comparisons against  
 324 equilibrium are very good up to the 200kN peak load achieved in the tests.

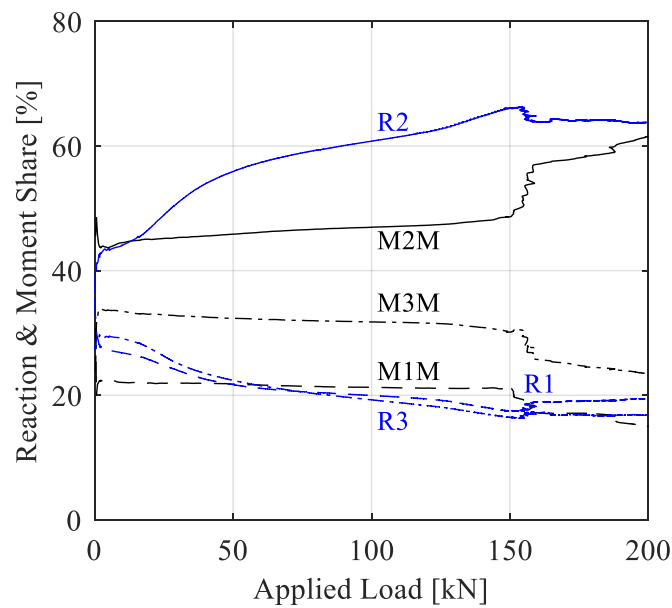
325 These above three sets of comparisons gave quite high confidence in the recorded load and strain  
 326 test data for assessing support reaction sharing and moment sharing. These are now discussed.



327  
 328 *Figure 13 – Equilibrium verification for Failure Test Data: (a) Reactions Relative to Applied Load, (b) Slab section axial*  
 329 *force relative to section axial force from joists combined, (c) Midspan moments*

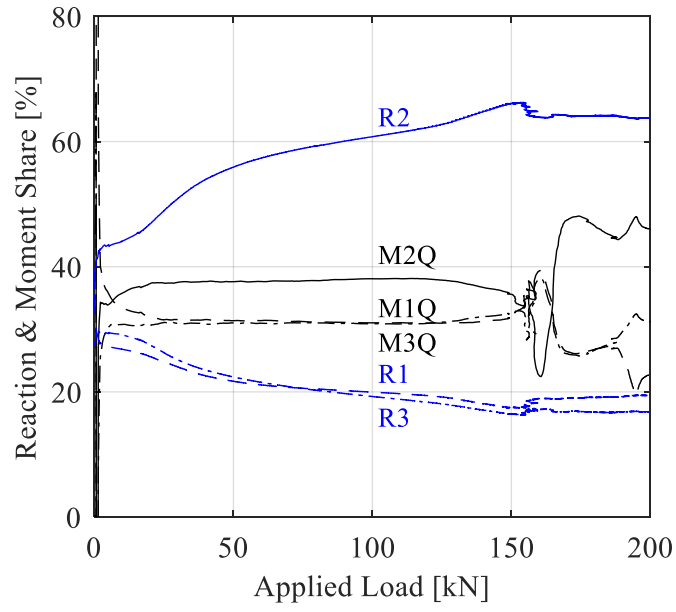
330

331 Figure 14 and Figure 15 show the relationship between support reaction share and bending moment  
 332 share at mid- and quarter- spans respectively. It is seen that the share of the maximum support reaction,  
 333 R2, varied considerably throughout the test, indeed increasing by a peak of 23% during the test. At  
 334 midspan (Figure 14), the maximum moment share M2M shows three distinct regimes, namely nonlinear  
 335 between 0kN and 40kN (which contrasts with the largely constant moment sharing seen during the  
 336 earlier 20kN test maybe due to changes in the specimen from repeated load tests), followed by constant  
 337 moment share between 40kN and 150kN, and finally rapidly changing moment share M2 in the  
 338 approach to failure. It is seen that the support reaction share exceeded both the mid- and quarter-span  
 339 moment shares. For the largest (i.e. J2) midspan moment the difference between the two sets of sharing  
 340 profiles peaked at about 18% at midspan (49% moment vs 67% reaction at 150 kN applied load) and a  
 341 highly significant 29% at quarter-span. This shows that design of a TCC floor based on a moment  
 342 distribution that mimics the support reaction distribution could lead to quite a conservative design.



343

344 *Figure 14 - Comparison of support reaction and mid-span moment share for joist J2 loaded at 200kN. R(X) signals summed*  
 345 *support Reaction of joist X, M(X)M signals bending Moment share of joist X at Mid-span.*



346

347 *Figure 15 - Comparison of support reaction and quarter-span moment share for joist J2 loaded at 200kN. R(X) signals*  
 348 *summed support Reaction of joist X, M(X)Q signals bending Moment share of joist X at Quarter-span.*

349

350

351

### 3.2 Wet concrete stage moment sharing

Mid-span deflection and soffit strain variations with time, as recorded from each joist during the concrete cast, are presented in Figure 16. The steps on both sets of plots are due to repetition of pouring then spreading of concrete onto the formwork interlayer. Each plateau corresponds to the pause between the previous batch of concrete having just been spread and the arrival of the new batch. It is seen that the strains at the conclusion of casting reached up to  $210 \mu\epsilon$ , broadly similar to the strains experienced under the low-level concentrated load test discussed in Section 3.3, and approximately 4% of the strains achieved at peak load during the failure test on the specimen discussed in Section 3.3. This shows that in comparison to service level loading, the strains induced by the casting and subsequent curing of the concrete are significant and should be considered, however they become of less importance when ULS design is considered.

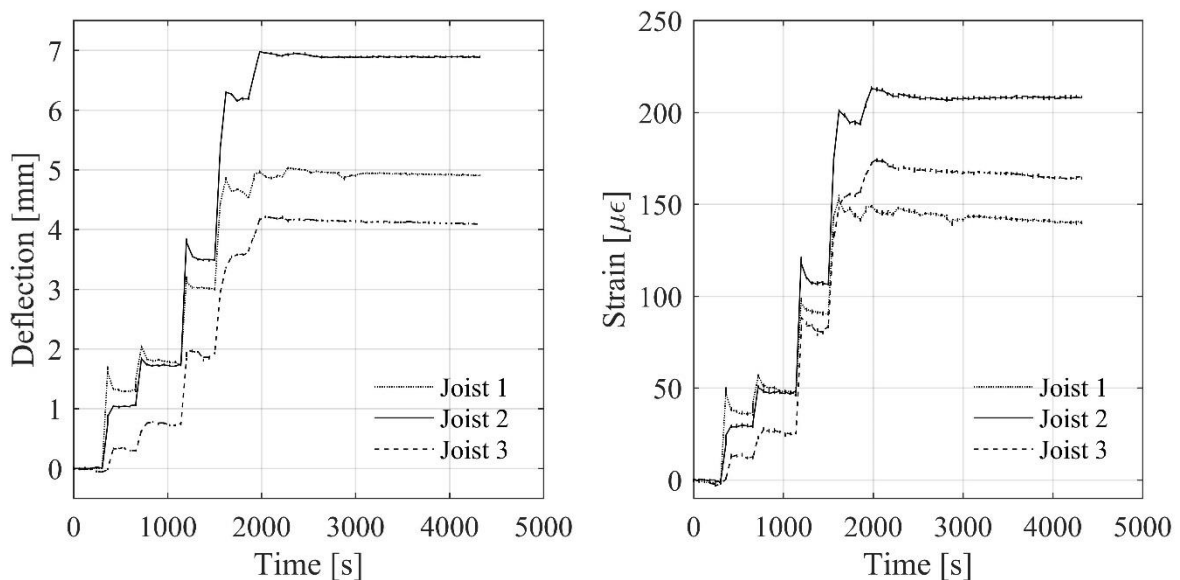


Figure 16 - (a) Deflection of joists mid-span, (b) joist soffit mid-span strains, during casting of the concrete slab

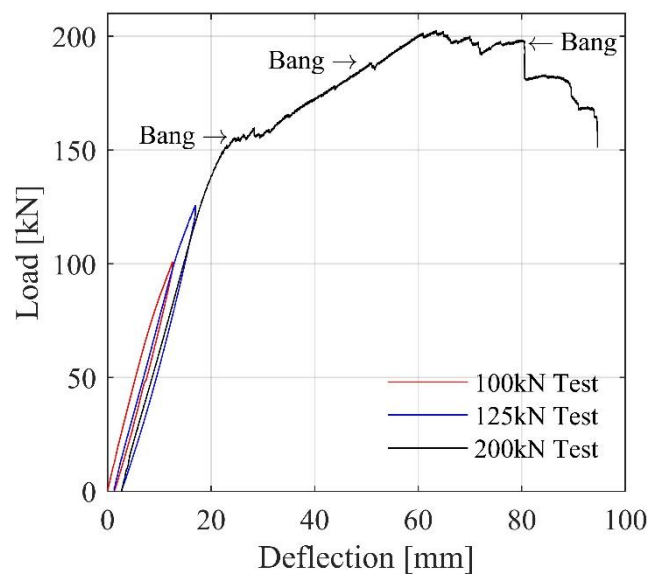
Let us now assume that the timber structure at this wet concrete stage acted as three adjacent T-sections in longitudinal bending, where each T-section comprised one joist in full composite action with the overhead interlayer of width halfway between the joist in question and its nearest neighbour(s). If the section flexural stiffness ( $EI$ ) value calculated for this section is combined with the midspan strains recorded at completion of casting, it is found that the outer and middle T-sections carried midspan moments due to wet concrete loading of 3.14 kNm, 3.7 kNm and 4.64 kNm respectively. This equates

369 to 27%, 32% and 41% moment sharing at midspan, not too dis-similar from the ratios seen at the  
 370 composite-stage when a midspan concentrated load was applied directly above the middle joist, see  
 371 Section 3.4. Note also that the sum of these three T-section moments equates to 11.4kNm, which closely  
 372 approximates the 12kNm value required for equilibrium. This vouches for the high quality of the  
 373 recorded strain data at the wet concrete stage.

### 374 3.3 Composite Stage – Other facets of global structural behaviour up to ultimate load

375 During the failure test the specimen was subjected to gradually increasing load first up to 100kN, then  
 376 up to 125kN, with a period of slow unloading after each, followed by a final loading to failure.

377



378 *Figure 17 - Load vs deflection of TCC specimen*

379 Figure 17 shows the corresponding load-deflection plots. It is seen that the specimen exhibited similar  
 380 responses to the repeated load cycles. Following each test there was some deflection (1.4mm and 1.3mm  
 381 respectively) that was not fully recovered, and therefore on each subsequent reloading cycle the  
 382 specimen moved along a slightly different trajectory. Figure 17 shows multiple and distinct stages of  
 383 behaviour, as follows:

- 384 • High-stiffness, linear load response up to just over 150kN;
- 385 • A drop to almost zero tangent stiffness over a short (approximately 8mm) range of deflections;



- 386 • A sharp rise in stiffness, akin to strain hardening, and of constant gradient up to peak load;
- 387 • A subsequent holding of the load at the roughly constant peak value, resembling ductile behaviour,
- 388 over a 20mm (60 – 80mm) range of deflections;
- 389 • Load drop to failure by (Figure 18) splitting of the slab at the support connectors, midspan flexural
- 390 fracture of J2, plus J2-aligned and oval cracks (traced in Figure 18(c)) in the slab.

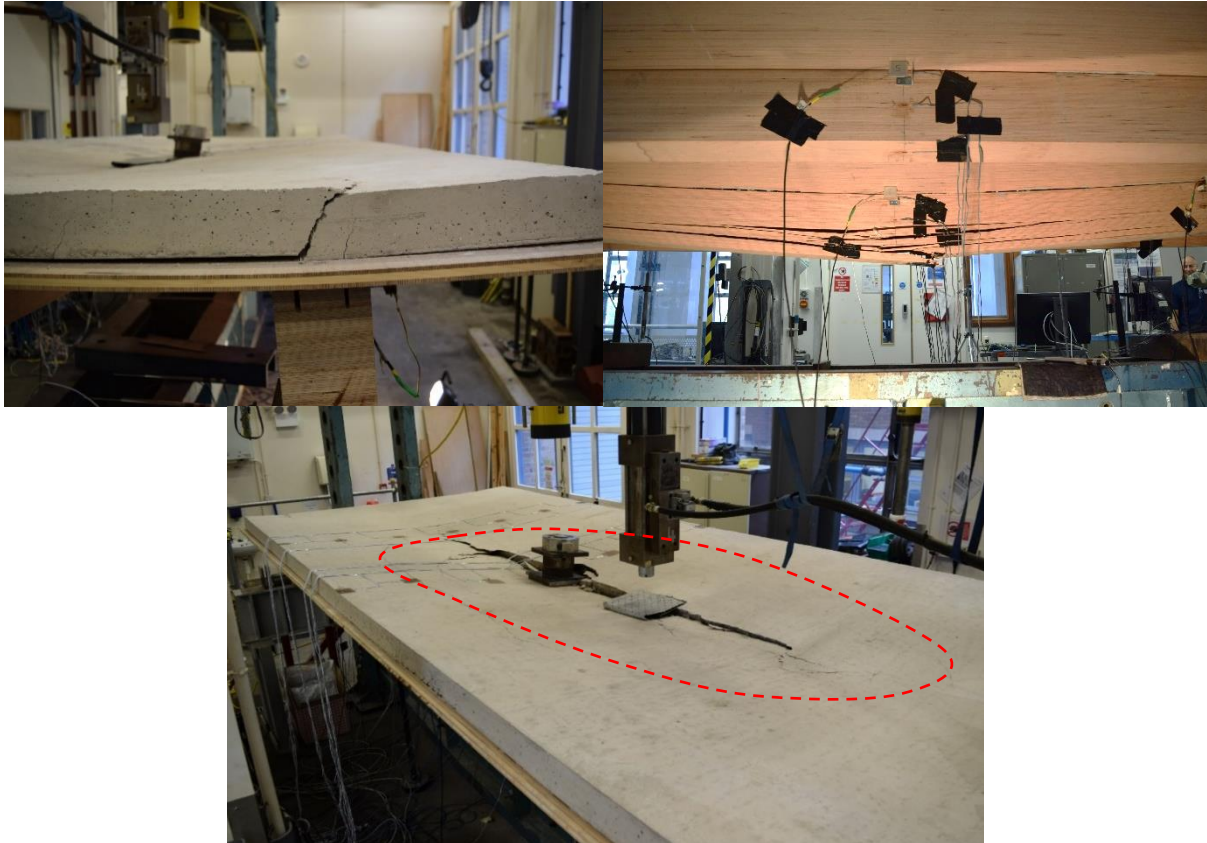


Figure 18 - TCC Specimen Failure Modes – (a) Slab end split, (b) Fracture of J2, (c) Oval cracks and major split

391 The slab itself deformed in a bowl-like manner (which might have created membrane effects that in  
392 turn induced the oval crack of Figure 18(c)), with significant deflection at its centre under the  
393 concentrated loading. This contrasts with the lower loads, where the bowl effect was not as  
394 pronounced, as shown in Figure 19.

395 During the short regime of sharp drop in stiffness (Figure 17) which started just above 150kN, oil had  
396 to be supplied at an increased rate to the piston of the loading jack, to continue increasing specimen  
397 deflections. It is seen that many of the kinks in the plot of Figure 17 were accompanied during the test  
398 by loud bangs, which signalled the above and other possible local failures in the timber interlayer and  
399 concrete slab.

400 Figure 20 shows the load redistribution which occurred, as the test progressed, from the outer joists (J1,  
401 J3) to the central joist (J2), with the total reaction share of the two outer joists reducing monotonically  
402 from over 50% at low loads, to 34% at the point of nonlinearity, indicating the decreasing ability of this  
403 TCC structure to distribute load transversely between its adjoining T-sections, and highlighting the  
404 nonlinearity of the support reaction sharing with increasing load.

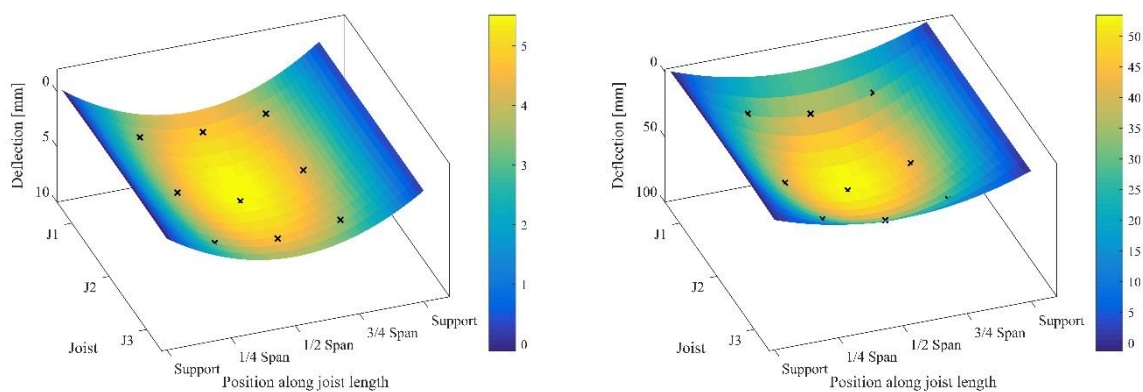
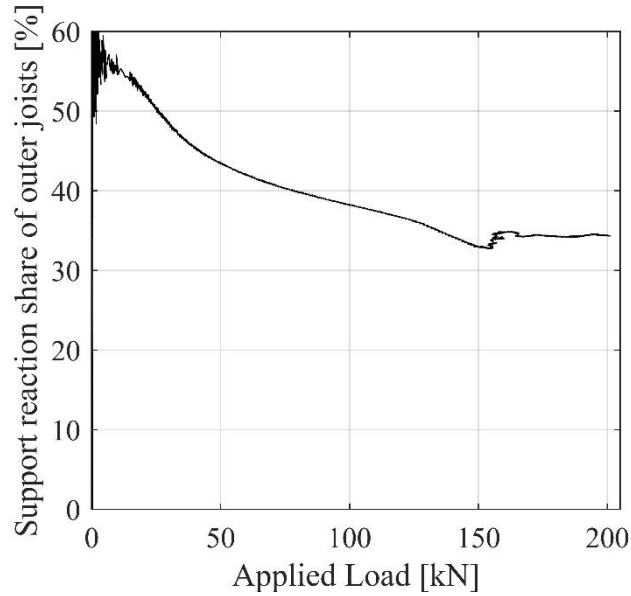


Figure 19 - Interpolated deflection profiles of the specimen at (a) 50kN and (b) 200kN. X's mark locations of measurement.



405

406

*Figure 20 - Load redistribution from outer joists to central joist*

407

Figure 21(a) shows the increasing support reaction with load for the six supports. The trend is generally

408

linear, although there is some notable nonlinearity at low and high loads. A small degree of specimen

409

asymmetry can be seen here as shown by the divergence in support reaction between near- and far-

410

supports of the same joist, especially for joist J2. Nevertheless, the general trend is as expected with the

411

central joist J2 taking an increasingly larger proportion of the total load. At 155kN there is a noticeable

412

discontinuity, which correlates with Figure 17. The support reactions after this point again increase

413

linearly but at a different gradient, as suggested by the corresponding deflections. This occurrence is

414 contemporaneous with the observation of vertical splitting at the connections in the end zones of the  
 415 slab as shown in Figure 18(a) and discussed above.

416 The deflection profiles of all joists are shown in Figure 21(b). The clear dominance of the J2 profile  
 417 over the J1, J3 profiles further reinforces the point that a quite high fraction of the applied load was  
 418 borne by the J2 T-section in relation to the outer T's. At 200kN the load and deflection sharing profiles

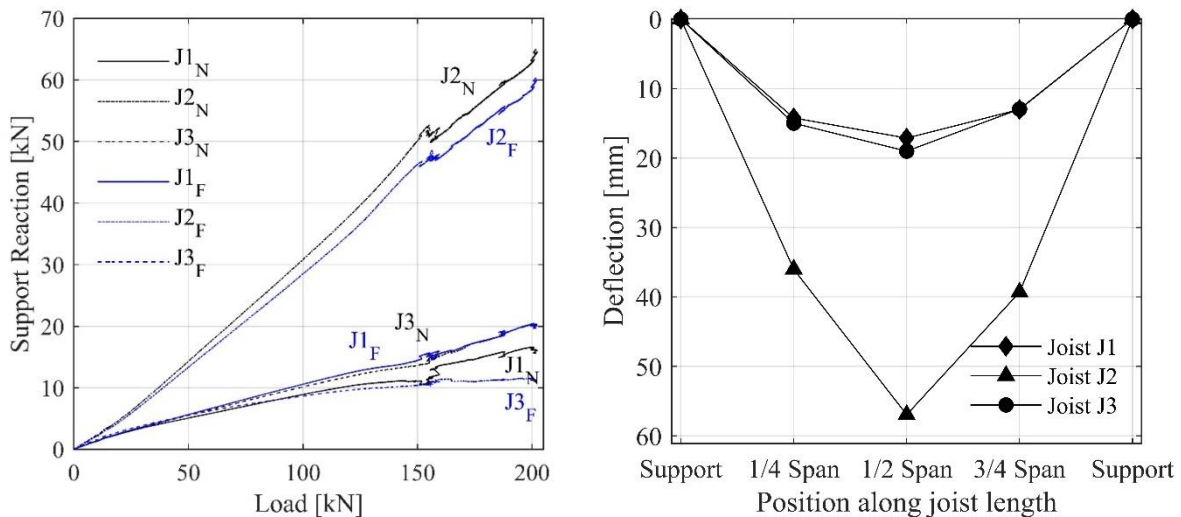


Figure 21 - (a) Support reaction profiles vs total applied load, (b) Deflection profiles at 200kN

419 are quite similar to each other, with the central joist taking 64% of the support reactions and 61% of the  
 420 deflections, with similar shares of around 18% each for the outer joists for both support reaction and  
 421 deflection.

### 422 3.3.1 Post-test visual observation of connections

423 Cutting of the specimen into pieces to facilitate disposal enabled confirmation that the oval and  
 424 longitudinal cracks shown in Figure 18(c) had propagated through the full-depth of the concrete slab.  
 425 Exposing the connectors by excavating out the concrete slab showed that these connectors remained  
 426 intact post-failure of the overall TCC specimen.

427 As Figure 22(a) shows, the midspan zone connections did not experience pull-out failure from the joists  
 428 as may have been initially thought due to some of the loud bangs heard during testing. Of course,  
 429 connector-to-slab failure cannot be ruled out. The connectors at the extremities of joist J2 were also still  
 430 firmly embedded within the timber after the test (Figure 22(b)) and showed no signs of deterioration at  
 431 the concrete-interlayer interface. The same is true for the connectors along J1 and J3, which also showed

432 no signs of deformation above or below the timber interlayer. Hence the splits in the slab above the  
433 joist, observed in the support zones as shown in Figure 18(a), might not have led to plastic deformation  
434 of the raised steel mesh connectors.

### 435 **3.4 Structural action under other load layouts in the elastic regime**

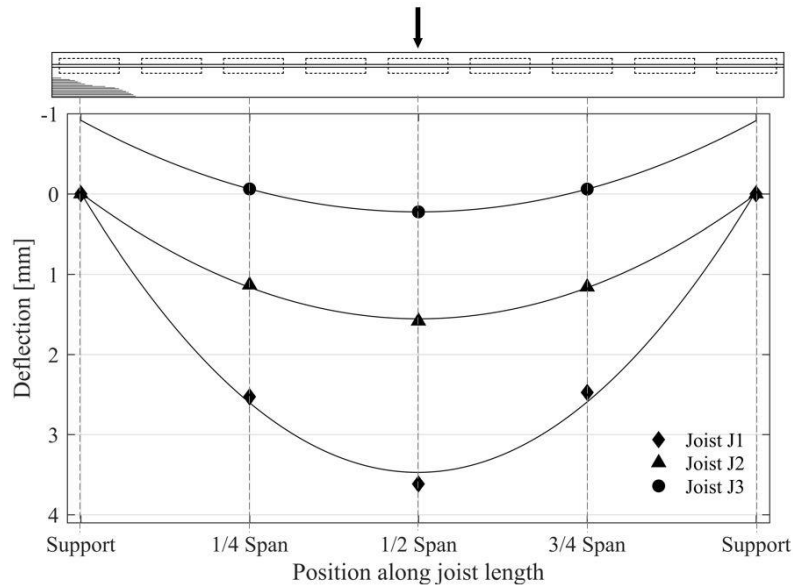
436 Recall that before the specimen was loaded to failure, it was also tested within the linear elastic loading



Figure 22 - Post-test observation of (a) central connector on joist J2, and (b) end connector on joist J2.

437 regime, by applying a concentrated pad load of up to 20kN to the slab directly above the centreline of  
438 each joist at mid-span. The transverse load sharing properties of this structure can be seen when  
439 comparing the deflections of the edge- and centrally-loaded joists, see Figure 23. Maximum deflection  
440 (at mid-span) of the edge-loaded tests reached 3.6mm for J1 and 3.5mm for J3, reducing to 0.2mm at  
441 each opposite edge-joist. The central joist deflected only 1.9mm when placed under the same 20kN  
442 load, decreasing to 1.5 and 1.6mm in the adjacent joists. The distribution capabilities of the specimen  
443 are clearly seen here, although a non-symmetry is highlighted by the difference in resultant deflections  
444 of the edge joists. This is due to inherent imperfections which arise in constructing such large specimens  
445 and needs to be considered throughout the design process.

446 Of further interest is the deflection at supports. Whilst no instrumentation was placed at the supports,  
447 an extrapolation of the recorded data shows that, for the edge-loaded tests, the joist furthest from the  
448 load exhibited negative deflection, i.e. uplift, of up to 1mm. This is shown graphically in Figure 23 for  
449 the test loading joist J1 at 20kN.



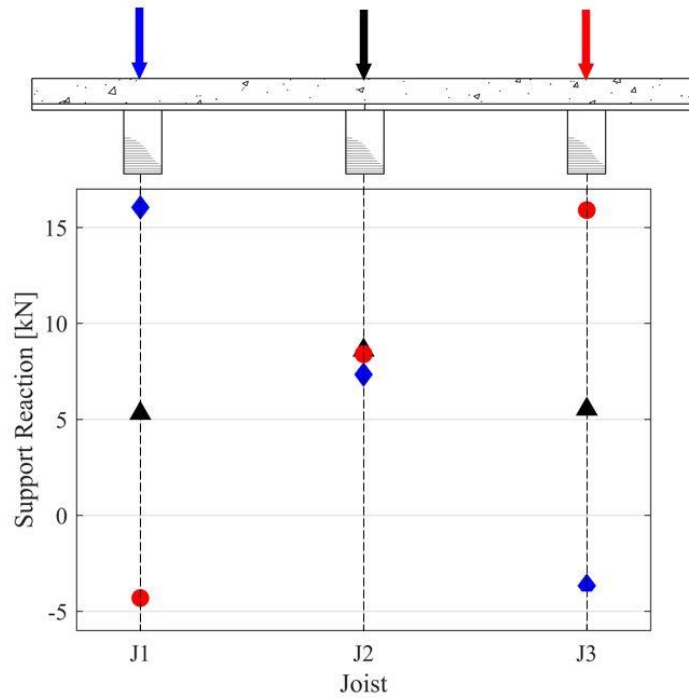
450

451 *Figure 23 - Interpolated deflection profiles of each joist for the 20kN test loaded at mid-span on joist J1*

452

453 The support reaction distributions for these elastic regime load tests again suggest good transverse  
 454 sharing capabilities. When the central joist was loaded, the central supports took 44% of the load.  
 455 However with only one edge joist loaded the corresponding edge supports carried 80% of the load, with  
 456 40% taken by the central supports and -20% taken by the far edge supports, representing an uplift force  
 457 on that far edge and reaffirming the extrapolation of deflection data to assume an uplift of the slab.  
 458 Small uplifts were able to be recorded, by zeroing the load cells with the specimen resting on them:  
 459 therefore, any reduction in load (i.e. uplift) up to the level of the unloaded specimen was able to be  
 460 noted. This uplift at low loads emphasises the excellent transverse stiffness of the slab, which allowed  
 461 it to distribute load when joist J2 was loaded, and in this situation, to have supported the uplift of the  
 462 slab. There is good consistency between support reaction totals for each end of the specimen, with the  
 463 far end taking 48% and 44% for the central- and edge- loaded tests respectively.

464 Figure 24 shows the summed support reactions for each of the three service-level load cases. These  
 465 profiles more closely approximate a symmetrical distribution than do their deflection counterparts. Note  
 466 that these support reaction sharing profiles do not mimic the highly nonlinear bending moment sharing  
 467 profiles as presented earlier in this paper. It is thus worth reiterating that provision of these experimental  
 468 moment sharing profiles is a novel feature of this study.



469

470

471

Figure 24 - Summed support reaction profiles for each of the 20kN low-load tests

472

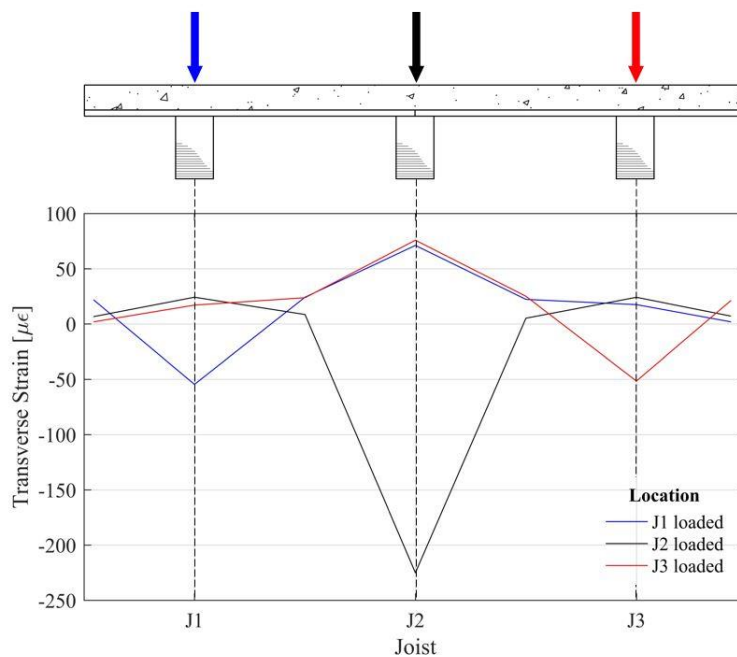
Profiles of transverse strain (tensile positive) for the top of the slab at midspan are given in Figure 25

473

for all three loads. The lower quarter-span readings are omitted. It is seen that the largest compressive

474

strain (236  $\mu\epsilon$ , due to transverse negative bending) occurred when load was above J2.



475

476

Figure 25 - Concrete transverse strain profiles at mid-span for the three low-level 20kN load cases.

477 Loading the edge joists led to lower peak transverse compressive strain (above the loaded joist) than  
 478 did loading the central joist, but it did lead to higher peak transverse tensile strain (above the adjacent  
 479 joist) than did loading J2. At quarter-span no transverse tensile strain was recorded in either edge-loaded  
 480 case. Comparing the longitudinal and transverse strains for loading above joist J1, the longitudinal  
 481 strains were lower in absolute magnitude at quarter-span, and higher at mid-span. In addition, at all four  
 482 locations away from the loaded joist, the recorded transverse strains indicated negative transverse  
 483 moments whereas the recorded longitudinal strains indicated positive longitudinal moments. Table 1  
 484 shows the ratios of longitudinal-to-transverse strains at these locations.

485 *Table 1 - Ratio of longitudinal to transverse strains for 20kN loading above joist J1*

Span location	Ratio of strains		
	J1	J2	J3
Quarter-span	0	-0.7	-0.8
Mid-Span	0.4	-1.6	-1.2

486

487 These results suggest that in the linear elastic regime the concrete slab provided good transverse  
 488 stiffness to enable effective load sharing between adjacent joists. This load sharing was most  
 489 pronounced in situations where the loaded joist could distribute load in two directions. At these loads  
 490 the transverse strains in the concrete were not significant.

### 491 **3.5 Shear Connection Behaviour**

#### 492 *3.5.1 Double Shear test results*

493 Results of the double shear tests (as defined in Section 2.1) conducted on each of the four connection  
 494 specimens are shown in Figure 26. The figure shows average load-slip curves for each specimen, taking  
 495 all four displacement measurements into account.

496 The average slip stiffness is 505.4 kN/mm across the four experimental specimens, which is an  
 497 extremely high value. Individual slip stiffnesses ( $k_s = 0,4 F_{est}/v_{i,mod}$ ) and maximum loads ( $F_{max}$ ) are  
 498 shown in Table 2, calculated using equations defined in BS EN 26891 [37]. The bulges in the initial



499 linear portions of the curves – representing the unload-reload cycles as required by the standard – are  
 500 small, indicating a low impact of small numbers of load cycles within the early elastic regime.

501

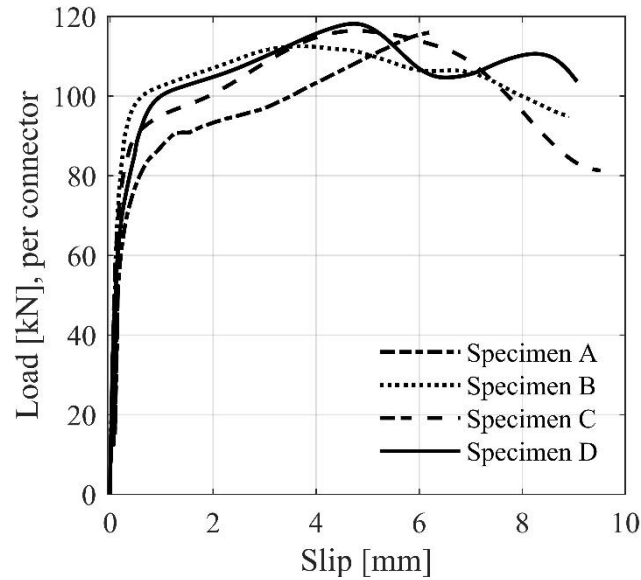


Figure 26 - Average load-slip curves for the four specimens

502

503

504

505 Table 2 - Load and stiffness properties of double-shear connection specimens

Specimen	Maximum load ( $F_{max}$ ) (kN)	Failure mode	Slip stiffness ( $k_s$ ) (kN/mm)
A	116.3	Shear rupture of mesh	406.7
B	112.6	Shear rupture of mesh	614.4
C	116.5	Shear rupture of mesh	581.4
D	118.4	Shear rupture of mesh	418.9
Average:	115.9		505.4
Coef. of Variation	0.021		0.213

506

507 Figure 26 suggests that the connectors also showed good ductility, characterised by a long plateau of  
 508 minimum approximately 8mm slip. The potentiometers used in the experiment recorded up to a limit

509 of 10mm slip. However, visual observation of markings placed on the joist and slab indicated that slip  
510 increased up to at least 15mm for each test before final ductile failure of the connection occurred.

511 For each specimen, there was plastic failure of the connector, at the interface between the *timber joist*  
512 *and the timber interlayer*. This highlighted the location where the slip-strain should be determined in  
513 through-depth strain plots in Section 3.1, and also where to mount linear potentiometers on the full-  
514 scale specimens so as to record slip between the joist and slab.



Figure 27 - Failure planes of the connectors in Specimen A at (a) the surface of the timber joist, and (b) the timber interlayer.

515 The specimens displayed significant shearing deformation of the connectors before failure. Figure 27(a)  
516 shows residual evidence of clear deformation of the connector, which remained well-embedded within  
517 the timber joist, whilst Figure 27(b) shows the corresponding remainder of the connector embedded  
518 into the concrete via the interlayer. The level of deformation of each connector before failure was high,  
519 evidencing the excellent ductility that was exhibited by Figure 26. Following the test, most slab and  
520 interlayer portions detached without force - the connector having completely sheared. In other cases,  
521 the majority of the connector had failed, but was still connected at discrete locations.

522 3.5.2 Connection behaviour in full-scale tests

523 During the elastic regime tests, slip between the slab and joist was negligible. This was deliberate, so  
 524 as to allow for multiple tests without affecting the structural properties of the specimen. However, at  
 525 higher loads in the failure test, the slips become much more pronounced, especially in joist J2 above  
 526 which the load was applied. Figure 28(a) shows the values of slip along each joist at 200kN, as measured  
 527 by the linear potentiometers. Figure 28(b) shows the slip profile for the central joist J2 with increasing  
 528 load up to 200kN.

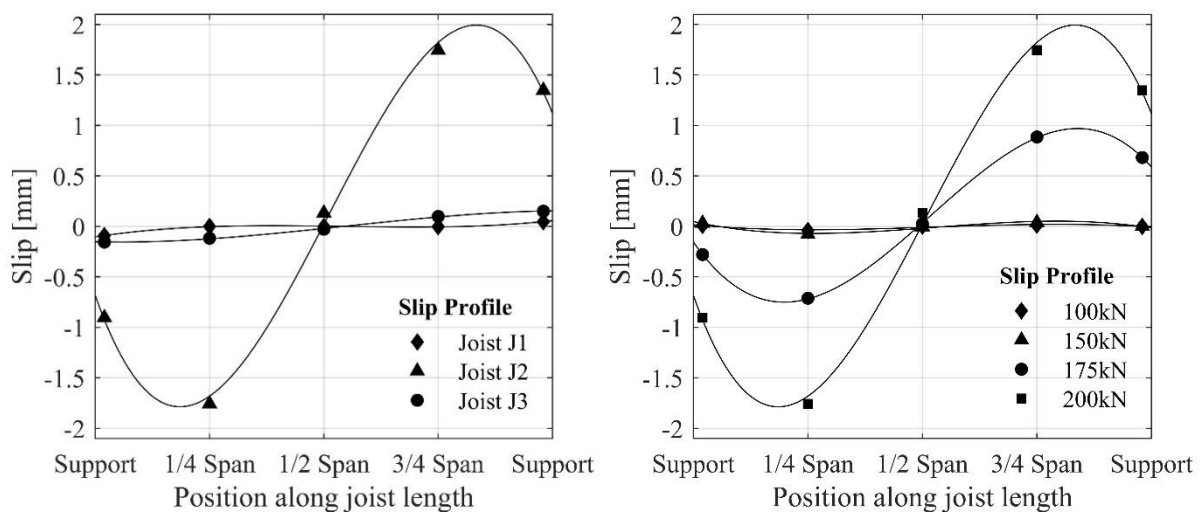


Figure 28 - (a) Slip gauge readings and interpolated slip profile at 200kN, and (b) interpolated slip profiles for J2 under increasing loads.

529 These slip profiles were used to determine the slip values at the centre of each of the nine shear  
 530 connectors along each joist. These slip values were then referenced back to the average connection  
 531 characteristic from Figure 26, to derive a longitudinal shear force value in each connector. It might be  
 532 expected that the slip, and corresponding force, were maximum at the extremes of the specimen, but as  
 533 seen in Figure 28 and this was clearly not the case. At applied loads exceeding 150 kN, this was probably  
 534 influenced by the splitting of the slab at the end-of-span connections as shown in Figure 18(a) and  
 535 described earlier. This splitting might well have compromised the abilities of these end connections to  
 536 transfer load.

537 In Figure 29(a) the full range of applied load vs connector slip is shown for the two most heavily loaded  
 538 connectors, 2 and 8, along the J2 T-section. On these plots the sign of the longitudinal slip is deliberately  
 539 kept the same (despite the reversal which occurred in practice) as midspan is crossed, so as to facilitate  
 540 comparison between these connection forces on both sides of midspan. Figure 29(b) shows, for each of  
 541 three different applied load levels, the corresponding forces that relate to the level of slip experienced  
 542 by each connector. It is unclear to what extent the slab cracks developed in the concrete significantly  
 543 above 150kN, and thus the plots are only shown up to 155kN, where there is confidence of a relationship  
 544 between slip and connector force.

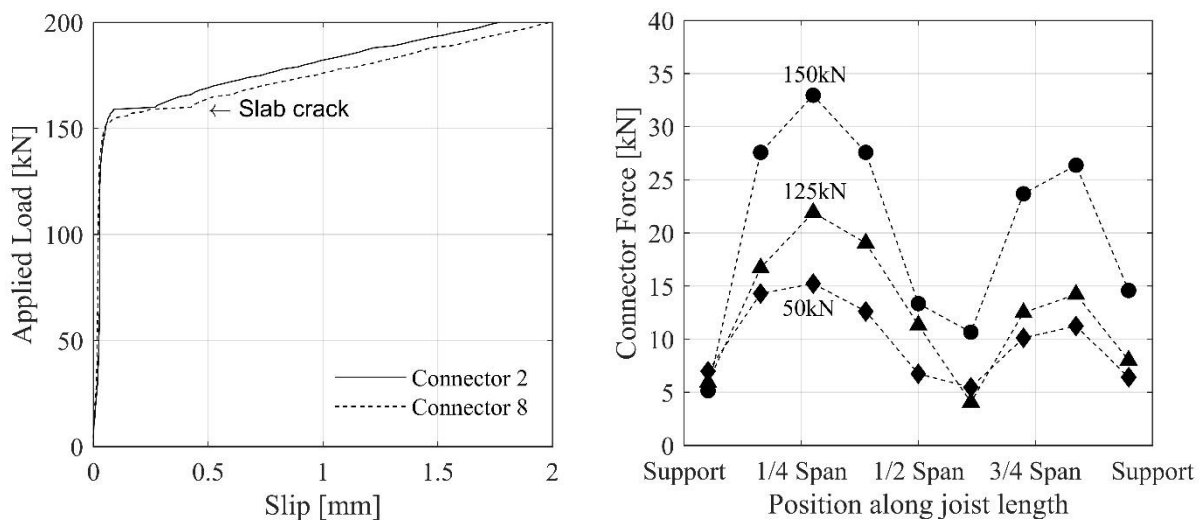


Figure 29 - (a) Applied load vs connector slip for connector 2 and connector 8, and (b) Connector forces in joist J2 for loads of 50kN, 125kN, and 150kN.

545 After reaching peak load on the TCC specimen, the connectors continued to deform. Records from the  
 546 linear potentiometers highlight that slip at the ends of the specimen between the concrete and timber  
 547 joist reached up to 7.8mm before the timber fracture occurred, highlighting the excellent ductility of the  
 548 specimen post-yield, agreeing well with the short ductility plateau following peak load in Figure 17.

549

### 550 3.6 Material Tests

551 Instrumented tensile tests on the timber used in the project led to an Elastic Modulus value of 18.3GPa,  
 552 higher than that of the manufacturer-quoted value of 16.7GPa. The results of the concrete cube tests are  
 553 presented in Table 3

554 *Table 3 - Concrete material properties*

Concrete Pour	Average Cube Strength (N/mm <sup>2</sup> )	Std. Dev.	Average Split Cylinder Strength (N/mm <sup>2</sup> )	Std. Dev.
Connector Specimens 1	52.4	5.6	4.2	0.29
Connector Specimens 2	53.3	3.9	4.2	0.57
Full-scale specimen	43.3	1.1	3.3	0.12

555

556 **4 Conclusions**

557 This paper presents experimental findings of a near full-scale multi-joist timber-concrete composite  
 558 floor specimen, fabricated from RC32/40 concrete and GL70 hardwood LVL joists, connected by  
 559 expanded steel mesh shear connectors.

560 The specimen was comprehensively instrumented to enable recording of support reactions,  
 561 displacements, timber-concrete slip and strains. The strains were used to calculate TCC section bending  
 562 moments through the use of a multi-layer-analysis (MLA) model. Before the failure test was conducted  
 563 on the specimen, elastic regime load tests under concentrated loads were applied at mid-span above  
 564 each joist to infer the transverse load sharing capabilities of the specimen. Following these elastic  
 565 regime tests, the specimen was loaded to failure under a concentrated load on the slab above the middle  
 566 joist, which led to failure at 202kN.

567 From the work presented in this paper, there are four key novelties that have emerged, as follows:

- 568 • The transverse distribution of support reactions and of ¼-span or midspan bending moments can  
 569 differ significantly from each other in multi-joist timber-concrete composite floors. For the TCC  
 570 specimen fabricated and tested to failure in this study, the highest share of support reaction was  
 571 found to have significantly exceeded that of the highest bending moment in the critical joist (67%  
 572 vs 49% at 150kN). If taken into consideration, this actual level of moment sharing can enable  
 573 increased design efficiency when compared to the alternative of basing moment sharing on support  
 574 reaction sharing. In tests, it is not sufficient purely to measure support reaction distribution in order  
 575 to understand the midspan moment distribution requirements for the design of TCC floors.

- 576 • Whilst within the elastic regime the support reaction and midspan moment shares were each largely  
577 constant (but different from each other) with increasing load, the support reaction shares showed  
578 increasing nonlinearity at higher loads, whilst the moment distributions remained relatively  
579 constant but gave way to sharp nonlinear changes in the close approach to failure.
- 580 • A highly effective means of checking the recorded strain data's reliability entails application of  
581 curvature compatibility to these data, followed by conversion of the resulting through-depth strain  
582 distributions into layered axial forces and then into layered moments both of which have been  
583 shown overall to satisfy global equilibrium requirements when algebraically summed. Very  
584 importantly, the overall TCC section moments obtained via summation of the layer contributions  
585 enabled test data-based quantification of transverse moment sharing.
- 586 • The 400 mm long expanded steel mesh plate is highly suited to the role of shear connector in  
587 timber-concrete composite floors. Its high slip stiffness and strength (average 505.4kN/mm and  
588 115.9kN respectively), combined with good ductility post-yield meant that it performed well under  
589 loading on the present test specimen up to failure.
- 590 • The timber-concrete composite flooring using this connection method, and in the layout presented  
591 in this paper, failed primarily due to fracture of the timber in the zone of peak moment directly  
592 under the applied load (for a simply-supported arrangement), following significant plastic  
593 deformation of the specific connections tested in this study.
- 594 • It is recommended that any future computational modelling of TCCs use a model of the complete  
595 structure rather than single T-joists sections, in order that the distribution of bending moments  
596 across the slab will be clear and automatically taken into account as part of TCC design.

597 Further work is required to characterise the behaviour of TCC floor specimens that have a less-dense  
598 shear connector arrangement, allowing the specimen to more fully take advantage of the ductility  
599 offered by the connections before failure. For future tests on multi-joist TCC floors, the through-depth  
600 strain gauge layout should be used in order to determine individual T-section moment values and  
601 hence transverse sharing, so as to grow the body of research which utilises this method. In the longer  
602 term, the development of an experimentally-underpinned database of transverse sharing of

603 longitudinal moments in multi-joint TCC floors under different realistic load forms could enable more  
604 structurally efficient design of such floors in future practice.

605

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609

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