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PREFABRICATED TIMBER CONCRETE COMPOSITES

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ABSTRACT: Timber-concrete composite floor panels offer a lightweight structural floor system that can be used with a panelised construction method. Currently, prefabricated timber-concrete composite (TCC) floor solutions are rare. This is partly due to a lack of knowledge regarding the shear connectors that join the timber and concrete elements and the impact of additional interlayers between the timber and concrete elements is even less well understood. This research project was undertaken as part of the broader work being done for the GenZero initiative by the UK's Department for Education (DfE) working with industrial partners Smith and Wallwork Engineers, Ecosystems Technologies, Thorp Precast, and several others. In particular, the impact of two different shear connectors on the proposed 8m by 1.8m TCC slabs was investigated: a concrete notch and a steel dowel. Full-scale slab and small-scale shear experiments were carried out, supplemented by computational analysis, to establish the properties of specific connectors and their impact on whole slab dynamic and static performance. The inclusion of any interlayer immediately reduced structural performance, but varying the interlayer thickness and build up only had a small impact. A critical feature of shear connector design was found to be the extent to which a connector is vertically restrained.

KEYWORDS: Timber-concrete composites, Off-site manufacturing, Vibration performance, Finite Element modelling

1 INTRODUCTION

Timber-concrete composite floor panels offer a relatively lightweight structural floor system that can be used with a panelised construction method. Currently, prefabricated timber-concrete composite (TCC) floor solutions are rare. This is in part due to a lack of knowledge regarding the shear connectors that join the timber and concrete elements, and the impact of additional interlayers between those timber and concrete elements, which is even less well understood.

This research project was undertaken as part of the broader work being done for the GenZero initiative [1] by the UK's Department for Education (DfE) working with industrial partners Smith and Wallwork Engineers, Ecosystems Technologies, and Thorp Precast.

1.1 GENZERO

The DfE have identified the need to build up to 200 new secondary schools each year in the 2030s. The GenZero initiative looks at how future secondary schools could be designed to meet this demand while aligning with the government's commitment to net zero carbon emissions by 2050. The key aims of the initial GenZero research project was to develop a flexible secondary school design system which could be applied to multiple sites and achieve net zero carbon in both construction and operation.

The DfE aim to pioneer a new method to build these secondary schools, using the principles of Design for Manufacture and Assembly (DfMA) to design a kit of high-performance parts from which new schools can be developed, and then quickly assembled onsite.

An initial prototype of a single classroom was displayed at the COP26 Glasgow summit, which gave the opportunity for testing and improving the understanding of the floor slab solution. Other prototyping projects are ongoing.

1.2 GENZERO FLOOR SLAB

For the majority of the GenZero design a high-performance timber concrete composite floor panel was proposed. These prefabricated panels aimed to meet the structural requirements, as well as provide benefits for fire separation between floors, acoustic separation and thermal mass. The concrete would also provide a hard-wearing architectural finish, reducing the number of on-site trades and would minimise whole life carbon compared to finishes that need to be replaced every few years.

The floor panels span 8m and are 1.8m wide. The panel build-up, Figure 1, is made of a 75mm concrete slab, on top of an acoustic resilient layer (~5mm) supported on a timber boarding layer (24mm) which also acts as a permanent formwork. The concrete then acts compositely with the glulam ribs below. There is one central rib of 200x400mm with two half ribs (100x400mm) at each edge to create a continuous appearance underneath while helping to protect the concrete edges. The glulam is made using UK grown timber with a material grade of GL16+.

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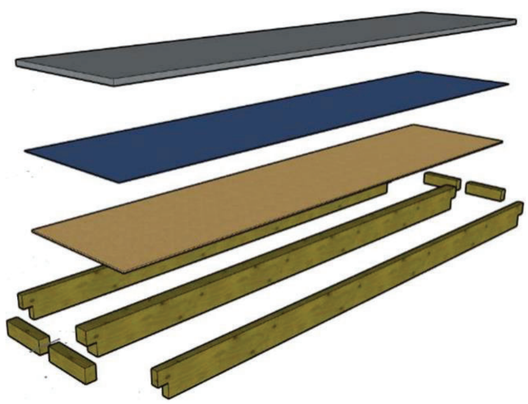

Figure 1: Exploded view of floor GenZero slab.

1.3 TECHNICAL MOTIVATION

The structural performance of timber or concrete is already well understood. What is less well understood are the methods of joining these materials to form a composite, especially with the inclusion of interlayers. The properties of these connectors have a significant impact on the performance of the overall structure, as the connectors dictate to what extent composite action occurs. This in turn determines the global stiffness of the slab. Then under large loads, the design of the shear connectors will govern the failure mode and the failure load. The key metrics of performance for a shear connector are its slip modulus and failure load, which can then be used to calculate the global slab properties.

There are evidently many methods to join timber and concrete to make a composite connector. The original GenZero research (a literature review and design exercise) identified two connector types to take forward from a literature review: a 20mm flanged steel dowel and a notch 50mm into the timber. See Figure 2. The assessment was based on a balance of structural performance, manufacturability and an attempt to limit the number of connectors for manufacturing ease and reduced noise transmission.

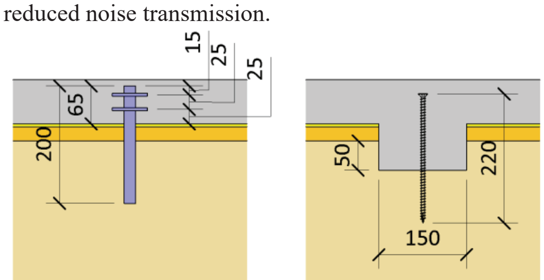

Figure 2: *Composite connectors, flanged dowel on the left, concrete notch on the right*

The two types of composite connector have been researched and tested before. However, the effect of adding interlayers between the concrete and the timber members is not adequately understood. Previous research has given flat reduction factors for strength and slip modulus values for interlayers. Table 1 shows the reduction factors put forward from the 2018 COST report "Design of timber-concrete composite structures" [2], which summarised available research.

Table 1: Reduction factors due to interlayers [2]

	Strength R.F. (%)	Stiffness R.F. (%)
Dowels	8	35
Notches	16	34

It is important to note that the reduction factors given in Table 1 are not a function of the interlayer thickness or interlayer material. It is intuitive to expect a thicker interlayer will have a greater impact on stiffness. The original GenZero prototype includes an interlayer of two materials with a thickness of ~ 30mm. More recent guidance published in March 2022 'Design of Timber Structures. Structural design of timber-concrete composite structures.' [3] is also limited on detail. For example, if the interlayer is as stiff as the timber "The Slip Modulus of dowel-type fasteners may be taken as that for a similar configuration without an interlayer with a reduction factor of 30%." Otherwise "the slip modulus should be determined by tests", which is what this report aims to provide for the GenZero design.

Previous research has typically used higher grade glulam timber, with greater stiffness and strength properties. The GenZero design differs by using softer UK grown glulam timber elements with a grade of GL16+. GL16+ was proposed by Edinburgh Napier University specifically for UK grown timber. Material properties for GL16+ are given in Table 2.

Table 2: *GL16+ timber properties*

Bending strength	$f_{m,g,k}$	22.5	N/mm^2
Tensile strength	$f_{t,0,g,k}$	18.0	N/mm^2
	$f_{t,90,g,k}$	0.5	N/mm^2
Compressive strength	$f_{c,0,g,k}$	22.5	N/mm^2
	$f_{c,90,g,k}$	2.5	N/mm^2
Shear strength	$f_{v,g,k}$	3.5	N/mm^2
	$f_{r,g,k}$	1.2	N/mm^2
Modulus of Elasticity	$E_{0,g,mean}$	8,400	N/mm^2
	$E_{0,g,0.05}$	7,000	N/mm^2
	$E_{90,g,mean}$	300	N/mm^2
Shear modulus	$G_{g,mean}$	650	N/mm^2
	$G_{r,g,mean}$	65	N/mm^2
Density	$\rho_{g,k}$	363	kg/m^3
	$\rho_{g,mean}$	400	kg/m^3

Dynamic performance typically governs the design of long-span timber floors such as the GenZero design. This research focuses on the small strain and dynamic behaviour of the composite shear connectors and full slab, with the aim of informing the development of prefabricated timber-concrete floor panels. This is done by analytical methods, finite element modelling of the shear connectors and the slab unit, small-scale lab testing

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comparing different shear connectors and interlayers, and full-scale lab testing of a complete slab focusing on its dynamic properties under footfall excitation.

2 CONNECTOR BEHAVIOUR

This section aims to establish the performance of the individual composite connectors identified for the GenZero prototype and explore the effect of the proposed interlayer. This is done through small scale tests of the connectors and through computational analysis.

2.1 EXPECTED VALUES FROM LITERATURE

The anticipated failure load and stiffness from the literature is given in Table 3. The flanged dowel values are based on existing research [4], with stiffness scaled with the timber density to power of 1.5. Values for the notch are based off guidance in the COST report [2], which gives a flat stiffness value based on notch width, with no relation to timber properties.

Table 3: *Predicted connector properties*

Connector	Predicted failure load, kN	Slip modulus, kN/mm
Flanged dowel	27	109
Flanged dowel w/ interlayer	25	71
Notch	29	120
Notch w/ interlayer	24	76.8

2.2 CONNECTOR SPECIMEN DESIGN

Four test specimens were constructed to cover both connectors and three different interlayers build ups, see Table 4 for a summary. The number of specimens was limited by the availability of UK grown glulam.

Table 4: *Small-scale test specimens (SF = Rothoblaas Silent Floor)*

Test	Shear connector	Interlayer depth, mm	Interlayer Composition
1	Dowel	0	None
2	Dowel	24	OSB
3	Dowel	29	SF + OSB
4	Notch	29	SF + OSB

The specimens were designed for symmetric shear tests with two connectors in each side, so that no additional moments would be placed on the shear connectors.
The geometry of each specimen was designed to replicate the full size GenZero slab, with the same timber dimensions as an outside rib and the same concrete thickness. Where the interlayer thickness is varied the dowel embedment depth is kept the same. The key variation from the full-scale slab is the concrete is unreinforced and has the same width as the beam.

2.2.1 CONCRETE

The GenZero design specified a concrete grade of RC30/37 with a 70% GGBS replacement mix. The concrete mix used for the tests did not include GGBS due to availability, however this should only affect curing times. Concrete test results are given in Table 6, showing the strength requirements were met.
The mix was relatively dry, making it harder to compact leading to concerns that voids may have formed in or near the connectors.

Table 5: *Concrete mix used in shear test specimens.*

Test type	Number of tests	Mean failure load (MPa)
Cube	3	42.4
Cylinder	2	32.7

2.3 SMALL SCALE TESTING METHOD

The small-scale shear tests were completed in accordance with BS EN 26891:1991 [5] to ensure relevance to all stakeholders. The loading regime is shown in Figure 3, specimens are loaded elastically, unloaded, and then loaded to failure. This allows connectors to embed and shows a hysteresis loop for the cycle. Testing was completed with an Instron machine using load control for the initial cycle then displacement control to prevent any sudden failure; Figure 4 shows a photo of the test set up.

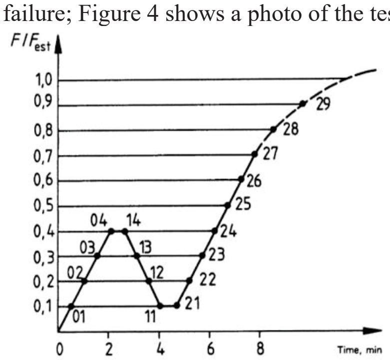

Figure 3: *Loading protocol [5]*

Figure 4: *Test set up, with specimen 1 after failure*

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The standard also provides the method to calculate the slip modulus from experimental results. This required calculating an estimate of the failure load for each specimen, which was typically done using values from literature. Test 4 was the exception, which was subjected to the same regime as Test 3 in order for the performance of the two different shear connectors to be better compared.

2.3.1 DIGITAL IMAGE CORRELATION

Digital Image Correlation (DIC), also called Particle Image Velocity (PIC), was used to measure strain in the specimens during testing. This is done by spray painting dots onto the specimen then comparing digital images to track deformations and calculate the strain field.

2.4 COMPUTATIONAL ANALYSIS

SIMSOLID was used to create a Finite Element Model of individual connections to extend the parameter field covered. FEA models were developed for the two connector types. This was proposed by the full-scale slab fabricator as it simpler to manufacture.

The interlayer was modelled as an air gap, to make a lower bound estimate where the interlayer material does not contribute to the stiffness, through friction or restraint.

2.5 RESULTS AND DISCUSSON

2.5.1 EXPERIMENTAL

Table 6 shows the characteristic connector properties calculated using BS EN 26891:1991 for each specimen with four connectors. The results for a single connector are then compared to the expected values from literature in Table 7. Initial impressions of the results show both the capacity and stiffness of the dowel connector drop significantly as the interlayer increases. Also, the notch connector slightly outperforms the dowel connector for the same interlayer.

Table 6: *Experimental specimen properties (4 connectors)*

	Property	Notation	Unit	Test 1	Test 2	Test 3	Test 4
1	**Maximum Load**	$\mathbf{F_{max}}$	**kN**	**107.5**	**78.9**	**69**	**108**
2	estimated maximum load	F_{est}	kN	100	90	80	80
3	initial slip	v_i	mm	0.88	1.54	1.54	1.41
4	modified initial slip	$v_{i,mod}$	mm	0.74	1.40	1.57	1.19
5	joint settlement	v_s	mm	0.14	0.14	-0.03	0.22
6	elastic slip	v_e	mm	0.57	0.76	1.01	0.45
7	initial slip modulus	k_i	kN/mm	45.3	23.4	20.7	22.7
8	**slip modulus**	$\mathbf{k_s}$	**kN/mm**	**53.7**	**25.8**	**20.4**	**27.0**
9	slip at 60% F_{est}	$v_{0.6}$	mm	1.31	2.23	2.51	1.77
10	modified slip at 60% F_{est}	$v_{0.6, mod}$	mm	1.15	1.94	2.49	1.49
11	slip at 80% F_{est}	$v_{0.8}$	mm	2.06	2.95	3.91	2.12
12	modified slip at 80% F_{est}	$v_{0.8, mod}$	mm	1.90	2.66	3.89	1.83

Compared to the literature the strength values are relatively similar. However, the experimental stiffness values are generally lower by a factor of 10, this is discussed later.

For the flanged dowel the literature gave a stiffness reduction of 35% where the experimental results show reductions of 51% and 62% for the OSB interlayer and the combined OSB and resilient layer interlayer respectively. This shows a flat reduction factor for any interlayer is unsuitable.

Table 7: *Predicted vs experimental connector properties*

Test	1	2	3	4
Predicted failure load, kN	27	25	25	24
Experimental failure load, kN	27	20	17	27
Predicted slip modulus, kN/mm	109	71	71	77
Experimental slip modulus, kN/mm	13.4	6.5	5.1	6.8

However, there is a significant discrepancy between the slip moduli of the experimental and literature values. This discrepancy can't be explained by the interlayer build-up as test 1 which had no interlayer is also significantly lower than the literature, suggesting something is wrong with the specimen design. None of the shear test specimens failed in the shear connectors. In each case there was bearing failure at the base of the specimen first. For example, Figure 4 show the cracking patterns of the concrete in failure of Test 1; note cracking at the base. The onset of this cracking may have reduced the stiffness of the shear connectors in the specimen. Therefore, the results from the small-scale tests are indicative of the relative performance of the shear connectors with given interlayer but are not reflective of the true absolute values of the slip moduli and shear strength.

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Only a narrow strip of concrete was used in each specimen, rather than having the concrete overhang to replicate a larger part of the slab. A larger concrete area would increase the crushing failure load. The use of a dry mix, resulting in voids inside the specimen (Section 2.3.6) will have exacerbated this problem. Finally, the concrete was completely un-reinforced. Including a reinforcement mesh would be more representative.

2.5.2 COMPUTATIONAL

The absolute values for slip modulus the FEA produced were substantially higher than the literature and experimental values, see Table 8. This is potentially the result of imperfections in real world samples. For example, within a concrete notch, the concrete face in unlikely to be perfectly smooth, meaning protruding elements are likely to bed into the timber until a sufficient surface area is in contact with the timber, effectively reducing the initial slip modulus. Similarly, this could happen with hole tolerances with the dowel connector.

Table 8: *FEA slip moduli*

Interlayer Thickness, mm	Slip Modulus, kN/mm (Normalised)		
	Rectangular notch	Flanged dowel	Dowel Experimental
0	363 (1.0)	297 (1.0)	13.4 (1.0)
5	225 (0.62)	144 (0.48)	6.5 (0.49)
29	206 (0.57)	93 (0.31)	5.1 (0.38)

More interesting is the normalised slip modulus for each connector type. These are shown for a variety of interlayer thickness in Figure 5. Each connector shows a significant drop in slip modulus as the gap first starts, then a more steady, approximately linear reduction. This indicates a flat reduction factor is suitable for thin interlayers approx. <5mm. However, for larger interlayers the linear drop does become pronounced, showing the limitations of the current literature.

The FEA models and experimental values also show a larger initial reduction compared to the literature values, in particular the dowel reduction is substantially larger. This suggests a larger experimental set with the improvements discussed in Section 2.5.1 is required to fully characterise the behaviour of these connectors.

3 FLOOR SLAB BEHAVIOUR

Two full size GenZero floor slabs were constructed for structural testing as part of the prototype build. Testing included impact vibration tests, and a 3-point bend test with cyclic loading. These tests aimed to verify whether the original design met the structural requirements and to explore the properties of the composite connectors through back calculation of the whole slab's behaviour.

3.1 SLAB DESIGN AND MANUFACTURE

Each slab has the same dimensions and build up with the full interlayer (24mm OSB and 5mm resilient layer), shown in Figure 6. Connector spacing was kept the same for both slabs to ensure a direct comparison. Along each rib connectors were spaced at 1m intervals. In the wider central rib, two dowels were used at each interval and double width notch was used.

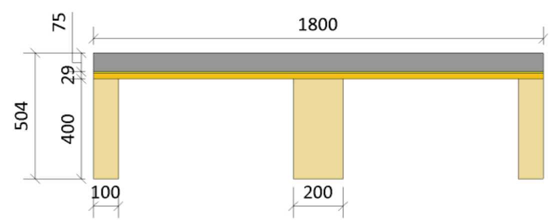

Figure 6: *GenZero slab cross-section*

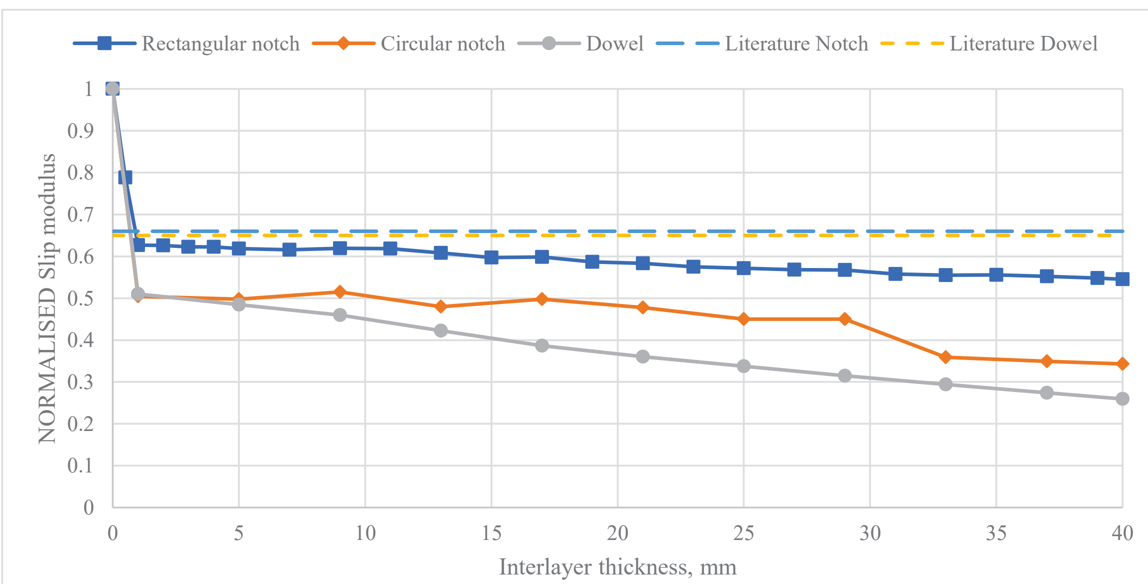

Figure 5: *Normalised slip modulus for different interlayer thickness, comparing results from SIMSOLID simulation and literature*

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ECOSystems Technologies fabricated the timber elements, connectors and applied the resilient layer, then the slabs were transported to Thorp Precast to pour and polish the concrete.

The C16+ timber that made up the glulam beams of the prototype outperformed initial expectations of stiffness. Research from Edinburgh Napier University demonstrated that the timber performed closer to a C24 classification, since the modulus of elasticity of the glulam was 11GPa rather than the initially assumed 8GPa. The high GGBS content of the concrete slab made power floating more difficult. The prototype slabs had a slightly curved profile on their surface, with more cover at the edges than in the middle. This meant a few of the steel dowel shear connectors were visible. Variation in the slabs' mass can be seen in Table 9.

The dowel connectors were intended to be glued in; however the epoxy was missed, which could have a significant impact on performance.

Table 9: *Prototype slab masses*

Prototype shear connector	Weight (kN)
Notch	33.28
Dowel	33.33

3.2 VIBRATION TESTING

3.2.1 TEST METHOD

Dynamic testing was completed using an instrumented hammer, see Figure 7, with accelerometers glued to the floor slab. The hammer was used to excite the slab vertically while it was simply supported at each end. This would enable measurement of the fundamental frequency of the slab, which is the most relevant for excitation from walking. The fundamental frequency of the slab is a property of the slab and therefore would be independent of where the hammer blow is struck and where the receiving accelerometer is placed. However, the exception is that a mode will neither be excited by the hammer nor received by the accelerometer if either of those two points on the slab coincide with a nodal point of the vibration in that mode. Therefore, data was collected from a variety of locations to help produce confidence in the nature of the resonant peak being detected. Numerous off-centre impacts from the instrumented hammer and off-centre accelerometer measurements were taken to avoid mistaking any torsional or minor axis flexural modes of vibration for the fundamental mode.

Figure 7: *Instrumented hammers on test slab*

3.2.2 DYNAMIC RESULTS

Figures 8 to 10 show the key frequency data for each slab. Figures 11 and 12 show a spiral plot for the notched connector slab based on the equation of motion:

$$F = m.\ddot{x} + v.\dot{x} = k.x$$

The accelerometer glued to the slab, records the acceleration of the slab, which can be numerically integrated to find the velocity and position. After the initial impact the external force, F = 0. Therefore, if each of the exponents are constant (the slabs' mass, stiffness and damping factor) the equation of motion defines a plane in space on which the oscillations of the slab create an ellipse. As the oscillations decay this ellipse decays into a spiral.

Table 10 shows the key results from these figures, showing the notch connector performs better than the dowel. From these values and the slab masses response factors for a range of input walking frequencies were calculated, using the method from "A Design Guide for Footfall Induced Vibration of Structures" [6], the results are shown in Figures 13 and 14.

Table 10: *Experimental dynamic values*

	Natural Frequency (Hz)	Damping Factor (% critical)
Dowel	8.6	2
Notch	10.2	4.5

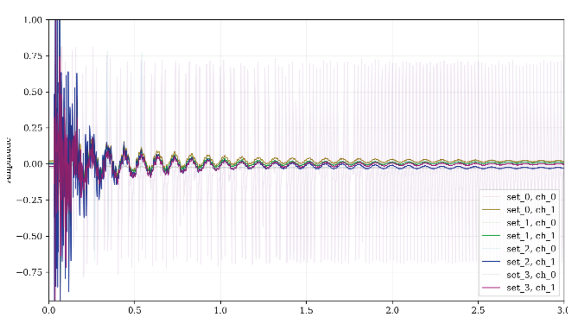

Figure 8: *Time domain data of notch connector slab*

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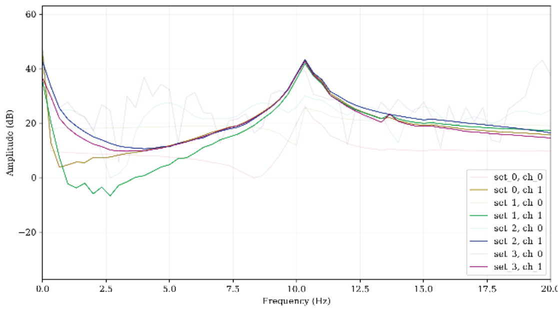

Figure 9: *Frequency domain data for notch connector slab*

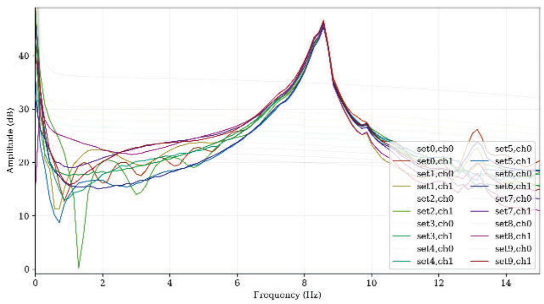

Figure 10: *Frequency domain data for the dowel connector slab*

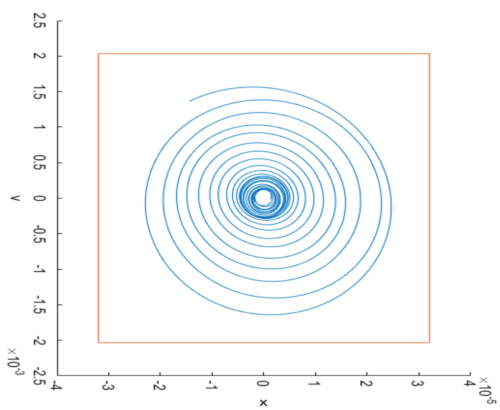

Figure 11: *Vibration decay spiral of notch connector slab*

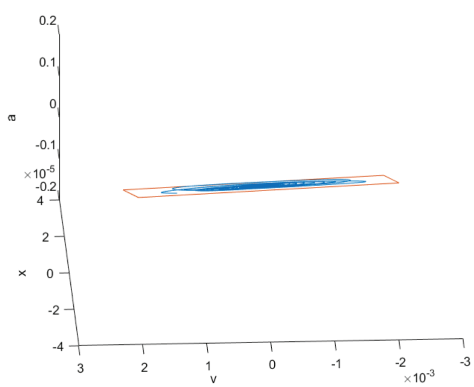

Figure 12: *Vibration decay spiral of notch connector slab*

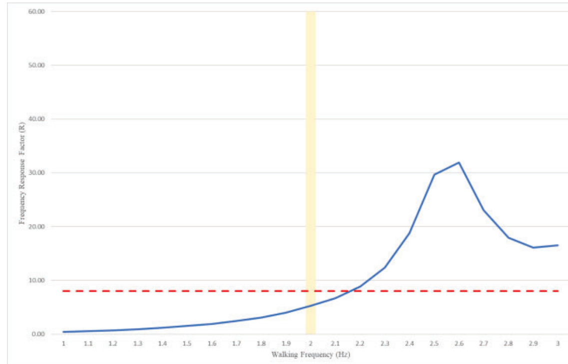

Figure 13: *Response factors for different walking frequencies of the notched connector slab, the yellow line indicates the max expected walking frequency in a classroom, the red line indicates the max permissible response.*

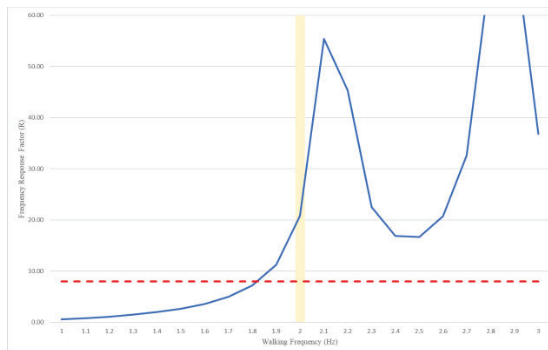

Figure 14: *Response factors for different walking frequencies of the dowel connector slab, the yellow line indicates the max expected walking frequency in a classroom, the red line indicates the max permissible response.*

3.2.3 DISCUSSION OF RESULTS

There is a clear difference between the appearance of the fundamental resonant peak of the notch slab and the equivalent resonant peak in the dowel slab, compare Figures 9 and 10. The clean resonant peak in the notched slab is what one would normally expect to see. The shape of the resonant peak in the dowel slab indicates that there are in fact two resonant peaks at very similar frequencies superimposed on top of one another. One possible explanation would be to suggest that there were two modes present with similar natural frequencies, for example a bending mode and a torsional mode that were both excited at the same time. However, this would be very unlikely given that torsional modes should only occur at higher frequencies due to the long thin shape of the slab. This was confirmed by impacting the slab at different locations. By impacting the slab near the edge, one would expect to excite a torsional mode, whereas impacting the slab along at centre line should excite no torsional modes. In all the above cases the same 8.6Hz peak was present with the same shape, and no torsional modes in the considered frequency range were observed.
Instead, the shape of the peak is indicative of an asymmetric slab. If the left half of the slab and the right

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half of the slab were built slightly differently, with subtly different masses and stiffnesses, then the left half of the slab would have a subtly different resonant frequency to the right half. Then the two resonant peaks would superimpose and join into one resonant peak with a shape similar to what was observed. This is the most likely explanation since it is consistent with the defects recorded, for example one of the glulam ribs was 10mm thinner in width than specified.

Often a linear damping factor is assumed to simplify analysis, Figures 11 and 12 demonstrate that damping was linear in both slabs. As discussed in Section 3.2.2, if the damping factor, v, was linear the spiral would lie on a plane, as can be seen. However, if the damping factor was non-linear a 'bowl' shape would be observed instead.

The notch shear connected slab performed better than the dowel slab. However, this is not due to the performance of the shear connector alone. The dowel slab had more issues with its manufacture and the dowel connectors were not glued into the timber.

3.3 3-POINT BEND TEST

3.3.1 TEST METHOD

There is no specific standard for testing TCC slabs, therefore a test method was developed based on the protocol for steel concrete composites (SCC). "Design of composite steel and concrete structures" [7] lays out methods for testing both shear connectors and composite slabs. This uses a four-point bend test with an initial cyclic loading phase before the slab is taken to failure.

To test the TCC slabs, a 3-point test is used with an initial phase of 25 cycles to 30% of the estimated failure load. A 3-point bend test was justified as the primary interest of this test is the slab stiffness, and it would make a shear failure more likely which is of greater interest to evaluate the shear connectors. The number of cycles is reduced from the SCC test as steel fatigue is less of an interest here, this is also seen in the small-scale test method where timber connections require a single cycle and SCC connectors require 25.

To collect the data, multiple techniques were used. DIC imaging was used to monitor the slab by the supports. LED monitoring was used to track relative movement of different regions of the slabs. Transducers were placed under each rib, to measure deflection.

3.3.2 RESULTS

Table 11 shows key results from the test. There was a hydraulic fault in the actuator during testing of the notch prototype slab. There was a control error with the hydraulic actuator, resulting in a sudden impact on the notch slab. This made a loud sound, including cracking from the slab. This impact recorded a maximum load of 105kN. However, this value is likely to be an overestimate due to the water hammer pressure excited within the actuator as a result of this sudden load.

The remainder of the test on the notch slab was conducted in displacement control, but the need to adjust the program led to interrupted data acquisition from the actuator.

The initial failure mode of the notch slab was in the shear connectors. The initial failure mode of the dowel slab was in bending.

Table 11: *Experimental slab performance.*

Prototype slab	**Notch**	**Dowel**
Elastic limit (kN)	93	100
Peak load (kN)	117	104
Bending stiffness (N/mm^2 $x10^{12}$)	59.3	23.7

Both slabs had significant post failure strength and ductility. The notch slab was able to resist the full extension of the actuator of approximately 160mm. When the load was removed the slab rebounded, but not to its original height. The dowel slab did not rebound after its final failure at its peak load near the maximum extension of the actuator.

3.3.3 DISCUSSION OF RESULTS

Using the methods of built-up sections, from EC5 [8] it is possible to back-calculate the slip modulus of the shear connectors in a slab from the experimental values of central deflection in bending as shown in Table 12.

Given that a negative slip modulus is impossible, the result indicates that the dowel slab is performing worse than would be expected if no composite action was occurring at all. This cannot be a consequence of the performance of the shear connector alone. Nevertheless, the result demonstrates that the dowel performed very poorly. The calculation was based on the design dimensions of the slab, but as discussed in Section 3.1 the prototype slabs contained defects.

Table 12: *Connector slip moduli calculated from bend test*

Prototype shear connector	Slip modulus (kN/mm)
Notch	45.0
Dowel	-1.1

Table 13: *Revised connector slip moduli calculated from bend test, to account for manufacture*

Prototype shear connector	Slip Modulus (kN/mm)
Notch	69.5
Dowel	1.0

A revised calculation was then performed, reducing the cross-section to match the as built dimension and accommodate for imperfections in manufacture. Specifically, Table 13 was produced by reducing the concrete depth by 10mm to 65mm, as some dowels were partially exposed from the polishing and assuming each rib had been planned by 10mm each on average, reducing to total width of glulam from 400mm to 360mm.

However, this does not account for the fact that the slab with dowel shear connectors was generally in a poorer condition with more defects before the test than the slab with notch shear connectors. This brings the slip modulus value of the notch more in line with the literature value. Due to the construction defects of the dowel slab, in particular the missing glue in the dowel connector, these full-scale results are disregarded.

3.4 DYNAMIC VS BENDING STIFFNESS

The bending stiffness of the two full scale slabs can be back calculated from both the vibration tests and the 3-point bend tests to show the effects of load duration, these are shown in Table 14. As is typically expected both slabs are stiffer during the dynamic tests, by 23% and 119% for the notch and dowel respectively. The greater drop in stiffness of the dowel connector slab suggests it is more susceptible to degradation over time, this may be from increased cracking in the concrete or greater embedment of the dowels into the timber.

Table 14: *Slab bending stiffness*

	Bending stiffness (N/mm^2 x10^{12})
Full composite action	122.3
No composite action	25.4
Notch (static)	59.3
Notch (dynamic)	73.2
Dowel (static)	23.7
Dowel (dynamic)	52.1

3.5 COMPUTATIONAL ANALYSIS

An ABAQUS FEA model of the full-scale slab was created to explore the internal forces acting within the slab to understand what elements are affecting the performance of the concrete slabs.

3.5.1 MODELING UNCERTAINTIES

The glulam was modelled as a homogenous anisotropic material rather than as a built-up section of lamina. The material properties assigned in the test were based on Eurocodes [7] in the case of concrete, this causes some uncertainty as the actual value as is dependent on cracking. Values for the glulam were based on recent research from Edinburgh Napier University, while mean values are used in the model there is still significant natural variation within timber causing more uncertainty.

3.5.2 MODELING INSIGHTS

The key insight provided from modelling was the importance of vertical restraint to the composite connectors. If a shear connector is allowed to lift, it provides much less resistance to shear than if it is held down. Figures 15 and 16 show the normal stress distributions in a TCC slab, for a notch shear connector both with vertical restraint, replicating the fully threaded screw in the connection, and without vertical restraint, no additional screw. When there is no restraint, the connectors lift and the slab provides virtually no structural contribution, Figure 16. In comparison Figure 15 shows the slab stressed, in a pattern consistent with the connector locations.

This also helps explain the particularly poor performance of the experimental dowel slab, because the glue was missed from the dowels meaning there was limited vertical restraint.

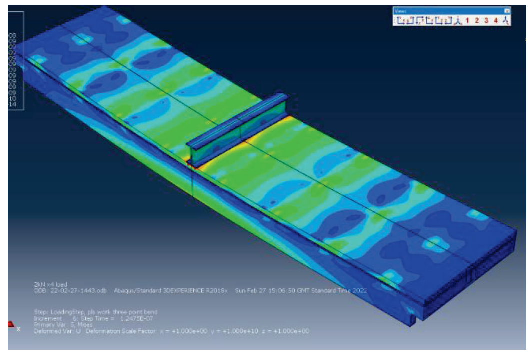

Figure 15: *Notch connector slab with vertical restraint*

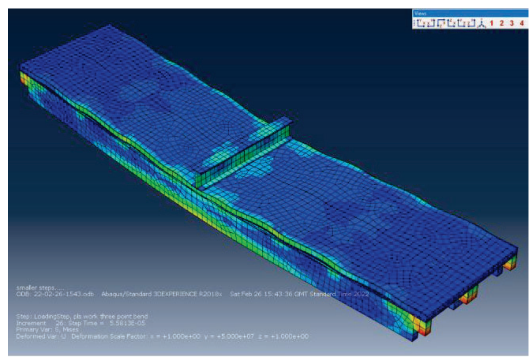

Figure 16: *Notch connector slab without vertical restraint*

4 GENZERO DESIGN EVALUATION

4.1 STRENGTH PERFORMANCE

To meet ULS requirements (G_k =0.5kN/m^2, Q_k=3.8kN/m^2) both slabs must be able to resist an equivalent central point load for bending strength: $P = 46$ kN, and an equivalent central point for shear strength: $P = 92\ kN$. Shear failure is the limiting parameter and both slabs pass.

Table 15: *Satisfaction of strength criteria*

	Required Maximum load (kN)	Achieved Maximum load (kN)	Result
Notch	92	117.5	**PASS**
Dowel	92	100.2	**PASS**

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4.2 DEFLECTION PERFORMANCE

Under SLS loading, the equivalent central point load, P = 38.9 kN. The instantaneous deflection limit = $span$ / 250 = 32.2mm. Both slabs pass.

Table 16: *Satisfaction of deflection criteria*

	Permitted deflection (mm)	Deflection under load (mm)	Result
Notch	22.4	7.9	**PASS**
Dowel	22.4	12.1	**PASS**

4.3 VIBRATION PERFORMANCE

GenZero put forward two vibration performance criteria, a minimum natural frequency of 8Hz with 10% live load applied, noting however vibration tests were completed with no live load. Then a maximum frequency response of 8 for excitation walking frequencies up to 2Hz as suggested in [6]. Both slabs pass the basic natural frequency test, only the notched connector passes the frequency response test.

Table 17: *Vibration criteria and results*

	Minimum resonance (Hz)	Natural frequency (Hz)	Result
Notch	8.00	10.22	**N/A**
Dowel	8.00	8.56	**N/A**

	Allowable FR	Maximum FR	Result
Notch	8	6.8	**PASS**
Dowel	8	22.1	**FAIL**

5 CONCLUSIONS

There was no case where an interlayer improved structural performance. Each connector saw a large initial drop off before a reduced linear decline. Therefore, in design an interlayer should be avoided. If absolutely required, the thickness of the interlayer should be minimised.

A key attribute of a composite connector is vertical restraint. The full-scale slab modelling and the missing epoxy of the dowels showed the significant impact of removing this restraint.

The notch connector outperformed the dowel in all tests completed. Future work could investigate the effect of notch shape to help simplify manufacture, circular notches are typically easier to cut. Initial modelling of a circular notch with similar dimensions to the rectangular notch produced a less stiff connector.

The GenZero slab design was shown as viable. The full-scale tests can be taken as a lower bound of performance due to the manufacturing imperfections. Key to making this solution a reality would be manufacturing consistency.

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