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THIN TOPPING TIMBER-CONCRETE COMPOSITE FLOORS

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A thesis submitted for the degree of Doctor of Philosophy

University of Bath

Department of Architecture and Civil Engineering

May 2013

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Abstract

A timber-concrete composite (TCC) combines timber and concrete, utilising the complementary properties of each material. The composite is designed in such a way that the timber resists combined tension and bending, whilst the concrete resists combined compression and bending. This construction technique can be used either in new build construction, or in refurbishment, for upgrading existing timber structures. Its use is most prolific in continental Europe, Australasia, and the United States of America but has yet to be widely used in the United Kingdom.

To date, the topping upgrades used have been 40mm thick or greater. Depending on the choice of shear connection, this can lead to a four-fold increase in strength and stiffness of the floor. However, in many practical refurbishment situations, such a large increase in stiffness is not required, therefore a thinner topping can suffice. The overarching aim of this study has been to develop a thin (20mm) topping timber-concrete composite upgrade with a view to improving the serviceability performance of existing timber floors. Particular emphasis was given to developing an understanding of how the upgrade changes the stiffness and transient vibration response of a timber floor.

Initially, an analytical study was carried out to define an appropriate topping thickness. An experimental testing programme was then completed to: characterise suitable shear connectors under static and cyclic loads, assess the benefit of the upgrade to the short-term bending performance of panels and floors, and evaluate the influence of the upgrade on the transient vibration response of a floor.

For refurbishing timber floors, a 20mm thick topping sufficiently increased the bending stiffness and improved the transient vibration response. The stiffness of the screw connectors was influenced by the thickness of the topping and the inclination of the screws. During the short-term bending tests, the gamma method provided a non-conservative prediction of composite bending stiffness. In the majority of cases the modal frequencies of the floors tested increased after upgrade, whilst the damping ratios decreased. The upgrade system was shown to be robust as cracking of the topping did not influence the short-term bending performance of panels.

Thin topping TCC upgrades offer a practical and effective solution to building practitioners, for improving the serviceability performance of existing timber floors.
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List of Symbols

$A$  Cross-sectional area

$C_p$  Dimensionless parameter

$D_{xy}$  Torsional rigidity of the floor

$D_x$  Bending stiffness parallel to the direction of the joists

$D_y$  Bending stiffness perpendicular to the direction of the joists

$E$  Modulus of Elasticity

$EI_h$  Bending stiffness perpendicular to the direction of the joists

$EI_{ef}$  Effective bending stiffness of composite

$EI_l$  Bending stiffness parallel to the direction of the joists

$EI_{max}$  Maximum effective composite stiffness

$EI_{timber}$  Effective bending stiffness of a timber floor

$E_{global}$  Global Modulus of Elasticity

$F$  Applied force

$F_{est}$  Estimated maximum load

$F_{max}$  Maximum load

$F_{x,max}$  The load at which the maximum displacement occurs each load cycle

$F_{x,min}$  The load at which the minimum displacement occurs each load cycle

$G$  Shear Modulus

$I$  Second moment of area

$K_{0.4}$  Slip modulus at 40% of the maximum load

$K_{0.6}$  Slip modulus at 60% of the maximum load
$K_e$  Elastic slip modulus

$K_s$  Serviceability limit state slip modulus

$M$  Bending Moment

$X$  Amplitude

$\ddot{x}$  Acceleration

$a$  Spacing between connectors

$a_{\text{max}}$  Peak accelerance

$b$  Floor breadth

$b_c$  Spacing of joists

$b_t$  Breadth of joists

$c$  Damping coefficient

$c_{\text{cr}}$  Critical damping coefficient

$d$  Diameter of connector

$e$  Interfacial strain difference

$e_d$  Energy dissipated

$f_1$  First modal frequency or natural frequency

$f_{i,j}$  Modal frequency

$h_c$  Topping depth

$h_t$  Timber joist depth

$i$  Mode number: x-direction

$j$  Mode number: y-direction

$k$  Stiffness or curvature

$k_{\text{amp}}$  The amplification factor to account for shear deformation is solid timber joists

$k_{\text{dist}}$  The proportion of load acting on a single joist

$l$  Span
\( m \) Mass of a body or mass per unit area

\( n \) Modular Ratio

\( n_{40} \) Number of first order modal frequencies below 40Hz

\( q_k \) Imposed load

\( s \) Longitudinal slip

\( s_{\text{end}} \) End slip

\( s_{\text{sep}} \) Vertical separation of timber and topping

\( t \) Thickness of interlayer

\( v \) Unit velocity response

\( w \) Instantaneous deflection

\( w_C \) Theoretical composite deflection with complete shear connection

\( w_I \) Experimental composite deflection with partial shear connection

\( w_N \) Theoretical composite deflection without shear connection

\( x \) Displacement

\( x_{\text{max}} \) The maximum displacement in a load cycle

\( x_{\text{min}} \) The minimum displacement in a load cycle

\( \Omega \) Ratio between the excitation frequency and undamped natural frequency

\( \gamma \) Shear bond coefficient

\( \omega \) Angular frequency

\( \omega_d \) Damped angular frequency

\( \omega_n \) Undamped angular frequency

\( \phi \) Screw inclination from the horizontal

\( \rho_m \) Mean density of timber

\( \varphi \) Phase Angle

\( \zeta \) Equivalent viscous damping ratio
1. Introduction

A timber-concrete composite combines timber and concrete, utilising the complementary properties of each material. The composite is designed in such a way that the timber resists combined tension and bending, whilst the concrete resists combined compression and bending. Recognised advantages of the TCC construction system are: its improved stiffness; strength; vibration performance; fire resistance and acoustic performance compared to timber floors, and its reduced self-weight and rapid execution in comparison to concrete slabs (Ceccotti, 1995; Yeoh et al., 2011b).

Use of TCCs is most prolific in continental Europe, Australasia, and the United States of America (Yeoh et al., 2011b) but as yet the system has not been widely used in the United Kingdom. There are two main applications of this construction technique; new build construction and upgrade of existing structures. In the UK there is a trend towards living in older properties which require upgrade and refurbishment, 81% of dwellings in England predate 1980 (Communities and Local Government, 2007). Conversely modern expectations are for buildings that perform at higher standards than in the past. In particular, apartments require acoustic separation and excessive vibration is unacceptable. These problems are more likely to be prevalent when existing buildings are converted into multiple dwellings, as neighbours move closer in proximity to one another, and buildings that are converted are particularly old (85% of converted buildings were originally built before 1919 (Communities and Local Government, 2007)). As the UK has a large existing building stock and a history of conservation, renewal and refurbishment, the development of a TCC system for upgrading existing timber floors seems most appropriate for introduction to the construction sector.

1.1. Background of Timber-Concrete Composites

The refurbishment of existing timber floors is already well documented. One of the earliest known examples is in Bratislava in 1960 (Postulka (1983), cited by Lukaszewska (2009)) where 6.3 x 180mm long nails were used as shear connectors between the timber and topping. The upgrade was estimated at
half the cost of replacing the existing floor and resulted in approximately a four-fold increase in the floor’s strength and stiffness. Furthermore in 1981, Lódź, Poland, (Godycki et al. (1984) cited by Van der Linden (1999)) 1000m² of timber floor was refurbished using concrete joined with nails. More recently there have been reports of historic buildings in Italy refurbished with concrete toppings using glued in dowels as shear connectors (Turrini & Piazza (1983) cited by Lukaszewska (2009)).

In the UK, some historical buildings retain lime ash floors, a mixture of under and over-burnt lime, wood ash, sand and water laid to a depth of approximately 75mm on reeds consolidated with clay, fixed between timber joists Wright (1999). These floors bear little resemblance to current TCC floors, which are much stiffer owing to the shear connection between the joists and concrete topping. Whilst it is not believed that concrete toppings have been used to upgrade UK timber floors, one recent example of a newly constructed floor exists; a cross laminated TCC dance floor in a Cambridgeshire school (Figure 1.1).

![Figure 1.1.: CLT-Concrete composite floor, Thomas Clarkson Academy Cambridgeshire. Left: Floor prepared for upgrade. Right: HBV Shear Connector (reproduced with permission from Ramboll UK)](image)

### 1.2. Rationale for Thin Topping Upgrades

Existing timber floors often suffer serviceability problems including: excessive deformation, poor vibration performance and unwanted noise transmission (Dias et al., 2013; Ceccotti, 1995). Whilst noise and vibration problems do not result in structural failure, the disruption to occupants and the cost of remediation can be high (Trada, 2012). To date, topping upgrades have been 40mm thick or greater. Depending on the choice of shear connection, this can lead to approximately a four-fold increase in the strength and stiffness of the floor (Ceccotti, 1995). However, in many practical refurbishment situations such a large increase in stiffness will not be required. For example, to stiffen
a floor for a change of use from residential (1.5kN/m² imposed load) to office (2.5kN/m²) occupancy would only require a 67% increase in stiffness. From this perspective it seems there is scope to reduce the thickness of the topping.

Advantages of thinner toppings include:

- a reduction in the change in floor to ceiling height;
- a reduction in the mass added to the existing structure;
- a reduction in the need for propping during construction due to the reduced additional mass.

1.3. Scope of Thesis

In order to successfully develop a thin topping upgrade system, constraints were put on the scope of the research. Priority was given to development of the shear connection system as it is the most critical part of a composite system (Van der Linden, 1999). Following this objective, preference was given to understanding the performance of the composite under static and dynamic loading.

Resources were not directed towards refinement of the topping material, instead an existing polymer modified screed was used, enabling investigation of the structural performance to take place. Effort was not given to important non-structural aspects of performance, such as acoustic and fire performance as they were considered as aspects suitable for a second study. Furthermore, attention was only given to instantaneous deflections and not long-term deflections as the duration of the project was insufficient to develop a suitable connection system as well as conducting creep testing.

1.4. Aims and Objectives

The overarching aim of this study was to develop a thin topping TCC upgrade to improve the serviceability performance of existing timber floors. Within this aim, particular importance was given to understanding how an upgrade changed the stiffness and vibration response of a timber floor. These aims were achieved by completing the following objectives:

- An understanding of how the thickness of a topping upgrade influences the change in stiffness and vibration response of a timber floor has been developed.
• Shear connectors from existing technologies, which could be suitable for use with thin toppings have been identified from pushout testing. Further experimental study, utilising a factorial design approach, established how the stiffness and strength of screw connectors was influenced by the thickness of the topping, screw inclination and timber density. In total, 108 specimens were tested.

• A method for establishing the dynamic stiffness and energy dissipation properties of connectors under cyclic loads has been devised.

• Nine panels have been subjected to short-term bending tests to investigate: how a topping upgrade influences the bending stiffness and strength of composite floors; the bending stiffness of panels is influenced by in-service loading; how accurately analytical models predict the bending stiffness of TCC floors and to establish whether cracking the topping affects the performance of TCC floors.

• Vibration tests of two floors have been completed to establish the influence of the topping on the vibration response with respect to: modal frequencies; modal damping ratios and accelerations.

• A short-term bending test of a floor to collapse has been completed to: compare floor performance to panel performance; load sharing between joists and the accuracy of analytical models for predicting the bending stiffness of TCC floors.

1.5. Thesis Outline

This thesis is comprised of eight chapters and four appendices. A review of research to date is presented in Chapter 2. A study investigating the influence of the topping thickness on the stiffness and vibration response of timber floors is presented in Chapter 3. Characterisation of shear connectors with thin toppings under static loads is reported in Chapter 4. A method for establishing the dynamic stiffness and energy dissipation properties of connectors under cyclic loads is reported in Chapter 5. The short-term bending testing of nine panels is reported in Chapter 6. Vibration testing of two floors and the short-term bending test of one floor to collapse, at the Building Research Establishment, is reported in Chapter 7. The thesis is concluded in Chapter 8 where suggestions are outlined for future research. A short introduction to vibrating systems is presented in Appendix A. A derivation of the equations for partial interaction is presented from first principles in Appendix B. The general differential equation found in Appendix B is solved in Appendix C for the case of a simply supported
beam subjected to a central point load. The gamma method (γ-method), an exact solution of the equations for partial interaction for the case of a simply supported beam with a bending moment that varies sinusoidally, is presented in Appendix D. Finally plans, sections and elevations of the panels tested in Chapter 6 and the floor tested in Chapter 7 are presented in Appendix E.
2. Literature Review

Ten main themes are reviewed in this chapter: the vibration of rectangular orthotropic plates; the vibration of timber floors; the vibration of TCC floors; the human perception of vibration; composite stiffness and analysis; TCC shear connectors; small amplitude cyclic testing of timber joints; the influence of the topping on TCC performance; the acoustic performance of timber and TCC floors and previous thin structural topping studies. A short summary concludes the chapter.

2.1. Vibration of Rectangular Orthotropic Plates

Timoshenko (1964) outlined how ribbed plates, which are similar to timber and TCC floors, could be modelled as orthotropic plates. Equation 2.1 for calculating the natural frequency of an orthotropic plate, simply supported along all its edges, was reviewed by Leissa (1969) and its derivation, attributed to Hearmon (1946).

\[
f_{i,j} = \frac{\pi}{2l^2 \sqrt{m}} \sqrt{D_x i^4 + 2D_{xy} i^2 j^2 \left( \frac{l}{b} \right)^2 + D_y j^4 \left( \frac{l}{b} \right)^4}
\]  

(2.1)

Where:

- \( f_{i,j} \) is the modal frequency under consideration.
- \( i \) is the mode number parallel to the direction of the joists.
- \( j \) is the mode number perpendicular to the direction of the joists.
- \( l \) is the floor span.
- \( b \) is the floor breadth.
- \( m \) is the mass per unit area of the floor.
- \( D_x \) is the bending stiffness parallel to the direction of the joists.
- \( D_y \) is the bending stiffness perpendicular to the direction of the joists.
- \( D_{xy} \) is the torsional rigidity of the floor.
Then Ohlsson (1988) assumed that for timber floors the torsional rigidity and the stiffness perpendicular to the direction of the joists were approximately equal, and concluded that Equation 2.1 could be simplified to form Equation 2.2.

\[
f_{1,j} = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}} \cdot \sqrt{1 + \left[2j^2 \left( \frac{l}{b} \right)^2 + j^4 \left( \frac{l}{b} \right)^4 \right] \cdot \frac{(EI)_b}{(EI)_l}}
\]  

(2.2)

Equation 2.3, adopted by EN1995-1-1, introduced a further simplification by presuming that the ratio of the transverse and the longitudinal stiffness of a timber floor is always small and therefore the second square root term of Equation 2.2 can be assumed to be equal to 1.

\[
f_{1} = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}}
\]  

(2.3)

As timber floors are often assumed to have little or negligible stiffness perpendicular to the joists the above assumption is valid for many floors. For countries where this is deemed inappropriate, due to contributions to the transverse stiffness from strutting, floorboards and plasterboard, Equation 2.2 is included in the appropriate National Annex. In the UK this is not the case and Equation 2.3 from part 1-1 of EN1995-1-1 (CEN, 2004a) is used. For a TCC floor, the stiffness perpendicular to the joists is far greater than that of a timber floor and if ignored would lead to unnecessarily conservative designs. For TCC floors constructed with cross-laminated timber, where the transverse stiffness of the timber alone is substantial, or more conventional timber joist and slab arrangements without a large degree of composite action, the error would be largest. Whereas a conventional arrangement with very good composite action would be the least affected.

2.2. Vibration of Timber Floors

The influence of supports, floor mass, stiffness perpendicular to the direction of the joists, and human occupancy on the transient vibration of timber floors are discussed in this section. The section is concluded with the design criteria provided by EN1995-1-1 part 1-1 to assess whether the response of a timber floor will be perceptible.

2.2.1. Influence of Supports

Chui et al. (2004) investigated the influence of end support condition on the static and dynamic response of timber floors. Two floors were modelled, floor
A with timber I-joists and floor B with open web joists, to compare direct bearing, toe-nailing, direct nailing and joist hanger type supports. Chui et al. (2004) found that only direct bearing produced a rigid support and that the other supports influenced the static deflection marginally, but reduced the natural frequency by up to 15%. The investigations also compared beam and rigid support conditions for floor aspect ratios of 1 and 2 finding that the static deflection was 7% lower and the natural frequency 17% to 30% lower for the beam support.

This study (Chui et al., 2004) highlights the importance of support stiffness to the vibration response of timber floors. The authors commented that the natural frequency of timber floors tested in the field were often lower than those tested in the lab. They indicated that this is sometimes caused by the shrinkage of joists at the support which leaves a gap between the support and part of the joist end. This in turn reduces the bearing area and consequently the bearing stiffness of the support. In practice a range of support stiffnesses will be found and whilst this undoubtedly affects the vibration performance of timber floors, it does not necessarily affect the extent to which a topping upgrade changes the vibration performance.

Chui & Smith (1989) reported how the support conditions of a timber beam affect its damping properties. They vibrated 1.05m lengths of white spruce with toggle clamp, knife edge and free-free spring supports and found that the specimens with free-free spring supports had an equivalent viscous damping ratio of 1%. This damping ratio is the material damping, as the spring supports have minimal stiffness compared to the beams. The toggle clamps had the highest damping, at an average of 8.75%, although this was presumed to be very high and a function of the test arrangement rather than an accurate modelling of the support conditions.

Chui (1986a) demonstrated that the vibration performance of timber floors was improved if they were supported on four sides instead of two. Chui (1986a) reported that when the floors were supported on four sides the modal frequencies had greater separation and the damping increased.

Ohlsson (1982) compared the damping ratios of floors tested in situ and in the laboratory. Equivalent viscous damping ratios were on average 0.9% for laboratory floors and 3.4% for in situ floor and consequently laboratory floors did not adequately replicate those tested in situ. Nonetheless, testing of laboratory floors can offer a conservative lower bound for similar in situ floors.
2.2.2. Influence of Floor Mass

Weckendorf et al. (2008) considered how the imposed mass on a timber floor affects the floor’s damping characteristics. An increase in dead weight was achieved by equally distributing sand bags on the surface of the floor. The imposed mass was increased from 17kg/m² to 72kg/m² in two increments. With the maximum imposed mass, the first three modal frequencies reduced by 45.1%, 52.2% and 54.6% respectively and the damping ratio for the first two modal frequencies reduced by 1.67% and 1.35% respectively whilst mode three increased by 0.45%. In contrast Chui (1986b) reported tests where a line load of 100kg was placed over the two unsupported edges of the floor to simulate the presence of furniture. It was found that the natural frequency dropped by 17% but the damping ratio increased by 48% for the first five modes. As the damping ratio of a system is equal to $c/c_r$, where $c_r$ is the critical damping of the floor and proportional to $1/\sqrt{km}$, the increase in damping observed by Chui is most likely to be a function of the changing edge restraint stiffness rather than the change in mass. Upgrading a timber floor with a thin topping will add significant mass (in the region of a 200% increase). This in conjunction with the change in stiffness should have a noticeable effect on the damping behaviour of the floor.

2.2.3. Influence of Stiffness Perpendicular to Joists

Chui (1986b) and Bainbridge & Mettem (1997) found strutting had little effect on the natural frequency of timber floors but that it did contribute to separating the higher modes. It was remarked that the modes of vibration are generally closer together for floors constructed from timber than other materials. This is due to the orthotropic nature of timber which reduces the transverse stiffness of the floor. Chui advises that the modes should be separated as far as possible to avoid superposition which could lead to higher accelerations. Bainbridge & Mettem (1997) noted that the EN1995-1-1 equation to predict the natural frequency of timber floors provides unnecessarily conservative values for UK timber floors. They suggested that this was because the behaviour of timber floors is somewhere between that of a beam (as modelled) and a plate. To overcome this they suggested an alternative equation which increases the natural frequency by 50% (Equation 2.4). However it can be observed that this is only an alternative to Equation 2.1 proposed by Leissa (1969) and simplified by Ohlsson (1988), which is capable of accounting for the increase in transverse stiffness.

$$f_1 = \frac{3\pi}{4l^2} \sqrt{\frac{(EI)l}{m}}$$  \hspace{1cm} (2.4)
Bainbridge & Mettem (1997) also described a test programme which investigated acceptable deflection levels and vibration responses of timber floors. The results of ceiling tests indicate that the interaction of plasterboard and timber joists to form an inverted T-beam is significant compared to the theoretical stiffness of bare joists. Only a small overall increase was found in the stiffness of the floor when the decking material was varied due to the fact that each decking material’s ratio of mass to stiffness was similar. In all the tests the damping of the floors was greater than the 1% allowed for in EN1995-1-1. In contrast Chui (1986b) reported that plywood provided better performance than other decking materials; chipboard and softwood floorboards. Whilst the floors with chipboard and floorboard decking had similar performance characteristics, the floor with plywood decking had greater transverse stiffness and a natural frequency 25% higher than the floor with chipboard. These results suggest that chipboard is a suitable material to replicate softwood floor boards when manufacturing laboratory test floors that replicate existing floors.

2.2.4. Influence of Human Occupancy

Chui (1986b) also considered the influence of the presence of a human body by exciting floors using heel drop tests and comparing with impact hammer tests. The results showed that the presence of a human body greatly increases the damping of the system. As floor vibrations can only be perceived to be unacceptable if the occupant is present, acceptable vibrations which are judged to be just acceptable without the presence of an occupant will form a conservative upper bound limit. Whilst the heel drop test has gained popularity for its simplicity, it is inappropriate for comparative testing as the magnitude and frequency components of the force input vary between persons applying the force and are not measured.

2.2.5. EN1995-1-1 Design Criteria

To predict whether the vibration response of a timber floor will be perceptible, EN1995-1-1 provides the following design criteria for residential floors with a natural frequency above 8Hz.

\[
\frac{w}{F} \leq a \tag{2.5}
\]

\[
v \leq b(d-f^{-1}) \tag{2.6}
\]

where:
\( w \) is the maximum instantaneous deflection caused by a vertical concentrated point load, \( F \), applied at any point of the floor.

\( v \) is the unit impulse velocity response, the maximum initial velocity caused by an ideal unit impulse (1 Ns). Components above 40Hz are disregarded.

\( \zeta \) is the modal damping ratio, assumed to be 0.02 for UK floors.

The limiting values of \( a \) and \( b \) are given in the UK National annex (BSI, 2006). For rectangular floors simply supported on all four sides the natural frequency, \( f_1 \), is given by:

\[
 f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}} \tag{2.7}
\]

where:

- \( l \) is the span of the floor
- \( m \) is the mass per unit area.
- \((EI)_l\) is the equivalent plate bending stiffness of the floor about an axis perpendicular to the direction of the joists.

The unit velocity impulse response may be approximated from:

\[
 v = \frac{4 (0.4 + 0.6n_{40})}{m \cdot b \cdot l + 200} \tag{2.8}
\]

where:

- \( n_{40} \) is the number of first order modes with modal frequencies below 40Hz.

The number of first order modes below 40Hz may be calculated from:

\[
 n_{40} = \left\{ \left( \frac{40}{f_1} \right)^2 - 1 \right\} \left( \frac{b}{T} \right)^4 \left( \frac{(EI)_l}{(EI)_b} \right)^{0.25} \tag{2.9}
\]

where:

- \((EI)_b\) is the equivalent plate bending stiffness of the floor about an axis parallel to the direction of the joists.

### 2.3. Methods to Improve Timber Floor Vibration Problems

Besides topping upgrades, alternative remedial measures exist for improving the vibration performance of problematic floors. The following measures will each be examined: relocation of activities, isolation of the excitation source, stiffening of the floor and increasing the damping.
2.3.1. Relocation of Activities

In some cases, it may be possible to relocate the activities which cause vibration problems to other parts of the floor or building (Allen & Pernica, 1998). Activities might include the operation of rotating machinery or aerobic activity. However, relocation may not be possible or it may be that a building or floor is to be converted for an alternative use and new vibration problems are expected.

2.3.2. Isolation of Excitation Source

When rotating machinery is the excitation source, the remedial action is often to isolate the machinery from the structure (Allen & Pernica, 1998). By placing the machinery on rubber mounts or springs, the dynamic input into the floor is altered leading to the interaction between the input and response to decrease. Whilst this method is appropriate for rotating machinery, it is not possible to isolate human excitation (Glisovic & Bosko, 2010) including aerobic activities and footfall, which for timber floors, are sufficient to cause a perceptible vibration response.

2.3.3. Increasing Floor Stiffness

Increasing the stiffness of the floor is a popular method of improving timber floor vibration performance (Chui, 1986a). As timber floors are orthotropic in their distribution of floor stiffness, the higher order modes of vibration tend to be closely spaced leading to a higher amplitude response. Methods of remediation usually focus on improving the stiffness of the floor perpendicular to the joists. Herringbone strutting and blocking, Figure 2.1, are popular methods of upgrade which usually don’t increase the natural frequency but do separate adjacent modes (Chui, 1986b; Bainbridge & Mettem, 1997; Weckendorf, 2009). According to Chui (1986a) this method of floor stiffening is difficult to achieve in practice because cutting the struts and blocks with sufficient precision to ensure a tight fit is not easy. Without a good fit, the effectiveness of the method is reduced.

An alternative to strutting and blocking is to glue and screw a plywood deck onto the existing floor to provide additional stiffness. Plywood has good stiffness perpendicular to the joists compared to other materials (Chui, 1986b) but, to utilise the stiffness to best effect, the plywood deck should be glued to the existing structure, undesirable as the intervention is irreversible. Weckendorf (2009) considered stiffening timber floors at sensitive locations
Figure 2.1.: Strutting and Blocking

(anti-nodes of modal frequencies) by doubling joists. The method was effective for reducing the amplitude of the first mode and for stiffening the edges of the floor when only supported on two sides. Whilst possible as a technique for new floors, it is less practical to apply to existing floors.

2.3.4. Increasing Floor Damping

The final option is to increase the damping of the floor. Additional damping is typically provided by a visco-elastic layer (Bernard, 2008; Ohlsson, 1982) or non-structural elements such as partitions and furniture. Dissipating the excitation energy shortens the duration of a transient response but has less effect on the initial acceleration of the floor, as there is insufficient time for the decay to take place.

2.4. Vibration of Timber-Concrete Composite Floors

Research focusing on the vibration of TCC floors has recently been described as “almost absent” from the literature (Yeoh, 2010). In a recent review article on TCC structures (Yeoh et al., 2011b) there was only brief reference made to the vibration of TCC structures. Whilst it is true that there has been less effort put into vibration studies in comparison to connector characterisation or beam flexural testing, there are a number of studies which are of interest. Table 2.1 summarises the state-of-the-art for TCC vibration testing and the following section comments on the findings with greatest implications for the upgrade of existing floors.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Description</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hu et al. (1998)</td>
<td>Three timber floors (3.6m wide spanning 4.47m) topped with 38mm thick concrete. 241mm I-joists at 600mm c/c. One floor had double headed nails as shear connectors. Laboratory tested. Vibration testing method: unknown.</td>
<td>Stiffness of all floors increased with the addition of the topping whilst natural frequency and r.m.s. acceleration decreased. The addition of the double headed nails was found to have no effect.</td>
</tr>
<tr>
<td>Taylor &amp; Hua (2000)</td>
<td>TJI floors with 19mm THK OSB overlaid with normal weight, lightweight and gypsum-based concrete. Laboratory tested. Vibration testing method: unknown.</td>
<td>Stiffness of all floors increased with the addition of the toppings, whilst the natural frequency decreased by between 48.3% and 21.0%. Floors topped with normal weight concrete exhibited the greatest reduction.</td>
</tr>
<tr>
<td>Toratti &amp; Kevarinmäki (2001)</td>
<td>A floor 2.4m x 6.0m constructed from nail plate trusses topped with between 60 and 80mm of concrete. Shear connectors: nail plates sandwiched between two trusses. Laboratory tested. Vibration testing method: walking test.</td>
<td>The natural frequency of the floor was 9.7Hz. Authors commented: that the floor was light and stiff enough to be a lightweight floor, and that performance would probably be unsatisfactory if the natural frequency was below 8Hz.</td>
</tr>
</tbody>
</table>
Table 2.1.: Summary of previous TCC vibration testing

<table>
<thead>
<tr>
<th>Study</th>
<th>Description</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mertens et al. (2007)</td>
<td>Four floors, each 4.6m square, 240x68mm joists at 600mm c/c with 18mm OSB sheathing. Floors were covered with 40mm THK anhydrite or concrete toppings either with or without screw connectors. Laboratory Tested. Vibration testing method: impact.</td>
<td>Topping and connectors both increased the stiffness of the floors. Connectors increased the transverse stiffness of the floor, separating the modes of vibration. Floors with connectors had a natural frequency 28% higher than those without. Compared to the timber floor $\zeta$ was higher in all upgraded floors. Max. accel. was reduced below EN1995-1-1 requirements.</td>
</tr>
<tr>
<td>Ghafar et al. (2008)</td>
<td>A single 400x64mm 10m span LVL beam, 17mm plywood interlayer, 65mm THK concrete topping with notched connections, tested with and without topping. Tested with block and roller conditions. Laboratory Tested. Vibration testing method: forced vibration.</td>
<td>With the addition of the topping the natural frequency of the beam reduced by 6% and $\zeta$ reduced from 2.3% to 1.3%. However, in absolute terms, the composite beam dissipated about 4 times the energy of the LVL beam. The support conditions affected both the natural frequency and damping ratio.</td>
</tr>
<tr>
<td>Ghafar et al. (2010)</td>
<td>A multi- span LVL-concrete floor constructed from four 250 x 45mm LVL joists each spanning 2.9m, supported on joist hangers and fixed back to back. Laboratory Tested. Vibration testing method: forced vibration and impact hammer.</td>
<td>Wave propagation to outer spans was reported. The accelerations at the joist hanger supports relative to the mid-span was greater than expected. Reducing the number of spans did not significantly change the natural frequency or damping ratios of the beams.</td>
</tr>
</tbody>
</table>
Table 2.1.: Summary of previous TCC vibration testing

<table>
<thead>
<tr>
<th>Study</th>
<th>Description</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rijal et al. (2011)</td>
<td>Four 250x48mm LVL beams spanning 5.8m and topped with 75mm concrete. Various connectors including: SFS screws, notches and birdmouth notches with coach screws. Laboratory Tested. Vibration testing method: impact hammer.</td>
<td>The natural frequency of the beams was between 8.93 and 10.08 Hz. Damping ratios varied between 0.97 and 1.86, depending on the number, rather than type of connectors.</td>
</tr>
<tr>
<td>Fragiacomo &amp; Lukaszewska</td>
<td>Two prefabricated glulam-concrete panels consisting of three 90 x 2700 x 4800mm glulam joists and a 60 x 1600 x 4800mm concrete slab with alternative connection systems. Laboratory Tested. Vibration testing method: impact hammer.</td>
<td>Fundamental frequencies ranged between 18.5 Hz and 18.9Hz, whilst damping ratios were between 7.1% and 7.3% for the natural frequency. Concluded that the panels were well constructed from a dynamic perspective.</td>
</tr>
</tbody>
</table>

2.4.1. Frequency

Toratti & Keväinmäki (2001) suggested that both timber and TCC floors should be classified as high frequency floors as they are light and stiff enough to ensure that the natural frequency at which they vibrate is above 8Hz. This suggests that the existing provision in EN1995-1-1 for assessing timber floors may be easily adapted for TCC floors.

The natural frequency at which a timber floor vibrates is proportional to the square root of the bending stiffness of the floor divided by its mass per unit area.

\[ f_1 \propto \sqrt{\frac{k}{m}} \]  

Adding a topping to a timber floor increases its bending stiffness and mass, changing the natural frequency at which it vibrates. Researchers (Hu et al.
have found that the increased mass can outweigh the beneficial increase in stiffness resulting in a lowering of the natural frequency even for floors with near complete composite action (Ghafar et al. (2008)). Whilst it is correct to conclude that composite action is important to increasing the natural frequency of floors (Ghafar et al. (2008); Rijal et al. (2011)), it is clear that it is not a guarantee of good performance and other parameters should be considered.

2.4.2. Magnitude of Response

Mertens et al. (2007), unlike other authors (Fragiacomo & Lukaszewska, 2011; Ghafar et al., 2008; Toratti & Kevarinmäki, 2001), reported the peak acceleration of the floors when subjected to direct impact loading. Four floors were constructed, 4.6m square with 68 x 240mm joists at 600mc/c and 18mm OSB nailed sheathing, supported on 140mm concrete blockwork walls. Floor 2 had an additional 40mm thick anhydrite topping without connectors. Floors 3 and 4 had 40mm anhydrite and 40mm C25/30 concrete toppings respectively with connectors joining the timber joists and topping elements (a double row of screws spaced at 200mm c/c). A summary of the results is shown in Table 2.2 and illustrates the dramatic improvement in the performance of the floor. Indeed floor 4 exhibits an acceleration which is far lower than required, therefore, there seems to be scope for reducing the thickness of the topping, allowing higher peak accelerations.

Table 2.2.: Softwood joist-concrete composite floor vibration results (Mertens et al. (2007))

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Static Deflection</th>
<th>$f_1$</th>
<th>$\zeta$</th>
<th>$a_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm)</td>
<td>(Hz)</td>
<td>(%)</td>
<td>(mm/sec²/N)</td>
</tr>
<tr>
<td>1- Timber Floor (T.F.)</td>
<td>0.76</td>
<td>20.25</td>
<td>1.02</td>
<td>100</td>
</tr>
<tr>
<td>2- T.F. + anhydrite</td>
<td>0.26</td>
<td>16.50</td>
<td>1.96</td>
<td>50</td>
</tr>
<tr>
<td>3- T.F. + anhydrite + connectors</td>
<td>0.16</td>
<td>21.05</td>
<td>1.30</td>
<td>3.6</td>
</tr>
<tr>
<td>4- T.F. + concrete + connectors</td>
<td>0.14</td>
<td>20.00</td>
<td>1.20</td>
<td>0.00115</td>
</tr>
</tbody>
</table>

2.4.3. Damping

Damping, an important consideration in the vibration response of floors, is not well understood for timber floors and less so for timber-concrete floors. EN1995-1-1 Part 1 suggests designers allow 1% damping in residential
timber floors whilst the UK NA advises 2%. Ceccotti (1995) suggests that the damping ratio of TCC floors is closer to 2%, compared to timber floors which have damping ratios nearer to 1%. Measured damping ratios for TCC floors and beams have ranged from 1.0% to 7.5% in testing, depending on the type and spacing of connectors and whether prefabricated or cast in situ concrete slabs were assessed (Ghafar et al., 2008; Rijal et al., 2011; Fragiacomo & Lukaszewska, 2011). Results from Rijal et al. (2011) suggest that the spacing of the connectors rather than the connector type has the greatest effect on the damping of a TCC beam. A beam with two thirds of the connections of another had a damping ratio 85% higher. Fewer connections permits greater interfacial slip between the timber and topping and greater slip between surfaces allows for greater energy dissipation by friction. However the results refer only to the damping ratio and not the absolute amount of energy dissipated by each beam. As the damping ratio is a ratio of energy dissipated to critical damping, it is not clear from this report whether the number of connections is reducing the critical damping or allowing greater frictional energy dissipation and which is more crucial to the change in damping ratio.

2.4.4. Limitations of Panel and Laboratory Testing

Timber floors behave in an orthotropic manner (Chui, 1986a) but with a concrete topping the stiffness perpendicular to the direction of the joists is increased and the behaviour is more akin to a ribbed plate. Compared to a timber floor the higher modes of vibration have greater separation which reduces the perceptibility of the vibration response (Mertens et al. (2007)). Table 2.1 shows that most vibration testing has been conducted on panels rather than floors (a panel is a section of a floor usually comprising of between one and three joists). This is because vibration testing of panels is an economical and simple extension of an existing programme of static flexural testing. Whilst vibration testing of panels can provide useful comparative information it does not represent the vibration behaviour of complete floors. For example the change in isotropy reported by Mertens et al. (2007) cannot be found experimentally by testing panels. In short, there is no substitute for the testing of complete floors. A further shortcoming of existing testing is the lack of in situ testing. Testing of timber floors has shown that the performance of in situ floors is different to laboratory floors (Chui et al., 2004).
2.5. Human Perception of Vibrations

Standards such as, ISO 2631-2:1989 provide criteria for relating the physical vibration response of a structure to the human perception of the same vibration response. In situ vibration testing with occupants is too complex to replicate in the laboratory and therefore tests are usually done in situ (Ohlsson, 1982; Johnson, 1994). Instead of measuring the human perception of upgraded floors, the intention is to compare the measured responses before and after upgrade. Therefore the literature discussing the human perception of vibration will not be reviewed here and instead a summary will be given of the points made by Ohlsson (1988) in the design guide “Springiness and Human-Induced Floor Vibrations”, from which EN1995-1-1 (1995) has formed the clauses described in section 2.2.5.

Ohlsson stated that there were three main factors affecting the perception of vibrations:

- Duration - Vibrations which are quickly damped are less perceptible compared to those which have a longer duration.
- Activity - If an occupant is resting, then vibrations are more perceptible than for an occupant who is walking around.
- Motivation and proximity - If the occupant is observing the source of the vibration, then they are more tolerant than if they are in an adjacent room and unaware of the source.

As it is the intention of this research project to upgrade existing floors, the following considerations should be highlighted:

- Converting existing buildings into multiple apartments will decrease tolerance levels for vibration in that building.
- Damping is a significant factor for the perception of vibration but damping in existing timber floors is not well understood and, as is explained in section 2.2, is variable. New damping mechanisms from an upgrade, principally the shear connectors, should be considered as they may offer more reliable sources of energy dissipation.

2.6. Composite Stiffness and Analysis

In this section the methods for analysing and predicting the stiffness of composite beams are examined. As composite beams consist of two or more elements joined together with shear connectors, the effective bending stiffness and strains and stresses within the section are dependent on the
effectiveness of the connection. The more effective the shear connection the greater the composite action between the elements. The behaviour has two boundaries (Yam, 1980):

- An upper bound where the connection is fully rigid and no interfacial slip occurs between the timber and topping. For this case the transformed section method can be used to calculate the effective bending stiffness and the strains and stresses within the section.

- A lower bound where there is no shear connection and the timber and topping elements act independently of each other. As there is no shear connection the maximum interfacial slip occurs and there is no transfer of shear forces between the elements. The effective bending stiffness of the section can be found by summing the stiffness of the individual parts.

For most TCC beams there is a shear connection between the timber and topping which is not fully rigid but does prevent some interfacial slip. In this situation the slip allowed to take place induces an appreciable strain difference, $e$, at the interface of the timber and topping (Figure 2.2). In the calculation of the strain and stress distributions the horizontal forces in the section ($C$ and $T$) and the strain difference ($e$) have to be taken into account. This situation is referred to as partial composite action or partial interaction (Yam, 1980).

![Partial Interaction Strain Distribution](image)

Figure 2.2.: Partial Interaction Strain Distribution

### 2.6.1. Methods of Analysis

Partial interaction theory was first derived by Stüssi (1947). The theory allows for the interfacial slip of composite beams by providing a prediction of a beam’s central deflection, provided that the connectors are evenly spaced along the length of the beam. It is assumed that the curvature of both elements is the same and there is no separation of the elements. For brevity the complete solution for a simply supported beam loaded in three point
bending is not provided within the main section of this report instead refer to appendix B, which provides an explanation of the development and solution of the second order differential equation.

EN1995-1-1 Annex B sets out the so called ‘gamma method’ (γ-method); the exact solution for the differential equation of partial composite action for the case of a simply supported beam with a bending moment that varies sinusoidally. It is used repeatedly in the literature to predict the serviceability limit state and ultimate limit state deflection of TCC floor sections and has been found to have excellent agreement with experimental results (Ceccotti, 2002; Fragiacomo & Lukaszewska, 2011; Fragiacomo, 2012; Persaud & Symons, 2005; Yeoh, 2010). The difference between the effective bending stiffness calculated using the γ-method and the exact solution for a simply supported beam subjected to three point bending is very small (Persaud & Symons, 2005). When compared with a UDL, the correlation is greater than with the 3-point bending solution, as the bending moment profile of a simply supported beam with a uniformly distributed load is closer to the γ-method sinusoidal approximation.

These methods are popular as they are simple to apply and provide a high degree of accuracy but other analytical methods, outlined below, have been developed to include alternative end conditions, loading patterns and second order effects.

Girhammar & Gopu (1991) presented a model based on Euler-Bernoulli beam theory. The model allows beams and columns with partial composite action to be analysed when they are subjected to a combination of bending moment and axial load, including second-order, P-∆, effects. The method is most suited to columns where axial loading is prevalent. Later the procedure was generalised and extended (Girhammar & Pan (2007)), presenting the ordinary differential equations and general solutions for the deflection, internal forces, boundary equations and buckling length coefficients.

Effort has also been made to analyse the dynamic stiffness of composite beams subjected to free and forced vibrations (Girhammar et al. (2009)). The paper presented partial differential equations and general solutions for the deflection and internal forces of Euler-Bernoulli beams with interlayer slip subjected to free and forced flexural vibrations. The work based on the Euler-Bernoulli approach sought to overcome the inherent problems in the theory; namely that it ignores the effect of shear deformations and rotary inertia which leads to the overestimation of vibration frequencies. Excellent agreement was found between the model and experimental results with respect of the calculation to initial peak velocity of partially composite beams subjected to step loading. However the authors concluded that the effects of
shear deformation and rotary inertia are negligible (about 3% error) if: the beam length:beam depth ratio is greater than ten, and the mode number is less than three.

### 2.7. Connector Behaviour

The connectors in a TCC transfer shear forces between the two layers as they undergo bending, thereby reducing the slip that otherwise occurs. The behaviour of the shear connector joining the timber floor to the topping upgrade has a large influence on the overall behaviour of the composite. It heavily influences the effective stiffness of the composite section and the manner in which it fails (Dias et al., 2010). Consequently there have been many research studies developing different connector types for both refurbishment applications and new timber-concrete composite floors. Different types of connector include: nails, screws, dowels, nail plates, notches, angle brackets, stud connectors and glued joints (Ahmadi & Saka, 1993; Clouston et al., 2004; Deam et al., 2007; Dias et al., 2007; Fragiacomomo et al., 2006; Frangi & Fontana, 2003; Lukaszewska et al., 2008; Van der Linden, 1999; Yeoh et al., 2011a). There has also been attention given to connectors for precast systems, which attempt to overcome the loss of effective stiffness attributable to shrinkage of the topping (Lukaszewska et al., 2008). An overview of the behaviour of existing connector types in conventional topping thicknesses is presented in Figure 2.3.

For a thin topping refurbishment application the ideal connector attributes are: good stiffness; minimal damage made to the existing floor structure; reversible intervention; minimal cost and a failure mechanism which imparts ductility to the composite structure; achieved either by formation of plastic hinges or progressive loss of composite action. Some existing connector types are less appropriate than others. Glued connections are the stiffest but fail in a brittle manner and are non-reversible, whilst nail plates are more ductile at failure but are less stiff and difficult to fit to an existing floor without removal of the floorboards. Notch connections are stiff but like glued joints, fail in a brittle manner. This can be overcome by inserting a dowel into the notched area to provide post failure strength. Dowel and screw connections are also appropriate, they fail in a ductile manner, are economical to install and cause minimal damage to the existing fabric of the floor. Of the existing connector types, the dowel and notch connectors seem to have the most potential with thin toppings, their behaviour with thick toppings will be discussed in sections 2.7.1 and 2.7.2.
2.7.1. Screw and Dowel Connectors

Screw and dowel connectors consist of a metal dowel inserted into the timber joist with the head side exposed then covered with concrete. The dowels can be nails, screws, plain or threaded bar and are either hammered, screwed or glued into the timber. Generally they are cheap and inexpensive to install but they are less stiff than other connector types and consequently a greater number are required to achieve a satisfactory degree of composite action.

Whilst previous testing of TCC joints has often focused on the testing of new connector types for specific applications (Ahmadi & Saka, 1993; Deam et al., 2007; Lukaszewska et al., 2008), more recently there has been interest in identifying factors which affect the stiffness and strength of dowel type connections. Dias et al. (2010) reported that dowel joints constructed from chestnut were 20% stiffer than those made from maritime pine and 50% stiffer than spruce. There was no relationship between the compressive strength of the concrete and the slip modulus of the connection, and the shape and yield strength of the dowel were found to have no influence.

Studies by van Van der Linden (1999), Dias et al. (2010), Jorge et al. (2011) and Jackulikova (2013) have all demonstrated that the inclusion of an interlayer (otherwise known as a floorboard) decreases the slip stiffness.
of a TCC joint. Van Der Linden reported a decrease of 50% with a 28mm interlayer, Dias et al. reported a decrease of 35% with a 20mm interlayer, Jorge et al. reported a decrease of 25-34% with a 25mm interlayer and Jackulikova reported decreases of 68.3% and 78.5% with 18mm and 36mm thick interlayers respectively. Ceccotti (2002) suggests serviceability moduli of $0.75 K_{ser}$, $0.66 K_{ser}$ and $0.5 K_{ser}$ for $t/d$ of 2, 3 and 4, respectively (where $t$ is the depth of the interlayer and $d$ is the effective diameter of the dowel). Gelfi et al. (2002) presented a beam on elastic foundation model which presumed that the interlayer has zero stiffness. However Jackulikova (2013) showed that the stiffness of the interlayer affects the stiffness of the joint. Dowel joints with 18mm thick plywood interlayers were 110% stiffer than dowel joints with 18mm thick particleboard interlayers.

Screw and dowel connectors are often inserted perpendicular to the joist. In this scenario the connector primarily resists the shear force through bending of the dowel leading to a low slip modulus. An alternative solution is to install inclined screws, allowing the screw to be loaded axially as well as in bending, resulting in a stiffer connection. Experimental tests have tended to focus on screws inclined at 45° (Frangi & Fontana, 2003; Steinberg et al., 2003) but Persaud et al. (2010) have suggested a model for estimating the stiffness of screws inclined at any angle. The model was based on the popular beam on elastic foundation theory and showed reasonable correlation with the trend in experimental results but tended to overestimate the magnitude of the joint stiffness by approximately 20%. Persaud et al. (2010) attributed this error to the incorrect assumption that there was no deformation occurring in the concrete layer.

The penetration depth of a dowel is the distance that it is embedded into the timber or concrete element under consideration. It has been shown that the penetration depth into the timber joist and concrete affect the stiffness of a dowel timber-concrete joint. Ahmadi & Saka (1993) tested small dowel connectors and reported that a timber penetration depth of 11$d$ was sufficient. Gelfi et al. (2002) extended the beam on elastic foundation model to define the minimum length that a connector should be embedded in each material. It was concluded that a minimum embedment depth of 5$d$ in timber and 3$d$ in concrete is sufficient to achieve the maximum shear resistance and 90% of the maximum stiffness. This implies that fasteners embedded in thin toppings are likely to be less stiff than would be seen with thicker toppings. In contrast to both these results EN1995-1-1 stipulates that the pointside penetration of a nail should be greater or equal to 8$d$. 

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2.7.2. Notches and Plugs

Like dowels, notches and plugs are popular connector types. They are formed by either routing and chiselling a rectangular section from the timber or drilling a circular hole out of the timber. Circular holes are called plugs whereas rectangular cuts across the whole breadth of the timber section are called notches. The area of removed timber is then allowed to fill with concrete when the slab is poured creating a bearing area against which slip is resisted. The connections tend to be very stiff, reported slip moduli are between 80 and 300kN/mm depending on the dimensions and shape of the notch and the bearing stiffnesses of the timber and concrete (Deam et al., 2007; Yeoh et al., 2011a). Failure occurs when the concrete shears across the notch at the timber-concrete interface. It is sudden and brittle but has been mitigated by including a dowel to provide post failure strength.

Notches and plugs are generally considered to be expensive to install due to their time intensive production. However far fewer are required compared to dowel type connectors because of their superior stiffness and strength. In comparison to dowel connectors, notches provide a solution which is less reversible and causes more damage to the existing fabric. It has been reported that segregation of the concrete within the notch led to a reduction in stiffness (Yeoh et al. 2011a) and this would likely be a heightened problem for smaller diameter plug connectors which would be necessary for narrower joists.

2.8. Small Amplitude Cyclic Testing Of Joints

Within joints there are two sources of damping, the damping of the materials and frictional damping at the interfaces (Chou & Polensek, 1987). Damping of a system can be visualised by plotting a load-displacement or stress-strain plot. For example in Figure 2.4 the area within the ellipse of the load-displacement plot is the energy dissipated in one cycle \( e_d \). Thorby (2008) derived Equation 2.11 which relates the energy dissipated to the damping, \( c \), which in turn can be extended to express the dimensionless damping ratio in terms of the elliptical area and the total energy in the system (Equation 2.12).

\[
e_d = \pi c X^2 \omega \\
\zeta = \frac{e_d}{2\pi k X^2 \Omega}
\]

where:

\( \Omega = \omega/\omega_n \) is the relationship between the excitation frequency and the undamped natural frequency.
Polensek (1975) (cited by Chui & Smith (1989)) reported damping values for nailed timber-timber joints by “calculating directly the energy loss during through-zero cyclic loading of small joint assemblies.” It was reported that high viscous damping ratios of over 28% were obtained. These seem far from realistic or even plausible, although they are not expressed as the total energy dissipation capability of a timber floor, which would give a clearer indication of their reliability. For simulating joints in a floor the technique of through zero-cyclic loading is not the correct loading model as there will always be a minimum imposed load applied to the floor. Instead a non-zero cycle, which ensures the joint always remains in compression, would be more appropriate.

Chou & Polensek (1987) investigated the effect of drying of nailed timber-timber joints on the stiffness and damping of the joints. Specimens comprised of single nailed plywood-stud joints, which were loaded cyclically either side of zero. Specimens were subjected to five rounds of testing at 12%, 28%, 18%, 12% and 12% moisture content. The results were analysed using an analysis of variance and the level of significance was set at 1%. Chou & Polensek (1987) found that damping ratios and joint stiffness were smaller for joints which had been assembled at a high moisture content and tested dry than for those assembled and tested at the same moisture content. Higher loads resulted in lower slip moduli and higher damping ratios for joints without gaps between the plywood and the stud. For joints with a gap, the damping ratio decreased as there was no frictional loss between the plywood and the stud.
Yeh et al. (1971) identified damping sources in wood structures. They concluded that damping in wood structures is attributable to either the material or the joints and that the energy dissipation within the joints is 13 times greater than the material. The authors suggested that the damping within joints consisted of ‘frictional stick’ and the slip at the interface.

Yeh et al. (1971) constructed nailed, symmetrical plywood and hemlock pushout specimens and subjected each of them to 100 slow, quasi-static cycles with the specimen always in compression. It was found that the load-slip behaviour corresponded to a bi-linear relationship which was followed by hysteresis. Yeh et al. (1971) remarked that the load-slip behaviour did not change throughout the 100 cycles and that the connection was undamaged at the end of the test.

Chui (1986b) suggested using a “joist-to-flooring connection which allows an appreciable amount of relative movement to occur”, with the aim of increasing the damping of the system. In a TCC system the connectors undergo slip based on their stiffness and position along the beam which may lead to significant frictional damping in addition to that already available to the existing floor.

2.9. Influence of the Topping

2.9.1. Topping Tensile Strength

Van der Linden (1999) undertook a parametric study to investigate the influence of topping properties on the stiffness and strength of composite beams. He concluded that the tensile strength of the topping influences the ultimate strength of composite beams; if part of the topping is in the tensile zone of the composite. However once the topping is cracked, the tensile strength has little influence on the stiffness and strength of the composite beam, less than 3% and 1% respectively.

2.9.2. Topping Compressive Strength

Dias et al. (2007) tested dowel timber-concrete joints and varied the concrete strength. The authors concluded that the capacity of the joint increased with the strength; although the effect was most significant at lower strengths, as at higher strengths the concrete acted as a perfect clamp and no further increase was observed. Dias et al. (2010) tested dowel timber-concrete joints and varied the concrete strength. Dias et al. (2010) found low variation in
slip modulus between the different concrete density specimens and found no evidence of a relationship between the slip modulus of the connectors and the compressive strength or foundation modulus of the topping. For a composite beam, Van der Linden (1999) concluded, from a parametric study, that a 30% variation in topping compressive strength had negligible effect on the strength of the composite beam.

2.9.3. Topping Modulus of Elasticity

Van der Linden (1999) also considered how the Modulus of Elasticity (MOE) of the topping influenced the stiffness and strength of a composite beam. The MOE was found to have a greater influence than the compressive strength and tensile strength of the concrete. Furthermore, for a 30% variation in a parameter, the influence of the MOE on the stiffness and strength of the composite beams was greater than the influence of the connectors; less than 9% and 6% respectively for the MOE, versus less than 8% and 3% respectively for the connectors. Although, when considering the entire practical range of MOE and connector stiffness, the influence of the connectors, on the strength and stiffness of composite beams, is greater.

2.9.4. Topping Density

Low density concrete toppings can be used as an alternative to normal weight concrete when an existing structure is incapable of supporting the permanent load of the topping (Dias et al., 2007). Several studies (Dias et al., 2007, 2010; Fragiacomo, 2012; Jorge et al., 2010; Steinberg et al., 2003) have utilised lightweight concrete as an alternative to normal weight concrete in TCC structures, but only one of these studies (Jorge et al., 2010) has compared the performance of lightweight concrete with normal weight concrete. Jorge et al. (2010) tested timber-concrete joints and beams constructed from lightweight concrete under long term loads. The authors concluded that the creep behaviour of TCC structures constructed from lightweight concrete was superior to those formed from normal weight concrete.

2.9.5. Topping Reinforcement

Ceccotti (1995) suggests reinforcing the topping to avoid loss of stiffness due to cracking in the tensile zone. Van der Linden (1999) reported that the reinforcement provided only a small contribution to the stiffness and strength of a composite beam and its main role was to reduce shrinkage cracking.
Fibre reinforcement, an alternative to steel mesh, was tested in notched pushout specimens by Heiduschke & Kasal (2003) to improve the shear strength of the shear connection. Notches 50mm and 100mm in diameter were tested, and found to have a shear strength of 13.1 and 10.8 N/mm² respectively. As no comparison between fibre-reinforced and conventional reinforcement was made no information is available regarding their relative performance.

2.9.6. Topping Shrinkage

Shrinkage of the concrete topping reduces the flexural stiffness, in service conditions, of composite floors, as the slip caused by shrinkage of the topping is in the opposite direction to when it is loaded (Al-deen et al., 2011). To overcome these problems, development of a precast TCC solution has taken place, which includes: connector characterisation, panel short-term and long-term bending tests and numerical analysis (Lukaszewska et al., 2008, 2010; Fragiacomo & Lukaszewska, 2011).

2.9.7. Topping Thickness

Topping thickness varies between studies. Examples include: Van der Linden (1999) who tested panels with a 110mm thick topping, Fragiacomo (2012) who upgraded a floor with a 60mm topping, Clouston et al. (2005) who tested composite beams with a 120mm thick topping and Meierhofer (1993) who tested composite slabs with a 80mm thick topping. Van der Linden (1999) identified that the maximum increase in stiffness that could be attained between zero and full connection was 300%. The optimum is achieved when the bending stiffness of the timber and topping are equal, and is unique to each geometrical arrangement of joists and topping. He suggested that if there was too great a deviation from the optimum then either the timber or topping would dominate the performance of the composite structure. For example, a floor which comprises, 50mm wide, 200mm deep joists at 400mm centres with a modular ratio between timber and topping equal to three, the topping thickness required to achieve a balanced cross-section is approximately 40mm.

2.10. Acoustic Performance

MacKenzie & Waters-Fuller (2004) summarised the typical sources of domestic noise. The summary is reproduced in table 2.3 and provides a
classification for each source. There are two classifications: airborne and impact. Airborne sound emanates from within the room and impact sound from when the structure of the room is struck and vibrates.

Table 2.3.: Classification of typical sources of domestic noise (MacKenzie & Waters-Fuller (2004))

<table>
<thead>
<tr>
<th>Source</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>television/radio</td>
<td>airborne</td>
</tr>
<tr>
<td>amplified music and non amplified music</td>
<td>airborne</td>
</tr>
<tr>
<td>people speaking or shouting</td>
<td>airborne</td>
</tr>
<tr>
<td>doors / cupboards closing</td>
<td>impact</td>
</tr>
<tr>
<td>plugs being pushed into sockets</td>
<td>impact</td>
</tr>
<tr>
<td>footfall noise</td>
<td>impact</td>
</tr>
<tr>
<td>furniture being moved (such as chairs)</td>
<td>impact</td>
</tr>
<tr>
<td>vacuum cleaners or washing machines</td>
<td>airborne and impact</td>
</tr>
</tbody>
</table>

Langdon et al. (1981) surveyed the opinions of occupants and compared their opinions with the sound transmission of the occupant’s dwellings. They concluded that when sound insulation is of a higher quality than average (and hence airborne sound transmission is low) the contribution of impact noises is far more significant. MacKenzie & Waters-Fuller (2004) summarised the mechanisms by which footfall induced sound is transmitted. The mechanisms are illustrated in Figure 2.5.

- Direct transmission is the mechanism by which the resonant motion of the floor radiates sound to the room below.
- Flanking transmission takes place when the resonant motion of the floor transmits sound to the adjacent walls, which then radiates into the room below.
- Forced transmission of low frequency noise is caused by forced motion of the floor causing compression of the air in the room below which changes the pressure outside of the eardrum. Transmission by this method is strongly dependent on the stiffness of the floor structure and hence it is a bigger problem for timber floors than those of concrete or steel. MacKenzie & Waters-Fuller (2004) explained that unlike concrete floors, timber floors do not have the...

'...mass, stiffness and impedance properties at low frequencies... As a result forced motion sound transmission is still highly dominant with a carpet finish.'

(Schmid, 2008) highlighted the lack of in situ acoustic measurements of TCC floors. To combat this (Schmid, 2008) tested eight floors and presented
numerical correlations for airborne and impact sound insulation. It was concluded that predictions could be made with a tolerance of ±3dB and that the build quality of the floor had a substantial influence on the acoustic performance of the floor.

For TCC structures the connectors flanking transmission may be enhanced by the shear connectors providing an additional pathway between concrete and timber. However a topping will stiffen the structure, which will reduce forced sound transmission. Thin toppings will have less effect on reducing airborne sound transmission than thicker toppings, as airborne sound transmission through a floor construction is primarily based on its mass per unit area (Schmid, 2008). The mass law suggests that for every doubling of mass there is a 6dB reduction in airborne sound transmission. Therefore an 80mm thick topping would have an airborne sound insulation 12dB higher than a 20mm topping.
2.11. Previous Thin Structural Topping Studies

2.11.1. Push out tests

TCC connections are typically constructed with a topping thickness of greater than 40mm. The effect of the topping thickness on the behaviour of the connectors is not usually considered.

Whitworth (2006) tested four specimen types with three repeats tested for each arrangement. A 20mm interlayer constructed from redwood floorboards separated the timber joist and concrete topping, a poly-ethylene sheet was used to prevent moisture ingress into the timber boards and 4.5mm diameter, 100mm long round wire nails were used throughout. Typical load-displacement plots for each specimen type are shown in Figure 2.6 with a description of each specimen below the figure. It was found that there was no appreciable difference in stiffness at 40% of the maximum load between the specimen types, the average serviceability slip modulus per connector was 0.4kN/mm.

Neve (2009) tested nail connectors with a 25mm thick topping and a 18mm particleboard interlayer. Symmetrical pushout specimens, each containing 16 connectors were loaded to failure at a constant displacement rate of 5mm/min. Whilst the nail connectors demonstrated a ductile failure mechanism, the slip modulus was low in comparison to other connector types found in the literature, ranging from 0.3kN/mm to 0.4kN/mm. A summary of the specimens is given in Table 2.4 and the load-slip displacement plots in Figure 2.7.

The main findings were:

- Specimen S1 corroborated the findings of Whitworth (2006),
- Inclining the screw connectors by 45° increased the stiffness of the connection by 111%.
- Reducing the embedment depth was shown to have little effect on the stiffness of the connection.

Neve concluded that the most desirable connectors were smooth round wire nails, as the difference in stiffness of the connectors did not lead to an appreciable difference in the stiffness of a composite beam and this connector had the most ductile failure mode. However, the reason why there was no appreciable difference in the stiffness of a composite beam using any of these connectors, was because the comparative stiffness between these, and other connectors in the literature was very low. In turn this meant that even in a long span floor, where composite action is easiest to achieve (Van der
Linden, 1999), the nail connectors would achieve minimal composite action. Therefore the perceived advantage of connector ductility, observed in the connectors at large displacements, is incorrect, as there is minimal composite action to be lost and large connector slip translates to beam curvatures large enough to cause failure of a timber beam before connector ductility is observed.

For S1 the predicted EN1995-1-1 $K_s$ value, allowing for the slight reduction recommended by Dias et al. (2010), is 1.5kN/mm which means the experimental stiffness of S1 is 72% lower than predicted. This reduction due to the presence of an interlayer is significantly higher than found by authors testing dowels in thick toppings (Dias et al., 2010; Jorge et al., 2011; Van der Linden, 1999) and is of particular concern as the floorboards are required as permanent formwork.
Table 2.4.: Specimen description (Neve (2009))

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Connector Type</th>
<th>Connector Orientation (°)</th>
<th>Embedment in Concrete (mm)</th>
<th>$K_{sc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>4.5mm x 100mm smooth round wire</td>
<td>90</td>
<td>20</td>
<td>413</td>
</tr>
<tr>
<td>S2</td>
<td>3.75mm x 75mm smooth round wire</td>
<td>90</td>
<td>20</td>
<td>295</td>
</tr>
<tr>
<td>S3</td>
<td>5.0mm x 100mm self tapping screw</td>
<td>90</td>
<td>20</td>
<td>306</td>
</tr>
<tr>
<td>S4</td>
<td>5.0mm x 80mm self tapping screw</td>
<td>90</td>
<td>20</td>
<td>309</td>
</tr>
<tr>
<td>S5</td>
<td>5.0mm x 100mm self tapping screw</td>
<td>±45</td>
<td>20</td>
<td>645</td>
</tr>
<tr>
<td>S6</td>
<td>5.0mm x 100mm annular ring shank nail</td>
<td>90</td>
<td>20</td>
<td>419</td>
</tr>
<tr>
<td>S7</td>
<td>3.75mm x 75mm annular ring shank nail</td>
<td>90</td>
<td>20</td>
<td>315</td>
</tr>
<tr>
<td>S8</td>
<td>3.75mm x 75mm annular ring shank nail</td>
<td>90</td>
<td>15</td>
<td>316</td>
</tr>
</tbody>
</table>

Figure 2.7.: Summary of connector behaviour (reproduced from Neve (2008))
2.11.2. Flexural Tests

Stearn (2006) and Gethin (2007) both tested 3m span thin TCC floor panels in three point bending. Each floor was constructed from three joists spaced at 400mm centres, with 20mm thick floorboards and a plastic sheet separating the topping from the floorboards. The floors were loaded at mid-span through a spreader beam until one of the joists failed in combined tension and bending at the location of a prominent knot within the middle third of the span. Figure 2.8 presents the load-mid-span displacement at the centre of the span whilst Table 2.5 provides a description of each specimen.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Number of Connectors</th>
<th>Topping Depth</th>
<th>Concrete Grade (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td>- - -</td>
<td>- - -</td>
<td>- - -</td>
</tr>
<tr>
<td>C1</td>
<td>48 nails per joist</td>
<td>25</td>
<td>60</td>
</tr>
<tr>
<td>C2</td>
<td>24 nails per joist</td>
<td>25</td>
<td>60</td>
</tr>
<tr>
<td>C3</td>
<td>48 nails per joist</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>P1</td>
<td>48 nails per joist</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>P2</td>
<td>96 nails per joist</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>P3</td>
<td>48 nails per joist</td>
<td>25</td>
<td>30 with Febond SBR</td>
</tr>
</tbody>
</table>

All of the panels behaved linearly to failure, as the curvature of the specimens was insufficient to allow ductile behaviour in the connectors to occur. The authors found that the displacements predicted by the γ-method were in good agreement with the experimental results, as the behaviour remained linear throughout the test. From Figure 2.8 it is clear that only a small amount of composite action was achieved. Indeed the γ-method predicts that a shear bond coefficient of only 0.085 was achieved for the floor sections with 48 connectors per joist. This is in part due to the short span (a 6m span floor would have achieved a shear bond coefficient of 0.271 with the same connector spacing) but mainly due to the low slip stiffness of the connectors.

The large number of shear connectors required to achieve a small degree of composite action suggests that the cost benefit of the quick installation of the nail connectors is outweighed by the number required. A stiffer shear connection is required to better exploit the potential of the topping upgrade.
Figure 2.8.: Summary of Flexural Tests (Stearn (2006) and Gethin (2007))
2.12. Concluding Comments

A broad range of literature has been reviewed and the main findings are summarised below.

- The $\gamma$-method provided by EN1995-1-1 has been shown to be accurate for predicting static deflections of TCC panels, whilst the load-slip behaviour of the connectors remains linear. It has also been shown to be reliable for timber-concrete panels with thin toppings.

- A method for predicting the vibration of orthotropic plates has been identified. As there has been limited experimental investigation of the response of TCC floors, a comparison is yet to be made between this theoretical method of predicting modal frequencies of vibration and experimental results.

- Many research studies have comprised of pushout tests to characterise the load-slip behaviour of a variety of connectors. Dowel and plug connectors have both been well investigated with thick toppings, but have had, at best, limited characterisation with thin toppings. The previous thin topping studies have found dowel connector stiffnesses to be very low and whilst this could be partially attributed to the presence of an interlayer, the depth of topping may also be a contributing factor. Nonetheless both connector types appear to be appropriate for this new application but require further testing. In addition to the topping thickness, it will be most important to identify the effect of the dowel inclination and plug diameter on the stiffness of thin topping TCC shear connectors.

- Previous thin structural topping studies have not only studied the load-slip behaviour of the connectors but also the flexural behaviour of floor sections. Low connector stiffness led to poor composite action and a brittle, tensile failure of the timber joists. With thin toppings, a stiffer connector could lead to a compression failure of the topping and therefore a new topping material with enhanced mechanical properties may be required.

- Studies have not yet considered upgrading timber floors with toppings to specifically improve vibration performance whilst previous methods of improving floor performance have included strutting, blocking and plywood decking. As yet there have been few TCC floor vibration studies and those which have been undertaken have often been comparative studies of panels. Complete TCC floor modal testing is required to gain further understanding in this research field.
The vibration response of a timber floor is dependent on the mass, stiffness and damping properties of the floor. All three will change with the upgrade of a timber floor and whilst the change in mass and stiffness can be calculated the change in damping is difficult to predict. Although supports may provide the greatest proportion of energy dissipation, it is new opportunities to dissipate energy which are of most interest, in particular the energy dissipation at a TCC shear connector. Chui (1986b) suggested that connectors in timber floors could be designed to allow greater energy dissipation by permitting greater slip to occur. Clearly if a TCC shear connector allowed greater slip the increase in floor stiffness would be compromised, nonetheless all shear connectors permit some slip, therefore dissipating energy. To evaluate the magnitude of energy dissipation an appropriate test method needs to be defined as previous small amplitude cyclic testing of timber-timber joints has produced unrealistic values.
3. | Topping Thickness

Conventional TCC upgrades utilise toppings that are 40mm thick or greater; these have four disadvantages. Thick toppings (40mm or above) add significant mass to the existing structure, which apart from unduly stressing the existing structure, can necessitate the use of propping to minimise creep deflections. Thick topping upgrades also reduce the floor to ceiling height which may be problematic for structures where this dimension is already small. When thicker toppings are used, greater alterations have to be made to existing doorways and the position of other building services. In this chapter consideration is given to how thick the topping upgrade should be to provide an appropriate improvement in floor stiffness and transient vibration performance. The chapter is divided into several sections: how the stiffness of a floor changes with varying topping thickness is considered in section 3.1; how the floor mass changes with varying topping thickness is considered in section 3.2; how the floor fundamental frequency changes with varying topping thickness is considered in section 3.3; the analytical model is compared with experimental testing in section 3.4; a sensitivity analysis is presented in section 3.5 and how a topping upgrade affects the two-way spanning action of the floor and the spread of higher modes of vibration is described in section 3.6.

3.1. Change in Stiffness

TCCs are usually designed so that the neutral axis of the fully composite section lies at the interface between the timber and concrete. This ensures that the concrete primarily resists combined compression and bending, and the timber resists combined tension and bending; consequently utilising the properties of the materials most appropriately. For traditional UK timber floors this typically results in a topping thickness of 40mm or greater (Figure 3.1). However this approach has two flaws:

- The approach ignores the magnitude of improvement in floor performance which is achieved by the upgrade. Improving the stiffness of a floor rather than its ultimate strength is the most important
consideration, as timber floors are most deficient in this regard. By upgrading with a thick topping the floor stiffness is increased many times over, far in excess of what is required, whereas thin topping upgrades can provide an improvement in floor stiffness that is proportionate to the deficiency (Figure 3.2).

- Whilst concrete has little tensile strength, timber performs well in compression relative to its tensile strength. Therefore TCCs can be designed so that the neutral axis of a fully composite section lies within the timber joist. This approach ensures that the concrete remains in compression and utilises the best properties of both materials.

An alternative method for defining an appropriate topping thickness for upgrading begins by first considering how the position of the neutral axis, in a fully composite section, changes as the topping thickness is increased (Figure 3.1). For a traditional timber floor joist, the depth of the neutral axis

![Figure 3.1: Change in Neutral Axis Position for a typical UK joist as concrete topping depth is increased, full composite action assumed C_p=24, h_t=200mm](image)

does not follow a linear relationship with topping thickness. As the topping thickness is initially increased, the neutral axis moves rapidly up through the section, before slowing and becoming almost linear thereafter. In turn this affects the rate at which the bending stiffness of the composite section increases as the topping depth is increased (Figure 3.2).
Van der Linden’s dimensionless parameter, $C_p$, (Van der Linden, 1999) describes the aspect ratio of a TCC panel. Typical traditional UK timber floors upgraded with a topping have a $C_p$ between 16 and 32, whereas a CLT-concrete composite floor has an approximate $C_p$ of 3.

$$C_p = \frac{E_c}{E_t} \cdot \frac{b_c}{b_t}$$

(3.1)

Where:

$E_c$ is the modulus of elasticity of the topping.

$E_t$ is the modulus of elasticity of the timber joists.

$b_c$ is the spacing of the timber joists.

$b_t$ is the breadth of the timber joists.

The increase in stiffness with topping depth, tends towards a linear relationship as $C_p$ approaches 1. As the floor becomes similar to a floor with discrete joists (i.e. $C_p$ increases), the initial increase in bending stiffness becomes more significant. By simplifying a floor to a series of panel elements (where the depth of the joists, $h_t$, is 200mm), it is seen that for a traditional timber floor the maximum increase in stiffness that could be achieved with a 20mm thick topping is between 150% ($C_p=16$) and 200% ($C_p=32$) whereas a 40mm
topping could achieve an increase in stiffness between 225% \((C_p=16)\) and 280% \((C_p=32)\).

### 3.2. Change in Mass

In contrast to the change in bending stiffness, the change in mass is always linear with topping thickness. The difference in mass between a traditional UK timber floor and a concrete topping is very large. Table 3.1 lists the mass of the components in a typical floor which is then compared in Figure 3.3 with the total mass of a timber floor with a topping upgrade 20mm thick and 50mm thick. The following conclusions can be summarised from the data:

Table 3.1.: Mass of a typical UK timber floor

<table>
<thead>
<tr>
<th>Description</th>
<th>Mass (kg/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18mm Thick Floorboards</td>
<td>7.0</td>
</tr>
<tr>
<td>200x50mm joists @400mm c/c</td>
<td>9.8</td>
</tr>
<tr>
<td>Plasterboard and skim</td>
<td>10.2</td>
</tr>
<tr>
<td>Total</td>
<td>27.0</td>
</tr>
</tbody>
</table>

Figure 3.3.: Change in Mass
• Timber floors are lightweight structures and consequently will respond to footfall impacts with relatively large amplitude, high frequency transient vibration.

• The mass of a topping, even if very thin, is significant and for practical topping thicknesses (greater than 20mm), is many times greater than that of the timber floor. Therefore if a timber floor is upgraded with a cementitious topping the amplitude of any footfall induced transient vibration will be appreciably reduced even with a thin topping.

• If only low interaction between the topping and timber floor can be achieved, then there is a possibility that, as the additional mass is very large, the fundamental frequency of the floor could be reduced to such an extent that the floor responds in a resonant manner to footfall impacts. This should be avoided as it poses a greater threat to the structural performance of the floor, and is most critical for long span floors which are more susceptible to a low frequency response, coinciding with the frequency of footfall.

• If only low interaction between the topping and timber floor can be achieved, then thicker toppings will lead to larger long-term creep deflections, due to the self-weight of the topping. In this regard, thinner toppings are preferable.

3.3. Change in Frequency

In section 3.2 it was concluded that the additional mass added to a floor from a topping upgrade should result in a significant reduction in the magnitude of the floor’s transient vibration response but that low composite action could result in a reduction in the fundamental frequency and lead to a resonant response. In addition, higher frequency vibrations are generally less perceptible to occupants (ISO, 1989) so it is beneficial to increase the modal frequencies of the transient vibration response. As the increase in floor stiffness with increasing topping depth is not linear, there is an optimal topping thickness for every floor.

The fundamental frequency of a panel is estimated from Equation 3.2:

\[ f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)}{m}} \]  

Where:

\[ m \] is the floor mass per unit area, in kg/m².
\( l \) is the floor span, in m.

\((EI)_l\) is the equivalent plate bending stiffness of the panel about an axis perpendicular to the direction of the joists, in N/m²/m.

The change in fundamental frequency of a panel upgraded with a topping is estimated from Equation 3.3:

\[
\Delta f_1 = \left( \sqrt{\frac{(EI)_{TCC} \cdot m_T}{(EI)_T \cdot m_{TCC}}} - 1 \right) \cdot 100\% \tag{3.3}
\]

Where:

- \( m_T \) is the mass per unit area, in kg/m², of the timber panel.
- \( m_{TCC} \) is the mass per unit area, in kg/m², of the upgraded panel.
- \((EI)_T\) is the equivalent plate bending stiffness, about an axis perpendicular to the direction of the joists, in N/m²/m, of the timber panel.
- \((EI)_{TCC}\) is the equivalent plate bending stiffness, about an axis perpendicular to the direction of the joists, in N/m²/m, of the upgraded panel.

In the case of the composite panel the equivalent plate bending stiffness was calculated according to the \(\gamma\)-method (Appendix D). As the inclusion of mass in the analysis precludes a non-dimensional analysis a specific case is described in Table 3.2. These floor properties in conjunction with the \(\gamma\)-method were used to plot Figure 3.4, the change in fundamental frequency against the topping depth, \(h_c\).

<table>
<thead>
<tr>
<th>( b_t ) (mm)</th>
<th>( h_t ) (mm)</th>
<th>( \rho_t ) (kg/m³)</th>
<th>( E_t ) (N/mm²)</th>
<th>( \gamma )</th>
<th>( h_c ) (mm)</th>
<th>( \rho_c ) (kg/m³)</th>
<th>( E_c ) (N/mm²)</th>
<th>( q_k ) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>200</td>
<td>400</td>
<td>10000</td>
<td>1.0</td>
<td>400</td>
<td>2300</td>
<td>30000</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Initially the rate at which \(\Delta f_1\) increases is large until it peaks at a topping depth of 12mm. The change in fundamental frequency between 12mm and 100mm always remains greater than 1 but is not as large for 12mm thick toppings. In this example the peak in improvement for topping thicknesses between 0mm and 100mm occurs at 12mm.
3.4. Experimental Comparison

To validate the analytical model two types of panels were constructed. Type 1 panels, A and B, were formed of a single joist spanning 2.8m and type 2 panels, C, D and E, were formed of two joists spanning 4.5m. Details of their construction and dimensions are recorded in Chapter 6 alongside their short-term bending performance.

Each panel underwent vibration testing before and after the topping was added. The panels were simply supported at their ends, whilst clamps were used to provide some torsional restraint. Preventing the torsional modes from occurring aided the analysis of the data. Each panel was subjected to excitation by a quick release method, where each panel was provided with an initial displacement from a mass hung from the underside. When the cable suspending the mass was cut, the panel rebounded, causing it to vibrate freely. The acceleration time response of the panels was measured by accelerometers mounted on the top of the specimens. The fundamental frequency of each panel was found by transforming the data from the time domain to the frequency domain, using the Fast Fourier Transform method.

Results for each panel are presented in Table 3.4 alongside predicted natural frequencies. Values were estimated using the $\gamma$-method with the stiffness of
the connectors estimated from the first cycle of loading \( (K_s) \) and the second cycle of loading \( (K_e) \) of pushout tests. These stiffness properties used in the analysis are reported in Chapter 4 and reproduced in Table 3.3. Note that the predicted change in natural frequencies presented here differ from Skinner et al. (2014). Unlike the previous analysis these results:

- take account of: the composite action between the particleboard and joists of the timber panels,
- include the effect of shear lag on the effective bending stiffness of the panels as discussed in Chapter 6,
- use the modulus of elasticity measured from cylinder tests in Chapter 6 instead of a modulus of elasticity estimated from EN1992-1-1 (2004b).

<table>
<thead>
<tr>
<th>Panel No.</th>
<th>( K_s ) (N/mm)</th>
<th>( K_e ) (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4150</td>
<td>7300</td>
</tr>
<tr>
<td>B</td>
<td>4150</td>
<td>7300</td>
</tr>
<tr>
<td>C</td>
<td>4150</td>
<td>7300</td>
</tr>
<tr>
<td>D</td>
<td>4335</td>
<td>7580</td>
</tr>
<tr>
<td>E</td>
<td>4150</td>
<td>7300</td>
</tr>
</tbody>
</table>

Table 3.4.: Comparison between predicted and experimental fundamental frequency

<table>
<thead>
<tr>
<th>Panel No.</th>
<th>Expt. ( f_1 ) (Hz)</th>
<th>Expt. TCC ( f_1 ) (Hz)</th>
<th>Expt. ( \Delta f_1 ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>16.9</td>
<td>16.0</td>
<td>-4.7</td>
</tr>
<tr>
<td>B</td>
<td>16.6</td>
<td>19.0</td>
<td>14.4</td>
</tr>
<tr>
<td>C</td>
<td>14.4</td>
<td>13.2</td>
<td>-8.5</td>
</tr>
<tr>
<td>D</td>
<td>15.3</td>
<td>13.2</td>
<td>-13.4</td>
</tr>
<tr>
<td>E</td>
<td>15.6</td>
<td>12.5</td>
<td>-20.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Panel No.</th>
<th>Expt. ( \Delta f_1 ) (%)</th>
<th>Predicted ( \Delta f_1 ) (Analyt. ( EI_{ef} ) from ( K_s ) (%)</th>
<th>Predicted ( \Delta f_1 ) (Analyt. ( EI_{ef} ) from ( K_e ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-4.7</td>
<td>-20.7</td>
<td>-11.1</td>
</tr>
<tr>
<td>B</td>
<td>14.4</td>
<td>-5.4</td>
<td>3.6</td>
</tr>
<tr>
<td>C</td>
<td>-8.5</td>
<td>-16.5</td>
<td>-10.7</td>
</tr>
<tr>
<td>D</td>
<td>-13.4</td>
<td>-16.7</td>
<td>-10.5</td>
</tr>
<tr>
<td>E</td>
<td>-20.0</td>
<td>-17.2</td>
<td>-11.3</td>
</tr>
</tbody>
</table>

As predicted the fundamental frequency of vibration decreased with the addition of the concrete topping for all but panel B but correlation between experimental and predicted results varied for each panel. For all panels,
expect panel E, the predicted value, estimated using $K_e$ was more accurate than the predicted value estimated from $K_s$. This is as expected as $K_e$ is the property of the connectors, which best describes their in-service stiffness. Predicted values that were estimated using $K_s$ differed from the experimental change in fundamental frequency by between 2.8% and 19.8% whereas the difference between values predicted using $K_e$ and the experimental change in frequency were between 2.2% and 10.8%. For all panels the predicted values estimated with $K_s$ were more conservative than those estimated with $K_e$.

The analysis, as will be seen in section 3.5, is most sensitive to the interaction achieved between the topping and timber. This may go some way to accounting for any discrepancy in result as poor installation of connectors in a short panel will have a larger affect than in a large panel. This is because interaction is easier to achieve in longer span panels (Van der Linden, 1999) as there is a larger length over which to transfer the interfacial forces. As floors with longer spans are most likely to have vibration problem than the shorter spans used in this test, the small discrepancy in results should not be of concern.

### 3.5. Sensitivity Analysis

The following section investigates the sensitivity of the analysis to the parameters listed in Table 3.2.

#### 3.5.1. Composite Action

In case one (Figure 3.5) the shear bond coefficients ($\gamma$) of 0.25, 0.5 and 1 were studied. The shear bond coefficient, a term from Annex B of EN1995-1-1, was used to describe the extent of interaction between timber and topping, with 0 signifying no interaction and 1 complete interaction. Of the five factors considered it is the most important. Not only does it have the largest effect on $\Delta f_1$ it also has the largest effect on the topping thickness at which this peak in performance occurs. The topping thickness at which the greatest increase in frequency was attained, reduced by 58% between complete interaction to a shear bond coefficient of 0.25, whilst the increase in frequency diminished by 88%.

#### 3.5.2. Joist Depth

Case two (Figure 3.6) was used to study the sensitivity of the analysis to joist depth. Greatest improvement in performance was found to be for joists with
least depth. The topping thickness at which the greatest increase in frequency was attained, increased by 11.5% from 200 mm to 300 mm deep joists, whilst the increase in frequency diminished by 26.2%.

### 3.5.3. Superimposed Dead Load

In case three (Figure 3.7) the superimposed dead load of the floor was considered. In the analysis, the superimposed dead imposed load was assumed to be the same before and after the topping was added. As with the previous case, three scenarios were considered; loads of 0.35, 0.20 and 0.05 kN/m². As the superimposed dead load on the floor increases, the effect of the mass of the topping is diminished as it becomes a smaller proportion of the total mass of the system. Therefore floors with large existing superimposed dead loads will show greater increase in frequency. Low loads also tended to narrow the peak in performance in comparison to the scenario with higher loads. Whilst it has now been shown that timber floors are lightweight, and thus the service load has a large effect on the natural frequency, the analysis presumed a uniformly distributed load which differs from actual floors, where the load is often unevenly distributed around the edges of the floor (Ohlsson, 1982). Application of load nearer the supports reduces the observed effect.
3.5.4. Modulus of Elasticity of Topping

In case four (Figure 3.8) the static topping Modulus of elasticity (MOE) was investigated. Three levels of the factor were considered; 20,000N/mm², 30,000N/mm² and 60,000N/mm². Topping with higher MOE led to a stiffer composite section, which in turn caused a greater increase in fundamental frequency. Topping with MOE of 60,000N/mm² narrowed the peak in performance and led to the peak occurring at thinner topping depths than for topping with a MOE of 20,000N/mm². The peak improvement in performance with a 60,000N/mm² topping was 90% and occurred at a topping thickness of 8mm, whereas a 20,000N/mm² topping had a peak frequency increase of 62% and occurred at 14.5mm.

The improvement in performance does not have a linear relationship with the topping MOE. The increase in change in frequency between 20,000N/mm² and 30,000N/mm² is greater than between 30,000N/mm² and 60,000N/mm² when considered as a proportion of the change in MOE.

3.5.5. Topping Density

In case five (Figure 3.9) topping densities of 1700kg/m³, 2000kg/m³, 2300kg/m³ and 2600kg/m³ were considered. Toppings that were least
dense performed best, with regards to increasing frequency, although the parameter has the least effect of all those studied. The difference between peak increase in frequency between 1700kg/m³ and 2600kg/m³ was only 12.8%. Compared to the difference in imposed loads considered in section 3.5.3, the additional permanent load when considering a 2600kg/m³ topping density rather than 1700kg/m³ is relatively small, only 0.18kN/m² at 20mm thick. Consequently the effects of the topping density were less accentuated, when the topping was thin, in comparison to the imposed load.

3.5.6. Summary

In all five cases the topping thickness at which the greatest increase in fundamental frequency was found, lies between 5 mm and 16.5 mm. Toppings of these thicknesses are less practical on site than conventional topping thicknesses and provide technical challenges, such as preventing excessive topping shrinkage. At currently achievable topping thicknesses of 20 - 100 mm, an appreciable increase in fundamental frequency is achievable. For example, for the cases presented, the difference in change in fundamental frequency between the greatest increase attainable and the increase with a 40 mm thick topping was less than 20% and often between 10% and 15%. However there is literature example describes the fundamental frequency
of a timber T-panel decreasing with the addition of a topping where there was a combination of unfavourable factors, despite high levels of interaction being achieved (Ghafar et al., 2008). This literature example illustrates that care should be taken when designing a concrete upgrade for a timber floor, as achieving good interaction between the existing floor and topping upgrade may not be sufficient to increase the fundamental frequency and all parameters need to be considered.

3.6. Two-Way Spanning Effects

The previous analysis has addressed the change in fundamental frequency of a T-panel. This approach has ignored how the orthotropic behaviour of timber floors can often result in closely spaced higher modes of vibration, which cause a phenomenon known as beats where adjacent modes coincide. This coincidence effect leads to a greater perception of the transient vibration response than would occur by solely allowing for the individual modes, consequently only considering the first mode of vibration is insufficient (Ohlsson, 1982, section 5.2 and 7.8).

For TCCs the behaviour tends towards that of a plate as the concrete topping becomes thicker, causing adjacent modes to separate and thus reducing their
interaction and unwanted coincidence effects. An approach to estimate the effect is to consider how the ratio of the transverse stiffness \((EI)_b\) to longitudinal stiffness \((EI)_l\), found in Equation 3.4, changes as the thickness of the topping increases, which in turn can be used to consider how the theoretical frequencies of higher order modes change. To simplify the analysis the timber floor was assumed to have negligible transverse stiffness and therefore behaved as a beam with one-dimensional stiffness.

Figure 3.10 confirms that the ratio of transverse to longitudinal stiffness does increase beneficially with the addition of a concrete topping. The increase is most prominent for composites with least interaction, as the analysis assumes that only the topping provides transverse stiffness and is therefore unaffected by the effects of composite action. However it is most likely that the perception of the complete transient vibration response would be lower for composites with complete interaction. Although the effects of including the transverse stiffness in the analysis of the modal frequencies would appear from Figure 3.10 to be small for thin toppings (less than 20mm), this is not the case. Equation 3.4, describing the frequency of a discrete mode, contains 2nd and 4th order terms, relating to the mode number \((j)\) being considered, which have greater significance as the mode number increases, in turn leading to increasing separation of the modes of vibration. With a square floor, 20mm thick topping and full composite action the second mode of vibration would
equal 1.05\(f_{1,1}\) whilst third and fourth modes would equal 1.22\(f_{1,1}\) and 1.57\(f_{1,1}\) respectively.

\[
f_{1,j} = \frac{\pi}{2\sqrt{2}} \sqrt{\frac{(EI)_f}{m}} \cdot \sqrt{1 + \left[ 2j^2 \left( \frac{l}{b} \right)^2 + j^4 \left( \frac{l}{b} \right)^4 \right] \cdot \frac{(EI)_b}{(EI)_f}}
\]

(3.4)

### 3.7. Concluding Comments

Timber floors and TCC floors have very different mass and stiffness properties, in particular the mass of a TCC floor is much greater than a timber floor, even when refurbishing with a thin topping. It is suggested that this has two consequences with regard to upgrading to improve transient vibration performance. First the magnitude of the transient vibration response will be significantly reduced, even if the topping is thin. Second the addition of the topping, in conjunction with poor composite action, could lead to the floor responding in a resonant manner, rather than a series of transient responses. To avoid this problem, the frequency of the first mode of vibration should not be reduced with the upgrade; depending on the properties of the existing floor this requires good composite action and the correct topping thickness.
A sensitivity analysis considered how the frequency of the first mode of vibration of a T-panel changed as the topping thickness was increased. It highlighted that the most important parameters to consider during an upgrade are composite action, topping MOE and the effect of existing superimposed dead load which contributes to the modal mass of the structure. The density of the topping and the depth of the joists were least important. The parameters not only affected the magnitude of the change in frequency of the first mode of vibration but also the topping thickness at which it occurred.

A topping upgrade increases the transverse stiffness of the floor, which in turn has a significant effect on the perception of the transient vibration response. The transverse stiffness has increasing effect as the mode number increases which in turn causes the modes of vibration to separate reducing the human perception of the overall response. This is in agreement with the findings reported by Mertens et al. (2007).

Thin toppings have been shown to be an appropriate approach to upgrading timber floors to reduce their serviceability deflections and improve their transient vibration response to footfall excitation. Whilst topping thicknesses of approximately 10mm are well suited to improving the frequency at which timber floors vibrate they are not sufficient for a meaningful reduction in serviceability deflection. Because existing timber floors are likely to be uneven, a 10mm thick topping would be difficult to apply without a large variance in thickness across the floor area. In addition, the surface area to volume ratio of the topping increases as the topping depth reduces, which in turn increases the likelihood of shrinkage cracking occurring. Therefore a topping thickness of 20mm not only achieves a greater increase in bending stiffness, but is more practical to apply.
4. | Static Pushout Tests

In this chapter the experimental programme characterising the behaviour of shear connectors for a thin topping application is presented. The properties of the connector influence the behaviour of the composite. Whilst some empirically based models exist for predicting the behaviour of simple dowel connections (Ceccotti, 1995; CEN, 2004a; Gelfi et al., 2002), behaviour of connectors is largely determined from experimental tests. Previous testing of joints has often focused on testing of new connectors types for specific applications (Ahmadi & Saka 1993; Deam et al. 2007; Lukaszewska et al. 2008), although more recently there has been some progress identifying factors affecting the stiffness and strength of dowel type connections (Dias et al. 2010).

The aim of the testing programme presented here was to identify potential connector types for use with thin toppings and to determine the factors which influence their behaviour. The testing was split into three phases. The first phase consisted of preliminary testing to establish appropriate connector types for use with thin toppings. The factors which influence the joint performance were considered in the second phase and the variation within groups of replicate specimens observed during the second phase was investigated in phase three. The experimental methodology, statistical analysis and mechanical properties of the pushout testing are reported.

4.1. Phase 1 Tests

4.1.1. Phase 1 Experimental Design

Many connector types have been tested with toppings of conventional thickness but not all are suitable for use with thin toppings as they require a minimum depth to cover the connector. The choice of connector types, for initial investigation, was narrowed by identifying the attributes that were most desirable for upgrading with a thin topping. Connectors that provide reasonable stiffness, are quick to install, cause minimal damage to the floor fabric and provide a good mechanical connection with a thin topping were...
considered the most appropriate. The test series which were chosen are listed in Table 4.1 and illustrated in Figure 4.1. The series were:

- inclined screws in compression (S12.5-C),
- inclined screws in tension (S12.5-T),
- inclined screws crossed (S12.5-X),
- vertical screws with a hole drilled in the interlayer (S12.5-0°+P),
- vertical screws without a hole drilled in the interlayer (S12.5-0°),
- plug connectors 25mm in diameter (P25-15),
- plug connectors 38mm in diameter (P38-15).

<table>
<thead>
<tr>
<th>Series</th>
<th>Topping depth (mm)</th>
<th>Plug dia (mm)</th>
<th>Plug depth (mm)</th>
<th>Interlayer thickness (mm)</th>
<th>Screw inclination (°)</th>
<th>Screw length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P25-15</td>
<td>12.5</td>
<td>25</td>
<td>33</td>
<td>18</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>P38-15</td>
<td>12.5</td>
<td>38</td>
<td>33</td>
<td>18</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>S12.5-0°</td>
<td>12.5</td>
<td>-</td>
<td>-</td>
<td>18</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td>S12.5-0°+P</td>
<td>12.5</td>
<td>25</td>
<td>18</td>
<td>18</td>
<td>-45</td>
<td>120</td>
</tr>
<tr>
<td>S12.5-C</td>
<td>12.5</td>
<td>-</td>
<td>-</td>
<td>18</td>
<td>+45</td>
<td>120</td>
</tr>
<tr>
<td>S12.5-T</td>
<td>12.5</td>
<td>-</td>
<td>-</td>
<td>18</td>
<td>±45</td>
<td>120</td>
</tr>
<tr>
<td>S12.5-X</td>
<td>12.5</td>
<td>-</td>
<td>-</td>
<td>18</td>
<td>±45</td>
<td>120</td>
</tr>
</tbody>
</table>

The specimens were constructed from kiln dried European Whitewood glulam blocks 300mm high, 225mm wide and 90mm deep. Prior to construction the glulam blocks were stored in 20/65 conditions (20°C and 65% humidity) to stabilise the moisture content of the timber. The moisture content, measured according to BS EN 13183-1:2002, and density of each
timber type are recorded in Table 4.4. An 18mm thick particleboard interlayer was separated from the topping by a plastic sheet. A proprietary, polymer modified, cementitious floor screed, Rotafix IFS, was used for the topping and HECO Topix, 6.0mm diameter self-drilling screws were used throughout to connect the timber to the topping (ETA-11/0284 (DIBt, 2011)). Each specimen was symmetrical in design, containing a total of four connectors. The symmetrical nature of the design ensured that eccentric loading was avoided. Transducers were arranged as illustrated in Figure 4.8 and measured the slip between timber and interlayer, timber and topping and in-plane rotation of the specimen. The mounting positions of the instruments were chosen to minimise the errors caused by elastic shortening of the specimen.

4.1.2. Loading Protocol

Currently there is no standard loading protocol for testing timber-concrete joints but researchers usually adopt the procedure laid out in BS EN 26891:1991 for determining the strength and deformation characteristics of timber-timber joints. The procedure consists of loading to 40% of the estimated maximum load, 0.4$F_{est}$, maintaining the load for 30 seconds, unloading to 0.1$F_{est}$, maintaining the load for a further 30 seconds before loading to failure. As joints are almost exclusively tested by this method, the stiffness of the joint after the second cycle of loading is not well understood, yet a slow cyclic load pattern better represents the in-service loading of a connection. It is important to know whether the stiffness of the connection deteriorates from cycles of loading. Therefore the loading protocol of BS EN 26891:19991 was modified to provide additional cycles of loading. The new protocol, Figure 4.2, has an additional three cycles of loading and unloading in addition to BS EN 26891:1991.

4.1.3. Results

The load-slip behaviour of each connector type is plotted in Figure 4.3 and the mean strength and stiffness is recorded in Table 4.2. Each plot represents the median behaviour for each connector.

From each test two parameters describing the stiffness of the connectors, $K_s$ and $K_e$ have been evaluated. The serviceability slip modulus, $K_s$, describes the elastic stiffness of a connector as it reaches a load on the first occasion in its load-time history. The elastic slip modulus, $K_e$, describes the elastic stiffness of the connector for subsequent loading cycles where the connector
Figure 4.2.: Amended loading protocol (where 0,1 etc. refer to the loading ramp number, percentage of estimated load)

Table 4.2.: Phase 1 pushout test results

<table>
<thead>
<tr>
<th>Series</th>
<th>$K_s$ (kN/mm)</th>
<th>CV (%)</th>
<th>$K_e$ (kN/mm)</th>
<th>CV (%)</th>
<th>$F_{max}$ (kN)</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P25-15</td>
<td>3.43</td>
<td>24.6</td>
<td>7.29</td>
<td>24.4</td>
<td>2.3</td>
<td>55.5</td>
</tr>
<tr>
<td>P38-15</td>
<td>6.54</td>
<td>32.1</td>
<td>10.65</td>
<td>15.9</td>
<td>5.0</td>
<td>7.9</td>
</tr>
<tr>
<td>S12.5-0°</td>
<td>0.32</td>
<td>18.9</td>
<td>1.47</td>
<td>21.3</td>
<td>2.4</td>
<td>32.8</td>
</tr>
<tr>
<td>S12.5-0°+P</td>
<td>1.40</td>
<td>21.9</td>
<td>2.69</td>
<td>6.8</td>
<td>3.0</td>
<td>11.7</td>
</tr>
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<td>0.78</td>
<td>30.3</td>
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<td>17.9</td>
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<tr>
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<td>6.86</td>
<td>17.7</td>
<td>2.1</td>
<td>20.4</td>
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<tr>
<td>S12.5-X</td>
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<td>5.71</td>
<td>19.6</td>
<td>0.85</td>
<td>13.7</td>
</tr>
</tbody>
</table>

* mean of 6 specimens.

has previously been subjected to higher loads in its load-time history. As the stiffness of the connectors did not change after the 2nd cycle of loading only results for the first two cycles are listed in Table 4.2. $K_s$ and $K_e$ are calculated from the experimental data by Equations 4.1 and 4.2 respectively.

\[
K_s = \frac{0.4F_{est}}{4 \left( v_{04} - v_{01} \right)} \tag{4.1}
\]

\[
K_e = \frac{0.4F_{est}}{4 \left( v_{24} - v_{21} \right)} \tag{4.2}
\]

where $v_{01}$, $v_{04}$, $v_{21}$ and $v_{24}$ are the relative displacements at the time points described by Figure 4.2 and $F_{est}$ is the estimated maximum load. All
Figure 4.3.: Typical load-slip plot of each connector type - full view and zoomed view
connector types displayed a higher elastic slip modulus than serviceability slip modulus. The improvement ranged from 36% for series S12.5-C, to 359% for series S12.5-0°. The plug type connectors and inclined screws acting in tension were the stiffest of the connector types. For plug connectors the direct bearing of the topping against timber, provides a stiff mechanism by which to transfer load. Inclining screws reduces the extent to which they act in bending. Angled, they carry a greater axial load which reduces their displacement per unit load.

Drilling a hole through the interlayer to create a concrete plug resulted in a stiffer connection. The deflection of the glulam relative to the topping and the deflection of the glulam relative to the interlayer were equal throughout the test. Whilst it might be concluded that the specimens of this series behaved in a similar manner to specimens which do not have an interlayer, the serviceability slip modulus was still (52%) less than would be predicted by the formula provided in EN1995-1-1 (Equation 4.3). This inaccuracy is worse than for Dias et al. (2010) who reported an overestimation of joint stiffness of 13% for dowelled joints, but similar to Branco et al. (2009) who reported an overestimation of 48% for nailed joints.

\[ K_s = 2 \left[ \frac{\rho_m^{1.5} d}{23} \right] \]  \hspace{1cm} (4.3)

Where:

- \( d \) is the diameter of the dowel,
- \( \rho_m \) is the mean density of the timber.

All of the connector types had low strengths compared to previous literature examples (Steinberg et al. 2003; Dias et al. 2007; Deam et al. 2007), but this was not surprising as the topping was much thinner than in these studies and failure took place within the topping for all connector types. Connectors in the S12.5-C series performed worst as the cover to the connectors was very small providing little material to bear against. Similar behaviour was also observed in the S12.5-X test series where two of the four screws in each specimen were acting in compression.

Extensive cracking of the topping was observed before 40% of the maximum load was achieved in specimens of test series S12.5-0°. Whilst the connection continued to deflect to 15mm, exhibiting good ductility, permanent damage to the topping had occurred before 40% of the maximum load and therefore failure was considered to have occurred before ductility was observed.

Test series P25-15, P38-15 and S12.5-0°+P all demonstrated identical failure mechanisms. For these connector types the concrete sheared following a
path over the top of the screw head (Figure 4.4). This was not the expected failure mechanism as the screw did not provide post-failure strength. Plug connections with thicker toppings, that provide greater cover to the dowel connectors, have been shown to fail at the interface between topping and timber (Deam et al. 2007). If the topping shears in this manner, then the dowel can provide some post-failure strength to the connection.

![Figure 4.4.: Plug connector shear failure](image)

Screws which acted in tension had a separate failure mode from the other test series. Figure 4.5 illustrates how the connectors pulled out from the topping at failure, shearing the material beneath the flange head from the remainder of the topping. However it is not the objective of the upgrade to improve the strength of the floor (see Chapter 3). Therefore the failure mechanism of the connector is of less consequence than for other TCC systems as the connectors could be designed to fail before the bending capacity of the timber joists is reached.

The screws in all of the connector types showed no sign of plastic deformation, as the connectors had not reached their yield moment, and the glulam provided no obvious indications of splitting or pullout of the connector. This would indicate that the screw strength is out of proportion to the topping depth, screws that are shorter in length would be more appropriate. From an economical point of view, the consequence of having low strength connections is that a greater number are required to sustain a service load.

### 4.1.4. Summary

The preliminary pushout tests have shown that notch type connectors have higher strength and stiffness than screw type connectors. However, because it is difficult to ensure the notch remains within the breadth of the joist when the installer’s view of the joists is obscured by the floorboards and the joists are narrow, they are less practical to install than screw connections. In comparison to screws, notches cause significantly more damage to the existing floor fabric and are less suited to a reversible upgrade application.
The performance of screw connections in thin toppings is variable and dependent on the inclination of the screw. Of the arrangements, screws acting in tension performed best in terms of stiffness but were relatively weak compared to screw connections with thick toppings reported in the literature. Failure was concentrated in the topping; the screw head pulling out of the topping. Withdrawal of the screw from the timber would be a more desirable failure mode and could be achieved by thickening the topping and shortening the pointside penetration of the screw.

**4.2. Phase 2 & 3 Tests**

Preliminary testing indicated that inclined screws acting in tension have the most promise as connectors for use with thin toppings. The following testing investigates the factors which affect the performance of these connectors.

**4.2.1. Experimental Methodology**

A factorial experimental design was used for phase 2 testing to assess how each factor affected the behaviour of the connector system. Contrary to considering single factors independently, a factorial design varies factors simultaneously thereby allowing fewer specimens to be tested and the effects of interaction between factors to be assessed alongside the main effects.
4.2.2. Phase 2 Experimental Design

Each factor was given two levels. The topping was either 12.5mm or 26.5mm thick, the density of the glulam was either 408kg/m³ or 544kg/m³ (mean values) and the inclination of the screws was either 35° or 55° from the horizontal. As there were three factors and two levels for each factor the number of groups/types of specimen was $2^3=8$. A pictorial representation of the experiment is given in Figure 4.6.

![Experimental design](image)

**Figure 4.6.: Experimental design**

The number of replicates within each group was chosen so as to have sufficient power to detect reasonable differences between groups. Of primary concern was the effect that the factors had on the serviceability slip modulus of the connector. It was estimated that the experimental design would have sufficient power to detect an effect size of $f=0.44$. Cohen's effect size, $f$, (Cohen, 1992) is a gauge of the difference between the means of two groups; the effect being the change in behaviour due to an applied factor. An effect size of 0.1 is considered small, 0.25 medium and 0.4 large. An estimate of the variance within groups was made from extensive literature results for dowelled connections (Dias, 2005) and the results reported from phase 1 tests, in section 4.1.3. This information was then used, in conjunction with power analysis software, G*Power 3.1 (Faul et al. 2009), to estimate that 6 replicates were required for each group; a total of 48 specimens.

4.2.3. Phase 3 Experimental Design

The second phase of testing highlighted that, for some specimen types, there was a large coefficient of variation within groups for the serviceability slip modulus. It was thought that the source of the variation might be due to variations in the bearing stiffness of the topping beneath the screw
head caused by poor compaction of the topping. To test this hypothesis a third experimental series was devised. A control group with 19.5mm thick topping and screws inclined at 35° were made and compared to groups with deliberate flaws. The flaws were made by fixing 15mm wide, 90mm long strips of insulation board beneath the screw heads. Two different specimen types were constructed, those with flaws 3mm thick and those with flaws 6mm thick. There were 6 replicates in each group giving a total of 18 specimens. Following the pushout tests the specimens were deconstructed to visually confirm that the topping had been well compacted beneath the screw head of the control specimens.

4.2.4. Statistical Methods

Statistical methods have been used to analyse the results. The methods are infrequently used in the presentation of pushout test results but are commonly used in other research fields. A three-way analysis of variance (ANOVA) test with a confidence interval of 95% was used to assess the statistical significance of the effect of the factors on $K_s$, $K_e$, $K_{0.6}$ and $F_{\text{max}}$. ANOVA provides a method for comparing multiple group means for statistical significance. By separating the variance which can be ascribed to each group a statistical test can be constructed; in this case the F-test. In the experiment three factors were varied and the stiffness and strength of the specimens measured. For the stiffness and strength measurements the sum of squares (a measure of variance) within groups ($SS_W$) and the total sum of squares ($SS_T$) were calculated. Using these values and Equation 4.4 the sum of squares between groups ($SS_B$) was calculated.

$$SS_T = SS_B + SS_W$$  \hspace{1cm} (4.4)

To provide mean square values the sum of squares values are weighted by the degrees of freedom. If the ratio of the mean square value between groups and the mean square value within groups (F-statistic or $F_0$) was large it suggests that the null hypothesis (that the factor has no effect on the response) should be rejected. To confirm whether this was the case, $F_0$ was compared to an f-distribution to find the probability of obtaining the same effect or larger if the null hypothesis was true; this probability is known as the p-value. If the p-value was less than 0.05 then it was considered that there was sufficient confidence to reject the null hypothesis. For a full description of the statistical procedure see Montgomery (2004).

The assumptions of the ANOVA test are that the factors are fixed, that the specimens are drawn from a population that can be described normally and
that the experiment is randomised. Whilst checking that these assumptions had been met it was noted that for some results there was a departure from normality. An alternative nonparametric method (Wobbrock et al., 2011) was used to verify that the observed departure from normality had not affected the conclusions of the ANOVA test. No significant deviation was found.

4.2.5. Specimen description

The specimens for all phases of testing were constructed from kiln dried glulam blocks 300mm high, 225mm wide and 90mm deep. Prior to construction the glulam blocks were stored in 20/65 conditions (20°C and 65% humidity) to stabilise the moisture content of the timber. The moisture content, measured according to BS EN 13183-1:2002, and density of each timber type are recorded in Table 4.4. An 18mm thick particleboard interlayer was separated from the topping by a plastic sheet. A proprietary, polymer modified, cementitious floor screed, Rotafix IFS, was used for the topping and HECO Schrauben Topix, 6.0mm diameter self-drilling screws were used throughout to connect the timber to the topping. The pushout tests were conducted 7 days after the topping had been placed, at which point the mean compressive and flexural strengths of the topping were 69.0N/mm² and 7.9N/mm² respectively. The mean flexural and compressive strength of the topping was established from 40 x 40 x 160mm prisms, which were stored in the same environment as the pushout specimens whilst they cured. They were then tested according to BS EN 1015-11:1999; in flexure the specimens were loaded at a constant displacement rate of 0.2mm/min and in compression at 0.5mm/min. The density of the topping was 2200kg/m³. Each specimen was symmetrical in design, containing a total of four connectors. The symmetrical nature of the design ensured that eccentric loading was avoided. A description of each specimen type is given in Table 4.3 which refers to the dimensions of Figure 4.7.

4.2.6. Loading Protocol

As stated in section 4.1.2 there is no standard for testing timber-concrete joints but researchers usually adopt the procedure laid out in EN 26891:1991 for determining the strength and deformation characteristics of timber-timber joints. The amended loading cycle used for the phase 1 tests showed that the stiffness of the joint does not deteriorate with several cycles of loading and the in service stiffness of the joint can be gleaned by using the procedures of EN 26891:1991. Whilst there have been some concerns raised about the accuracy of the procedure for calculating a joints stiffness (Dias et al.
Table 4.3.: Specimen description

<table>
<thead>
<tr>
<th>Series</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>(\phi)</th>
<th>(\rho_{mean})</th>
<th>Screw length (mm)</th>
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</thead>
<tbody>
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<td>Phase 2</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S12.5-408-35°</td>
<td>225</td>
<td>18</td>
<td>12.5</td>
<td>35</td>
<td>408</td>
<td>80</td>
</tr>
<tr>
<td>S26.5-408-35°</td>
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<td>18</td>
<td>26.5</td>
<td>35</td>
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<td>18</td>
<td>12.5</td>
<td>55</td>
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<td>80</td>
</tr>
<tr>
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<td>18</td>
<td>26.5</td>
<td>55</td>
<td>408</td>
<td>100</td>
</tr>
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<td>18</td>
<td>12.5</td>
<td>35</td>
<td>544</td>
<td>80</td>
</tr>
<tr>
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<td>26.5</td>
<td>35</td>
<td>544</td>
<td>100</td>
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<td>18</td>
<td>12.5</td>
<td>55</td>
<td>544</td>
<td>80</td>
</tr>
<tr>
<td>S26.5-544-55°</td>
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<td>18</td>
<td>26.5</td>
<td>55</td>
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</tr>
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<td>Phase 3</td>
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<td></td>
<td></td>
</tr>
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<td>19.5</td>
<td>35</td>
<td>432</td>
<td>80</td>
</tr>
<tr>
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<td>18</td>
<td>19.5</td>
<td>35</td>
<td>432</td>
<td>80</td>
</tr>
<tr>
<td>S19.5-35°-6mm</td>
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<td>18</td>
<td>19.5</td>
<td>35</td>
<td>432</td>
<td>80</td>
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</tbody>
</table>

Table 4.4.: Glulam specification

<table>
<thead>
<tr>
<th>Glulam Description</th>
<th>Testing Phase</th>
<th>Density [kg/m³]</th>
<th>M.C. [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>English Larch</td>
<td>2</td>
<td>544.0</td>
<td>13.4</td>
</tr>
<tr>
<td>English Western Red Cedar</td>
<td>2</td>
<td>407.5</td>
<td>16.0</td>
</tr>
<tr>
<td>European Whitewood</td>
<td>1 &amp; 3</td>
<td>432.4</td>
<td>12.8</td>
</tr>
</tbody>
</table>

(2010), the standard remains the most appropriate method for establishing the stiffness of timber-concrete composite joints and joints in these tests were loaded according to EN 26891:1991.

Transducers were arranged as illustrated in Figure 4.8 and measured the slip between timber and interlayer, timber and topping and in-plane rotation of the specimen. The mounting positions of the instruments were chosen to minimise the errors caused by elastic shortening of the specimen.

4.3. Results

The load-slip behaviour of the high density specimens is presented in Figure 4.9. For each specimen type the specimen best representing the mean specimen was chosen. Phase 2 results are presented in four sections: the displacement of the interlayer during the test and the inward rotation of the specimen is reported in section 4.3.1, the stiffness of the joints section is reported in section 4.3.2, the strength of the joints section is reported in section 4.3.3, the failure mode of the joints is reported in section 4.3.4 and the
results are compared to previous testing in section 4.3.5. Subsequently phase 3 results are presented and the effect of a deliberate flaw in the topping on the within group variance and behaviour of the connections is discussed.

4.3.1. Phase 2 - Interlayer Displacement and Specimen Rotation

During the test the displacement of the interlayer relative to the timber was monitored. The displacement of the interlayer as a proportion of the slip between the timber and the topping was then subjected to a three way ANOVA test with 95% confidence interval, to investigate whether the factors had affected the behaviour. None of the factors were found to influence the displacement of the interlayer.

During a symmetrical pushout test there is a tendency for the topping to rotate inwards as the timber is pushed through. The angle through which the topping rotated was calculated for both sides of the specimen using small angle theory and then averaged. A three-way ANOVA test with 95% confidence interval was undertaken to assess whether the factors affected rotation of the specimen during the test. It was found that at 40% of the maximum load rotation was greatest for specimens with high density timber, 35° inclined screws and 26.5mm toppings. However the rotation does not invalidate the test results as it was very small; generally less than 0.1°.

4.3.2. Phase 2 - Connection Stiffness

Table 4.6 presents the statistical results of the experimental factorial and Table 4.7 provides references for each factor. For reference the mechanical
properties of the connections are listed in Table 4.5.

<table>
<thead>
<tr>
<th>Series</th>
<th>$K_s$</th>
<th>CV</th>
<th>$K_e$</th>
<th>CV</th>
<th>$K_{0.6}$</th>
<th>CV</th>
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<tr>
<td></td>
<td>Mean*</td>
<td>(%)</td>
<td>Mean*</td>
<td>(%)</td>
<td>Mean*</td>
<td>(%)</td>
</tr>
<tr>
<td>S12.5-408-35°</td>
<td>6.14</td>
<td>69.2</td>
<td>9.16</td>
<td>38.1</td>
<td>4.59</td>
<td>56.2</td>
</tr>
<tr>
<td>S26.5-408-35°</td>
<td>7.52</td>
<td>32.8</td>
<td>14.39</td>
<td>43.5</td>
<td>6.76</td>
<td>39.2</td>
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<td>16.0</td>
<td>7.33</td>
<td>17.2</td>
<td>1.36</td>
<td>7.5</td>
</tr>
<tr>
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<td>49.0</td>
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<td>15.3</td>
<td>4.33</td>
<td>47.1</td>
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<tr>
<td>S12.5-544-35°</td>
<td>7.52</td>
<td>46.1</td>
<td>11.33</td>
<td>48.9</td>
<td>3.78</td>
<td>16.3</td>
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<tr>
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<td>15.93</td>
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<td>8.72</td>
<td>23.0</td>
<td>5.06</td>
<td>23.1</td>
</tr>
</tbody>
</table>

*mean of 6 specimens

At low loads, $F < 0.4 F_{\text{max}}$, the stiffness of the connector was most influenced by the inclination of the screw whilst the effect of the topping thickness was only significant on the second cycle of loading. However the coefficient of variation within groups, for the serviceability slip modulus was large compared to the slip modulus from the second cycle of loading, and so it seems reasonable to suggest that the observation of any effect was impeded by the large variance. As the screw inclination was the most significant factor, it was considered possible that the variance within groups was caused by
variance in the angle at which the screws were installed. To investigate this hypothesis the angle at which each screw was installed was measured and compared with the variance in stiffness within each group. No correlation was found between the variance and measured screw angle.

At higher loads, $0.6F_{\text{max}}$, although the stiffness of the connection was affected by the inclination of the screws the influence of the topping thickness was most significant. The density of the timber had no appreciable effect on the stiffness of the connections. Thick toppings with connectors inclined furthest from upright led to the stiffest arrangement.

Analytical approaches tend to assume that deformation does not take place in the topping. For example, clause 7.1 of EN1995-1-1 (2004), assumes the topping to be rigid, even though it is now understood that this is not the case (Dias et al. 2010; Persaud et al. 2010). As the stiffness of the connection in these tests was affected by the thickness of the topping it follows that deformation or rotation must occur within the topping layer. These results suggest that deformation is greatest when the topping is thinnest and therefore the reduction in slip modulus may be most easily observed in tests with thin toppings.
4.3.3. Phase 2 - Connection Strength

The mean strength and variance of each group of connections are presented in Table 4.8. As the variance within groups of connectors was smaller for the ultimate shear strength than the stiffness of the connectors, the differences between each group were more easily observed and all the main effects of the factors were found to have an influence on the shear strength of the connectors (Table 4.9). In addition a substantial interaction effect between the density and topping thickness factors was found (Figure 4.10). Specimens which had a thick topping and screws inclined at 35° tended to draw the connectors out of the timber whereas the other specimens had a more brittle failure, characterised by the screw head pulling out of the topping. Those specimens where the screws were withdrawn from the topping had a higher strength than the those where the screw head was pulled out from the topping. Of the main effects, the thickness of the topping was the most significant.

4.3.4. Phase 2 - Failure Mode

The failure of each specimen is described by one of three modes (Figure 4.11). Specimens with a 12.5mm thick topping failed by connector pullout from the topping (mode a), whereas specimens with thicker topping and screws angled at 35° failed due to the axial withdrawal of the screw from the timber (mode b). Specimens with the thickest topping and screws inclined at 55° either failed by mode b or by vertical cracking in the topping caused by the
Table 4.8.: Phase 2 connection strength

<table>
<thead>
<tr>
<th>Series</th>
<th>$F_{\text{max}}$ Mean* (kN/mm)</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S12.5-408-35°</td>
<td>2.27</td>
<td>32.4</td>
</tr>
<tr>
<td>S26.5-408-35°</td>
<td>4.42</td>
<td>14.4</td>
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<tr>
<td>S12.5-408-55°</td>
<td>2.72</td>
<td>20.8</td>
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<tr>
<td>S26.5-408-55°</td>
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<td>S12.5-544-55°</td>
<td>2.71</td>
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<td>S26.5-544-55°</td>
<td>7.55</td>
<td>7.6</td>
</tr>
</tbody>
</table>

*mean of 6 specimens

Figure 4.10.: Connection strength: topping thickness-timber density interaction

rotation of the screw head (mode c); the mode was dependent on the density of the timber with specimens constructed from the denser timber displaying failure mode c.

4.3.5. Phase 2 - Comparison with Previous Testing

For specimens S26.5-408-35° the mean slip modulus, secant modulus at 0.6 $F_{\text{max}}$ and shear strength of the connectors were 7.52 kN/mm, 6.76 kN/mm and 4.42 kN respectively. The stiffness of these connectors compares well to similar diameter screws tested in pairs inclined at ± 45°, with thicker toppings. Previously reported values vary between 14.4 kN/mm (Deam et al., 2007) and 15.1 kN/mm (Steinberg et al., 2003) for the slip modulus
Table 4.9.: Phase 2 ANOVA connection strength

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>$F_{max}$</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>75.2</td>
<td>1.0*10^-10</td>
</tr>
<tr>
<td>Y</td>
<td>7.25</td>
<td>1.0*10^-2</td>
</tr>
<tr>
<td>Z</td>
<td>515</td>
<td>1.9*10^-24</td>
</tr>
<tr>
<td>XY</td>
<td>2.94</td>
<td>0.09</td>
</tr>
<tr>
<td>XZ</td>
<td>54.6</td>
<td>5.5*10^-9</td>
</tr>
<tr>
<td>YZ</td>
<td>1.39</td>
<td>0.25</td>
</tr>
<tr>
<td>XYZ</td>
<td>0.15</td>
<td>0.71</td>
</tr>
</tbody>
</table>

Figure 4.11.: Failure modes

and between 12.7 kN/mm (Deam et al., 2007) and 14.0 kN/mm (Steinberg et al., 2003) for the secant modulus at 0.6 $F_{max}$. The shear strength of the screws vary between 12.5 kN (Steinberg et al., 2003) and 18.5 kN (Deam et al., 2007). The large spread of reported shear strengths is due to the different timber and concrete embedment depths which in turn causes either brittle fracture of the screw (Deam et al., 2007) or failure of the concrete (Steinberg et al., 2003). The shear strength of connectors in these tests are lower than for screw connections with thicker toppings as pullout of the screw either from the topping or timber is achieved at lower loads.

4.3.6. Phase 3 - Deliberate Topping Flaw

Table 4.10 presents the mechanical properties of the connectors in the phase 3 experiments and Figure 4.12 illustrates the load-slip behaviour for a typical specimen from each group. The serviceability slip modulus of the connectors
for all groups was between 3.84 and 4.83 kN/mm and the secant modulus at 0.6\(F_{\max}\) was between 4.44 and 4.67 kN/mm. These results demonstrate the stiffening behaviour of the connector which can be more clearly observed in Figure 4.12. This phenomena was not reported by Deam et al. (2007) and Steinberg et al. (2003) who both tested SFS screws inclined at ±45°.

![Figure 4.12.: Phase 3: typical load-slip behaviour](image)

**Table 4.10.: Phase 3 connection stiffness**

<table>
<thead>
<tr>
<th>Series</th>
<th>(K_s) Mean (kN/mm)</th>
<th>CV (%)</th>
<th>(K_e) Mean (kN/mm)</th>
<th>CV (%)</th>
<th>(K_{0.6}) Mean (kN/mm)</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S19.5-35°</td>
<td>3.84</td>
<td>30.8</td>
<td>7.02</td>
<td>11.4</td>
<td>4.59</td>
<td>21.6</td>
</tr>
<tr>
<td>S19.5-35°-3mm</td>
<td>3.95</td>
<td>18.4</td>
<td>7.93</td>
<td>19.5</td>
<td>4.44</td>
<td>35.0</td>
</tr>
<tr>
<td>S19.5-35°-6mm</td>
<td>4.83</td>
<td>41.3</td>
<td>7.80</td>
<td>31.3</td>
<td>4.67</td>
<td>13.9</td>
</tr>
</tbody>
</table>

Table 4.11 presents the statistical results of the one-way ANOVA test with 95% confidence limit for the phase 3 tests. The induced flaw was found to not have any measurable effect on the stiffness or strength of the connectors. This suggests that, for the specimens with the flaw, the stress distribution induced by the screw head arches over the zone of low stiffness. Consequently designers using this connector type in a thin topping should not be concerned about localised flaws within the topping as the connection is able to accommodate the imperfection without any loss in performance.
Table 4.11.: Phase 3 ANOVA connection stiffness

<table>
<thead>
<tr>
<th>Source of Variation</th>
<th>$k_s$</th>
<th>P-Value</th>
<th>$k_e$</th>
<th>P-Value</th>
<th>$k_{0.6}$</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insulation flaw</td>
<td>0.89</td>
<td>0.43</td>
<td>0.49</td>
<td>0.62</td>
<td>0.06</td>
<td>0.94</td>
</tr>
</tbody>
</table>

4.4. Concluding Comments

A factorial study investigated how the density, screw inclination and topping thickness influence the behaviour of timber-concrete composite joints constructed with inclined joints. A second phase studied the variance of stiffness within groups observed during the first phase of testing. It was found that the stiffness of the connections was greatly influenced by the inclination of the screw and the thickness of the topping, whilst the density of the timber was not found to have a significant effect. Connections with 26.5mm thick toppings and screws inclined at 35° from the horizontal were the stiffest. All the factors were found to affect the strength of the connection but the thickness of the topping was most significant.

It was concluded that as the stiffness of the connections in these tests was influenced by the thickness of the topping it follows that deformation or rotation must occur within the topping layer. This experimental testing has confirmed the assumption of previous research (Dias et al., 2010) that deformation must take place in the concrete around the connector. Thinner toppings tend to allow greater rotation of the screw head and deformation in the topping layer, resulting in reduced slip stiffness. Consequently thin topping upgrades will require a greater number of connectors to achieve the same composite action that would be achieved when screw connectors are used with conventional topping depths.

Large within group variance for the connection stiffness was found during the testing. No correlation was found between the variance and the measured angle of the screws. Poor compaction of the topping beneath the screw head was shown not to affect the stiffness of the connections, the variation in stiffness or the shear strength of the connectors. The source of the variance is still unknown but the consequence is that a greater number should be tested to acquire a satisfactory mean slip modulus for predicting the flexural behaviour of TCC beams and floors in practice.
5. Cyclic Pushout Tests

The cyclic pushout testing of shear connectors for a thin topping application is reported in this chapter. The behaviour of a TCC composite structure is heavily influenced by its shear connectors, which are usually characterised under short-term, monotonic loading according to EN 26891:1991 (CEN, 1991). This loading regime provides information about the stiffness and strength of the connections under short-term loading but does not give the dynamic stiffness of the connections or their ability to dissipate energy during cyclic loading. It would be informative for further development of the composite upgrade system to determine whether:

- at low amplitude cyclic loading the stiffness of the connectors is significantly different to when they are loaded under short-term monotonic loading;
- the energy dissipated by the connectors is significant;
- the frequency or amplitude of the loading affects either the stiffness or the damping properties of the connectors.

Previous research does include cyclic testing of TCC connectors, but only to understand fatigue loading problems (for example Rautenstrauch & Mueller (2011)), which are larger in magnitude than vibration loading. In contrast, cyclic testing of nailed timber-timber connections at in-service loads has been attempted (Chou, 1987), but only a limited number of tests have been completed.

Since testing of this nature has not been previously attempted, the displacement and loads are both small, and no standard loading protocol exists, the testing methodology will be developed before the test results are then reported. Aspects of the connector’s behaviour which will be reported include: dynamic stiffness and energy dissipation, with respect to frequency and displacement amplitude.

The chapter is structured as follows: the experimental methodology is described in section 5.1; the analysis methodology is presented in section 5.2; the stiffness of the connection under monotonic load are reported in section 5.3; the dynamic stiffness of the connectors under cyclic loads are presented.
in section 5.4; the capability of the connectors to dissipate energy is reported in section 5.5 and the findings are compared with previous testing of timber-timber joints in section 5.6.

5.1. Experimental Methodology

5.1.1. Cyclic Loading Protocol Design

There is at present no standard loading protocol for cyclic testing of TCC joints at the magnitudes of load found in-service. In designing a loading protocol for this purpose, it was first assumed that:

1. at all times, shear connectors in a TCC floor are loaded in shear as they resist the slip between the timber and topping due to a nominal static imposed load acting on the floor;

2. as a TCC floor vibrates due to a footfall input, the shear connectors are cyclically loaded, but always remain in compression because the amplitude of the cyclic load is not greater than the shear force caused by the nominal static imposed load acting on the floor;

3. that the magnitude of any load acting on a shear connector varies according to the position of the shear connector in the span of the floor. The connectors at the centre of the floor experience zero slip and therefore zero load, whilst those at the outside of the floor are subjected to the largest loads and deformation (Figure 5.1);.

4. that the magnitude of any load acting on each shear connector is also dependent on the spacing of the connectors.

Figure 5.1.: Interfacial shear flow from a 1 kN point load at mid-span (panel C: see chapter 6 for panel dimensions and material properties)
The proposed basic loading protocol consists of:

- an initial preload of 1.2 kN in compression, to fulfil assumption one;
- a set number of cycles of load about the preload, always maintaining the load in compression, to fulfil assumption two.

The force applied to a floor from a footfall is dependent on: the weight of the individual; the velocity at which they are moving; their choice of footwear and the stiffness of the floor they are walking on (Ohlsson, 1982). This potentially complex load input, was simplified to a 1kN point load, which, if applied at the centre of a TCC floor would first produce a positive bending moment and variable interfacial shear flow along the panel (Figure 5.1), before the panel vibrated until the energy has been dissipated. Because the shear force acting on each connector would be different, several amplitudes of cyclic loading were used to investigate how the load amplitude influences the dynamic stiffness and damping properties of the connectors (fulfilling assumptions three and four).

The final consideration is the frequency of the cyclic loading. It is understood that the rate of loading affects the properties of the connection (Polensek, 1975; Dolan, 1993) and as the frequency of vibration will vary from floor to floor, a range of frequencies should be tested. However, the frequency at which these floor types vibrate (over 8Hz (Johnson, 1994)), is higher than the rate at which most hydraulic servo machines can run, consequently the frequencies of interest could not be tested. As an alternative, a range of low frequencies were tested to indicate whether loading frequency significantly influenced the stiffness and damping properties of the connector.

### 5.1.2. Cyclic Load Cases

Seven load cases were chosen to investigate the influence of loading frequency and load amplitude, on the dynamic stiffness and damping properties of TCC shear connectors. Load cases one to four were chosen to investigate the influence of frequency. Load cases two and five to seven were chosen to investigate the influence of load amplitude. Details of each load case are presented in Table 5.1. Figure 5.2 describes an example loading protocol; load case two.

### 5.1.3. Monotonic Loading Protocol

Prior to and following the cyclic tests, the specimens were subjected to two ramps of monotonic load to assess the stiffness of the connections (Figure 5.3).
Table 5.1.: Load cases

<table>
<thead>
<tr>
<th>Load Case No.</th>
<th>Cycle Offset* (kN)</th>
<th>Load Amplitude* (kN)</th>
<th>Cycle Frequency (Hz)</th>
<th>No. cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.2</td>
<td>1.0</td>
<td>0.1</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
<td>1.0</td>
<td>0.2</td>
<td>200</td>
</tr>
<tr>
<td>3</td>
<td>1.2</td>
<td>1.0</td>
<td>0.5</td>
<td>200</td>
</tr>
<tr>
<td>4</td>
<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
<td>200</td>
</tr>
<tr>
<td>5</td>
<td>1.2</td>
<td>0.25</td>
<td>0.2</td>
<td>200</td>
</tr>
<tr>
<td>6</td>
<td>1.2</td>
<td>0.5</td>
<td>0.2</td>
<td>200</td>
</tr>
<tr>
<td>7</td>
<td>1.2</td>
<td>0.75</td>
<td>0.2</td>
<td>200</td>
</tr>
</tbody>
</table>

*load applied to the specimen (four connectors)
The stiffness of the connections was estimated from Equation 5.1:

\[ k = \frac{F_{0.4,est} - F_{0.1,est}}{v_{0.4} - v_{0.1}} \]  
(5.1)

Where:
- \( F_{0.4,est} \) is 40% of the estimated maximum load
- \( F_{0.1,est} \) is 10% of the estimated maximum load
- \( v_{0.4} \) is the displacement at 40% of the estimated maximum load
- \( v_{0.1} \) is the displacement at 10% of the estimated maximum load

An estimated maximum load, \( F_{est} \), of 2.25 kN per connector was chosen on the basis of previous testing (see chapter 4).

5.1.4. Specimen Construction

Previous cyclic testing of nailed timber-timber joints (Chou, 1987) has used asymmetric specimens, to allow the loading apparatus to clamp the specimen at both ends and load them in tension and compression. For these tests the loading protocol does not load the specimens in two directions, instead they remain in compression throughout the test. Therefore three specimen types could be considered: symmetrical (Figure 5.4a); pure shear (Figure 5.4b) and asymmetrical (Figure 5.4c).

![Figure 5.4: Specimen options](image)

Figure 5.4.: Specimen options: (a) symmetrical specimen, (b) pure shear specimen and (c) asymmetrical specimen

Pure shear specimens are more complicated to construct than either symmetrical or asymmetrical specimens. Symmetrical specimens are the simplest to load, since, unlike asymmetrical specimens, they do not require horizontal...
restraint, and unlike pure shear specimens, the joint does not have to be loaded precisely at its centre to avoid eccentricity of the load. On the other hand, as a symmetrical specimen is loaded, as well as resisting shear, it rotates inwards. These specimens can also take longer to construct than asymmetrical specimens, requiring two castings, if the topping is cast with the specimen laid horizontal rather than on end. Nonetheless, symmetrical pushout specimens are the most popular amongst researchers testing TCC connections under monotonic load (Monteiro et al., 2012) and so to provide continuity in test setup and specimen type, symmetrical specimens were chosen for these tests.

Four specimens were constructed from kiln dried, European Whitewood, glulam blocks measuring 300mm high, 225mm wide and 90mm deep. The glulam had a mean density of 418kg/m³ and moisture content of 12.5%. The 19.5mm thick screed topping was separated from the 18mm thick particleboard interlayer by a plastic sheet. A proprietary, polymer modified, cementitious floor screed, Rotafix IFS, was used for the topping and HECO Schrauben Topix, 6.0mm diameter self-drilling screws, inclined at 45°, were used throughout to connect the timber to the topping. Each specimen contained four connectors and was symmetrical in design. The pushout tests were conducted at 14 (system 1) and 345 (system 2) days after the topping had been placed. At 345 days, the mean compressive and flexural strengths of the topping were 64.4N/mm² and 11.2N/mm² respectively and the density of the topping was 2200kg/m³.

5.1.5. Test Arrangement

Specimens were supported at the topping on steel blocks. Displacement transducers were arranged as shown in Figure 5.5 and measured the relative displacement between the topping and the timber. They were positioned to mitigate the effects of elastic shortening of the specimen and the mounting blocks were stiffened to avoid unwanted displacement caused by the spring reaction force of the displacement transducer mechanism.

To establish whether the test methodology was reliable and provided reproducible results, each specimen was subjected to the seven load cases in two testing systems (Figure 5.6). System 1 consisted of an air pressurised actuator and 20 kN load cell mounted within a stiff table top frame. System 2 consisted of a hydraulic fluid pressurised actuator and internal load cell, calibrated for loads between 0 and 20 kN, mounted within a standard loading frame. In both tests systems, load was applied to the specimens in load control rather than in displacement control.
Figure 5.5.: Displacement transducer arrangement

Figure 5.6.: Test system 1 (left) and test system 2 (right)
5.2. Analysis Methodology

The data was processed using a purpose written MATLAB script. The stiffness of the connectors during each cycle of loading was evaluated by assuming a linear stiffness between the maximum \( x_{\text{max}} \) and minimum \( x_{\text{min}} \) displacements (Figure 5.7, Equation 5.2).

\[
k = \frac{F_{x,\text{max}} - F_{x,\text{min}}}{x_{\text{max}} - x_{\text{min}}} \tag{5.2}
\]

Where:

- \( x_{\text{max}} \) is the maximum displacement in each load cycle
- \( x_{\text{min}} \) is the minimum displacement in each load cycle
- \( F_{x,\text{max}} \) is the load at which the maximum displacement occurs each load cycle
- \( F_{x,\text{min}} \) is the load at which the minimum displacement occurs each load cycle

The energy dissipated during each cycle of loading, \( e_d \), was established from the area of the hysteresis cycle, assuming the area between two consecutive points and a datum \((y = 0)\) was a trapezoid. This method was sufficiently accurate and it was not necessary to use alternative, more accurate methods, to estimate the hysteresis area; in particular Simpson’s Rule, a numerical integration method for approximating definite integrals. With the hysteric energy dissipation, \( e_d \), established, it was expressed as a proportion of the
potential energy in the system and the equivalent viscous damping ratio, \( \zeta \), was found, Equation 5.3.

\[
\zeta = \frac{e_d}{2\pi kX^2\Omega}
\]  

Equation 5.3

Where:

\( \Omega = \frac{\omega}{\omega_n} \) is the ratio between the excitation frequency and the undamped natural frequency; \( \Omega \) is assumed to be equal to one.

5.3. Connection Stiffness Under Monotonic Load

The stiffness of the specimens under monotonic load before and after the cyclic tests are discussed in this section. Table 5.2 presents the stiffness of the specimens under monotonic loading during the first and second ramps prior to the cyclic tests, and the first ramp following the cyclic test.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Pre-cyclic loads</th>
<th>Post cyclic loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( k ) (1st ramp)</td>
<td>( k ) (2nd ramp)</td>
</tr>
<tr>
<td>A</td>
<td>2.41 (kN/mm)</td>
<td>2.63 (kN/mm)</td>
</tr>
<tr>
<td>B</td>
<td>3.99 (kN/mm)</td>
<td>5.40 (kN/mm)</td>
</tr>
<tr>
<td>C</td>
<td>3.47 (kN/mm)</td>
<td>4.33 (kN/mm)</td>
</tr>
<tr>
<td>D</td>
<td>2.56 (kN/mm)</td>
<td>3.48 (kN/mm)</td>
</tr>
</tbody>
</table>

The mean stiffness of the connectors during the first loading ramp, prior to cyclic testing, was 3.1 kN/mm. As expected, the connectors increased in stiffness during the second ramp to a mean of 4.0 kN/mm. After the cyclic tests all the specimens, apart from specimen B, had increased in stiffness compared to before the cyclic tests. However, if specimen B is discounted, a one-way analysis of variance has insufficient power to conclude that the cyclic loading stiffened the specimens.

5.4. Connection Dynamic Stiffness

5.4.1. Influence of Load Frequency

Figures 5.8a and 5.8b present the dynamic stiffness of the specimens with respect to frequency of loading. Specific load cases and testing systems are indicated in the figure by the number of the testing system preceding the...
specimen letter. For example, 2B refers to specimen B tested in system 2. Each point in the figures is a mean value of either 100 cycles (load case one) or 200 cycles (load cases two to seven). During the test the dynamic stiffness of the connections was not found to deviate beyond the scatter caused by the inaccuracy of the measuring devices and analysis methodology. Figure 5.8a presents the measured dynamic stiffness, whereas Figure 5.8b presents the results normalised to the first result in each series so as to better observe trends in the data.

![Graph](image)

Figure 5.8a shows that the mean stiffness of specimen B, for load cases one to four, decreased by 42% between testing in system 1 and system 2. This provides corroborating evidence, to that provided in Table 5.2, suggesting
that specimen B lost stiffness between testing using system 1 and 2; most likely due to damage during transportation.

In Figure 5.8b the dynamic stiffness of each specimen is normalised, and it becomes apparent that the choice of loading system has an effect on the measured stiffness with respect to frequency. Specimens tested in system 1 increase in dynamic stiffness with increasing frequency, whereas specimens tested in system 2 decrease in stiffness between 0.5Hz and 1.0Hz. Although it might appear that the influence of the testing system is significant, apart from specimen B, the difference between specimens tested in each system is within the coefficient of variance measured in the static tests; between 9% and 35%.

### 5.4.2. Influence of Load Amplitude

Figures 5.9a and 5.9b present the dynamic stiffness of the connection with respect to load amplitude. As shown in Figure 5.8a, the dynamic stiffness of specimen B decreased between tests in system 1 and 2 by an average of 36%, for load cases two and five to seven. Otherwise specimens tested in both systems follow the same general trend; decreasing dynamic stiffness with increasing load amplitude. The mean decrease in the dynamic stiffness of the specimens tested in system 1 was 20% between load amplitudes of 0.25 kN and 1.0 kN, whereas when tested in system 2 the mean decrease in dynamic stiffness was 32%.

With the dynamic stiffness normalised (Figure 5.9b) it can be seen that, as shown in Figure 5.9a, the testing system influences the dynamic stiffness of the specimens. Whilst specimens tested in system 1 reduce in dynamic stiffness with increasing load amplitude from 0.5kN, when tested in system 2 they decreased in dynamic stiffness from a load amplitude of 0.25kN. Compared to the loading frequency, the influence of the load amplitude was more significant. Whereas the change in dynamic stiffness of the specimens was at most 12% between 0.1Hz and 1.0Hz, the change in dynamic stiffness of the specimens was as high as 39% between 0.25kN and 1.0kN.

Correlation of results from the two testing systems was closest at larger load amplitudes and lower load frequencies (Figure 5.9a), which is cyclic loading most akin to monotonic loading. Nonetheless, the difference in dynamic stiffness between the specimens tested under cyclic load case 1 and ramp 2 prior to cyclic testing varies between 15% and 53%; with the specimens always least stiff under monotonic load.
Figure 5.9.: Connection dynamic stiffness vs load amplitude
5.5. Connection Energy Dissipation

5.5.1. Influence of Load Frequency

Figure 5.10 presents the equivalent viscous damping ratio with respect to loading frequency. Each point in the figure is a mean value of either 100 cycles (load case one) or 200 cycles (load cases two to seven). During the test the equivalent viscous damping ratio of the connections was not found to deviate beyond the scatter caused by the inaccuracy of the measuring devices and analysis methodology.

From Figure 5.10, it is seen that the testing system influences the measured equivalent viscous damping ratio. Specimens tested in both systems dissipate more energy as frequency increases, the mean equivalent viscous damping ratio of the specimens at 1 Hz is 17.0% for system 1, whereas it is only 10.8% for system 2.

5.5.2. Influence of Load Amplitude

With respect to load amplitude, Figure 5.13 illustrates a trend of increasing equivalent viscous damping ratio with increased load amplitude. The results of specimen A, from testing in system 2, have been discounted as there is additional damping prevalent at the lower load amplitudes, which is not observed with the other specimens. For system 1 the mean increase is 104%, whereas for system 2 the increase is only 12%.

As with the other test series, it is clear that the test system influences the energy dissipation that is measured. This is of concern as the test should
be repeatable without significant variance in results. The only significant difference between the test systems is the displacement transducers used to measure the slip between the timber and topping. It is possible that energy is dissipated through friction within the mechanism of the transducer. From this point of view, non-contact measuring systems might provide a more suitable method for measuring slip in small amplitude, cyclic tests.

5.6. Comparison with Previous Testing

From the results presented thus far in this chapter, it has been shown that the testing system influences the dynamic stiffness and energy dissipation properties of TCC connectors subjected to small amplitude cyclic loads. As small amplitude, SLS, cyclic testing of TCC joints has not previously been reported, it is difficult to gauge whether the phenomena being measured in these tests is realistic or if factors, other than the testing system, are influencing the observations. Whilst other data for TCC joints does not exist there has been an attempt to evaluate these properties from cyclic testing of timber-timber joints (Chou, 1987).

Chou (1987) tested specimens that were constructed from 50mm x 100mm x 400mm long Douglas fir studs and 100mm x 400mm long plywood connected with 2.64mm diameter nails. Four joint types were constructed: friction-joints, formed from stud overlapped with plywood by 100mm with a nail through the centre; gap-joints, identical to the friction-joint in all aspects apart from a 1.25mm gap separating the stud and plywood interface; wood-nail joints, a single stud and nail without the second plywood member, and plywood-nail joints, the wood-nail joint with plywood substituted for the stud. Joints were subjected to three cycles of loading, at five load amplitudes, which passed through zero so that the joint was loaded in tension and compression. The specimens were loaded in displacement control, at a rate of 38 mm/min.

Results for the dynamic stiffness and equivalent viscous damping ratio of the timber-timber joints were plotted against load amplitude and are reproduced in Figures 5.12 and 5.13 respectively, with data for TCC specimen A overlaid.

5.6.1. Dynamic Connection Stiffness

As with the TCC tests (sections 5.4 and 5.5), Chou (1987) reported that neither the dynamic stiffness nor damping was influenced by the number of cycles of loading. Chou found that the load amplitude affected the dynamic
Figure 5.11.: Connection equivalent viscous damping ratio vs load amplitude

Figure 5.12.: Connector dynamic stiffness with respect to load amplitude (Chou, 1987) with results 1A and 2A overlaid
stiffness of the joints more than the equivalent viscous damping ratio but the TCC results contradict this finding. In the TCC tests the equivalent viscous damping ratio increased by up to 282%, between load amplitudes of 0.25kN and 1.0kN, whilst the equivalent viscous damping ratio only increased by a maximum of 12% between 0.1 Hz and 1.0 Hz.

Chou (1987) found that the joints had a higher slip moduli at lower loads and suggested that crushing of the cellular structure of the timber and higher loads was the reason for this phenomena. Chou concluded that crushing of the cellular structure at higher loads led to greater energy dissipation. In the TCC joint tests, specimens tested with system 2 followed the trend observed by Chou, whereas when tested with system 1 the specimens initially gained stiffness with increased load amplitude (0.25kN to 0.5kN) before decreasing in stiffness with increasing load amplitude.

5.6.2. Connection Energy Dissipation

Chou (1987) found that friction between the members was the most significant energy dissipation mechanism. In particular the opening and closing of a joint had a significant influence on the energy dissipation of the joint. In view of these findings, the large variance of energy dissipation between TCC specimens could be explained by the gaps between timber, interlayer and topping, which were not controlled during manufacture of the specimens.

Chou (1987) remarked that the dynamic stiffness of the joints tended to converge at higher load amplitudes. It was reported that this was due to all joints having formed gaps at the higher load levels. Converging of joint stiffness was also observed for the TCC specimens (Figure 5.13). Whilst this may be due to all the joints having formed gaps between member interfaces at higher loads, it could equally be due to all joints surpassing the load threshold at which manufacturing inconsistencies, other than joint gap, have a significant influence. Whichever explanation is correct they both point towards the same trend; increasing joint uniformity at higher loads.
Figure 5.13.: Connector damping with respect to load amplitude (Chou, 1987) with results 1A and 2A overlaid
5.7. Concluding Comments

In this chapter the cyclic testing of TCC joints has been reported. As the tests were non-destructive, four specimens could be tested with two different testing systems. Seven load cases provided data to study the influence of loading frequency and load amplitude, on the connector dynamic stiffness and equivalent viscous damping ratio. The results were compared to previous testing of timber-timber joints (Chou, 1987) to provide assurance as to the quality of the test method and results.

In these small amplitude, cyclic load tests, it was found that the testing system influences the dynamic stiffness and equivalent viscous damping ratio of the connectors. This in turn suggests that data from this type of test is non-reliable and non-reproducible. As the test results were highly susceptible to influence by the testing system, only the following general observations can be concluded:

- neither dynamic stiffness nor damping of joints is affected by the number of cycles of load (up to 200 cycles);
- friction at the interfaces between the timber joist, interlayer and topping are likely to be the most significant contributors to energy dissipation in joints;
- as load amplitude increases, the variance in connector dynamic stiffness between the four specimens reduced. Chou (1987) observed the same trend and suggested that the effect was due to crushing of the cellular structure which reduced inconsistencies;
- general observed trends include: increasing loading frequency leads to reduced equivalent viscous damping ratio whilst increasing load amplitude leads to reduced dynamic stiffness and increased equivalent viscous damping ratio.

Further work needs to focus on the testing method before future testing is attempted. This should include:

- investigation to select the most appropriate test specimen for the tests. It may be that asymmetrical or pure shear specimens display smaller within group variance or influence the dynamic stiffness or energy dissipation properties of the connectors.
- a larger number of specimens to allow the statistical significance of results to be established;
- investigation to identify the most appropriate means of measuring displacements without unwanted energy loss within the mechanism. Non-contact devices may be more appropriate.
6. | Panel Flexural Tests

The flexural testing and analysis of timber panels upgraded with a thin cementitious topping is reported in this chapter.

Panels comprise of one, two or three joists and represent a section of a floor. Panels are a cost and time effective method of investigating composite behaviour which is representative of complete floors. Experimental results can be used to validate numerical models which in turn can be used to broaden the investigation through a parametric study. However, care should be taken when extrapolating the results to panels of different spans, as the behaviour will be different. For example if there are two panels of different span but with a shear connection of equal smeared slip modulus (where the smeared slip modulus is the connection stiffness per unit length), the longer panel will display greater composite interaction (Van der Linden, 1999).

The main aim of the test programme was to evaluate how the load-slip behaviour of the connectors influences panel composite performance. Performance characteristics which were considered included panel stiffness, strength and failure mode. Other aims were to assess the:

- effectiveness of the upgrade;
- the linearity of the load-displacement behaviour;
- correlation between experimental and analytical behaviour;
- effects of cycling load within the service range on the bending stiffness of the panels;
- effect of cracked topping on the performance of the panels;
- failure mode of the panels.

The chapter is structured as follows: the construction of the panels is described in section 6.1, the experimental procedures are described in section 6.2, preliminary testing of type 1 panels is reported in section 6.4, the deflection of unpropped panels during and after construction is discussed in section 6.5, the performance of 4.5m long, type 2 panels is reported in section 6.6 and the performance of 2.8m long, type 3 panels is reported in section 6.7. Throughout the chapter results are compared with behaviour predicted by partial interaction theory.
6.1. Panel Construction

6.1.1. Material Characterisation

Whilst the basic construction techniques were common to all panels, three types of panel were constructed. Type 1 panels, A and B, were formed of a single joist spanning 2.8m (Figure E.1, Appendix E), type 2 panels, C, D and E were formed of 2 joists spanning 4.5m (Figure E.2, Appendix E) and type 3 panels, were each formed of 2 joists cut from floor F spanning 2.8m (Figures E.3 and E.4, Appendix E).

The global modulus of elasticity (MOE) of each timber joist was established using an elastic 3-point bending test. Each joist was simply supported, whilst the central point load was applied. The load was increased from 0 to 1 kN over approximately 30 seconds, held for 30 seconds and then unloaded to 0.25kN. The cycle of loading was repeated twice more and the mean MOE was evaluated from the mean gradient of the load-displacement plots. The values are listed in Table 6.1 alongside the dimensions of the joist, mean density and moisture content, established according to BS EN 13813-1. The mean density and moisture content of joists in Floor F were evaluated from eight specimens taken from one joist, randomly sampled from the floor.

<table>
<thead>
<tr>
<th>Joist Description</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>$E_{global}$ (kN/mm²)</th>
<th>$\rho_m$ (kg/m³)</th>
<th>m.c. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>44</td>
<td>95</td>
<td>14.3</td>
<td>539.3</td>
<td>16.5</td>
</tr>
<tr>
<td>B</td>
<td>44</td>
<td>95</td>
<td>11.7</td>
<td>430.8</td>
<td>16.5</td>
</tr>
<tr>
<td>C1</td>
<td>44</td>
<td>170</td>
<td>14.2</td>
<td>533.6</td>
<td>14.0</td>
</tr>
<tr>
<td>C2</td>
<td>44</td>
<td>170</td>
<td>13.8</td>
<td>506.8</td>
<td>14.0</td>
</tr>
<tr>
<td>D1</td>
<td>44</td>
<td>170</td>
<td>11.7</td>
<td>498.5</td>
<td>13.3</td>
</tr>
<tr>
<td>D2</td>
<td>44</td>
<td>170</td>
<td>15.0</td>
<td>543.5</td>
<td>14.3</td>
</tr>
<tr>
<td>E1</td>
<td>44</td>
<td>170</td>
<td>14.8</td>
<td>533.2</td>
<td>13.9</td>
</tr>
<tr>
<td>E2</td>
<td>44</td>
<td>170</td>
<td>10.2</td>
<td>460.2</td>
<td>13.1</td>
</tr>
<tr>
<td>F1</td>
<td>44</td>
<td>120</td>
<td>13.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F2</td>
<td>44</td>
<td>120</td>
<td>11.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F3</td>
<td>44</td>
<td>120</td>
<td>9.51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F4</td>
<td>44</td>
<td>120</td>
<td>10.5</td>
<td>438.2*</td>
<td>11.5*</td>
</tr>
<tr>
<td>F5</td>
<td>44</td>
<td>120</td>
<td>10.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F6</td>
<td>44</td>
<td>120</td>
<td>12.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F7</td>
<td>44</td>
<td>120</td>
<td>15.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F8</td>
<td>44</td>
<td>120</td>
<td>12.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*mean of eight specimens

The mean flexural and compressive strength of the topping was established
from 40x40x160mm prisms which were tested on the same day as the relevant panel, using BS EN 1015-11 with an adapted load rate. In flexure the specimens were loaded at a constant displacement rate of 0.2mm/min and in compression at 0.5mm/min. The mean values are listed in Table 6.2.

Table 6.2.: Topping mean flexural and compressive strength

<table>
<thead>
<tr>
<th>Test Age</th>
<th>Mean Flexural Strength* (N/mm²)</th>
<th>Mean Compressive Strength* (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel A</td>
<td>9.3</td>
<td>74.2</td>
</tr>
<tr>
<td>Panel B</td>
<td>8.6</td>
<td>79.5</td>
</tr>
<tr>
<td>Panel C</td>
<td>7.6</td>
<td>70.1</td>
</tr>
<tr>
<td>Panel D</td>
<td>7.6</td>
<td>72.9</td>
</tr>
<tr>
<td>Panel E</td>
<td>7.5</td>
<td>68.6</td>
</tr>
<tr>
<td>Floor F Mix 1</td>
<td>7.3</td>
<td>66.7</td>
</tr>
<tr>
<td>Floor F Mix 2</td>
<td>7.2</td>
<td>62.8</td>
</tr>
</tbody>
</table>

* flexural strength - mean of three specimens  
*compressive strength - mean of six specimens

Six, 100mm diameter, 200mm high cylinders were also cast with the purpose of establishing the secant modulus of elasticity in compression. The cylinders were tested according to prEn 12390-13, with two transducers on opposing sides of each cylinder mounted across the middle third to measure the change in length of the cylinder. The initial secant modulus of elasticity, \( E_{C,0} \), and the stabilised secant modulus of elasticity, \( E_{C,S} \), are reported in Table 6.3. The mean initial modulus of elasticity (MOE) and stabilised secant MOE were 21.1 and 21.8 GPa. The coefficient of variation was 6% or less for both values.

Table 6.3.: Topping secant modulus of elasticity

<table>
<thead>
<tr>
<th>Cylinder No.</th>
<th>( E_{C,0} ) (GPa)</th>
<th>( E_{C,S} ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.3</td>
<td>19.7</td>
</tr>
<tr>
<td>2</td>
<td>22.7</td>
<td>23.3</td>
</tr>
<tr>
<td>3</td>
<td>21.2</td>
<td>21.8</td>
</tr>
<tr>
<td>4</td>
<td>22.0</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>20.1</td>
<td>21.9</td>
</tr>
<tr>
<td>6</td>
<td>21.6</td>
<td>22.4</td>
</tr>
<tr>
<td>Mean</td>
<td>21.1</td>
<td>21.8</td>
</tr>
<tr>
<td>CV (%)</td>
<td>5.9%</td>
<td>6.0%</td>
</tr>
</tbody>
</table>
6.1.2. Panel Manufacture

Each timber panel was constructed by fixing 18mm thick sheets of particleboard to the top sides of the joist with 40mm long wood screws at 400c/c. The resulting cross-section was either a single or double T-beam, 400mm or 800mm wide. Having constructed the panels their elastic bending stiffness was established by a 3-point bending test (Figure 6.1). Each panel was loaded with a line load applied across the centre of the panel whilst a displacement transducer positioned beneath each joist measured the mid-span deflection. The gradient of the load-deflection plot was then used in conjunction with Equation 6.1 to estimate the effective bending stiffness of each panel.

![Figure 6.1.: Establishing the effective bending stiffness of the timber panels](image)

$$w = \frac{Pl^3}{48EI} \quad (6.1)$$

With the effective bending stiffness of each panel established, the panels were upgraded using the following sequence:

1. the location for each screw connector was marked and the particleboard predrilled to aid the installation of the screws at the correct angle,
2. the panels were covered with a plastic sheet, which was stretched and held in place by formwork fixed to the side of the panel,
3. the screws were installed using a timber wedge to ensure a consistent inclination and a steel block was used to locate the screw head at the correct height from the surface of the particleboard,
4. the topping was placed and positioned over the panel to achieve a level finish (Figure 6.2). Each panel was covered for 5 days with a plastic sheet to minimise plastic shrinkage of the topping.
Panels A and B were constructed supported along their length, equivalent to fully propped. In this case the self-weight of the panel was shared by the whole composite section. The remaining panels were upgraded whilst simply supported and in this scenario the weight of the wet topping was supported by the timber panels. Consequently panels constructed in this manner suffered both instantaneous and creep deflection.

Table 6.4.: Panel Descriptions

<table>
<thead>
<tr>
<th>Panel</th>
<th>Span (mm)</th>
<th>Propped/ Unpropped</th>
<th>Angle of Screws (°)</th>
<th>Screw Spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2800</td>
<td>Propped</td>
<td>45</td>
<td>100</td>
</tr>
<tr>
<td>B</td>
<td>2800</td>
<td>Propped</td>
<td>45</td>
<td>200</td>
</tr>
<tr>
<td>C</td>
<td>4500</td>
<td>Unpropped</td>
<td>45</td>
<td>100</td>
</tr>
<tr>
<td>D</td>
<td>4500</td>
<td>Unpropped</td>
<td>35</td>
<td>100</td>
</tr>
<tr>
<td>E</td>
<td>4500</td>
<td>Unpropped</td>
<td>45</td>
<td>75-300</td>
</tr>
<tr>
<td>F1-F2</td>
<td>2800</td>
<td>Unpropped</td>
<td>35</td>
<td>50-200</td>
</tr>
<tr>
<td>F3-F4</td>
<td>2800</td>
<td>Unpropped</td>
<td>35</td>
<td>50-200</td>
</tr>
<tr>
<td>F5-F6</td>
<td>2800</td>
<td>Unpropped</td>
<td>35</td>
<td>50-200</td>
</tr>
<tr>
<td>F7-F8</td>
<td>2800</td>
<td>Unpropped</td>
<td>35</td>
<td>50-200</td>
</tr>
</tbody>
</table>

The spacing of the screws and layout of the panels are also illustrated in the plans, elevations and sections reported in Appendix E. The stiffness of the connectors, evaluated from pushout tests, described in Chapter 4, are reproduced in Table 6.5.

Table 6.5.: Connector Stiffness

<table>
<thead>
<tr>
<th>Screw</th>
<th>$k_s$ (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle (°)</td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>4335</td>
</tr>
<tr>
<td>45</td>
<td>4150</td>
</tr>
</tbody>
</table>

Figure 6.2.: Construction process: screw head protruding from panel, pinned support and topping placement
6.2. Experimental Methodology

6.2.1. Monitoring During Construction

Following placement of the topping, the mid-span deflection of each simply supported panel was monitored for 10 days, during which the temperature and relative humidity of the laboratory were recorded. The purpose of monitoring the panels and floor during and after construction was to investigate the consequences of unpropped construction. Comparing the initial deflection, 10 days after construction, with literature examples, was expected to give an indication of the need for propping to minimise long-term deflections. During construction the panels and floor were supported at the joist ends but were unsupported along the other edges. Displacement transducers, positioned at the centre of each joist, measured the deflection of the floor whilst temperature and relative humidity sensors monitored the environmental condition of the laboratory. The results are presented later, in section 6.5.

6.2.2. Short-term Bending Tests

Following curing of the topping the panels were subjected to short term bending tests. Each panel was loaded with six equally spaced line loads, approximating a uniformly distributed load (Figure 6.3). Other loading patterns were considered, including three and four point bending and the bending moment envelope from a moving point load but they were dismissed after the following considerations. It was regarded as important to choose a loading pattern which was practical to achieve and resulted in a shear distribution which demonstrated the failure mode likely to be encountered in practice. Four point bending is usually favoured over three

Figure 6.3.: Short-term bending test loading pattern
point bending, as the loading pattern ensures a pure bending failure if the panel fails within the middle third. However the connector behaviour studied in Chapter 4, in conjunction with predictions of interfacial slip suggests that for this system the panels will have lost composite action before the timber joists fail in combined bending and tension. Rather than study the failure of timber joists it was thought more important to observe how the loss of composite action progresses under a typical or worst case loading scenario. A four point bending loading regime is not a loading pattern experienced in practice and the shear distribution is different to that of a UDL (Figure 6.4). The bending moment envelope achieved by a point load at every position across the span of the panel, was shown by Gai (2012) to be a more critical loading scenario than a UDL for a simply supported floor. Gai (2012) mimicked this loading pattern by using two hydraulic pumps to apply five line loads (the central load is greater than the outer loads). Compared to six equal line loads, this loading pattern is less practical to achieve, as only a single hydraulic pump is required to apply the equal loads.

Figure 6.4.: Shear diagrams from a UDL, three-point bending and four-point bending

Figure 6.5 illustrates the shear and bending moment envelopes generated from the experimental setup (six point loads) and a theoretical UDL (total
load of six experimental point loads spread over the panel area). The figure highlights that the experimental loading is more onerous than the theoretical. The bending moment, which is mathematically related to the curvature of the panel, is 16.7% higher for the experimental load case than the theoretical load case throughout the bending moment envelope. To ensure that the comparison between theoretical and experimental deflections was valid, the theoretical deflections were increased to reflect the difference in bending moment by multiplying the theoretical deflections by 16.7%.

![Shear Force Graph](image1)

![Bending Moment Graph](image2)

Figure 6.5.: Experimental and theoretical shear and bending moment envelopes

Four loading protocols were used. Protocol 1, Figure 6.6, loaded the panel to 40% of the ultimate design load, held the load for 30 seconds, unloaded to 10% of the ultimate design load, held for 30 seconds before loading the panel to failure. Protocol 2, Figure 6.7, loaded the panel to failure without cycling the load. Protocol 3, Figure 6.8, had additional cycles of loading and unloading to protocol 1. The panels were cycled between 4% and 10%, 5% and 12.5%, 10% and 40%, and 15% and 65% of the estimated ultimate load before loading to failure. Protocol 4, Figure 6.9, comprised of an initial
30 cycles of loading between 2 load levels which increased by 100% every 10 cycles (A to C, Figure 6.9). The maximum and minimum loads were sustained for 30 seconds during each cycle. After the first 30 cycles the loading programme was repeated (cycles D to F, Figure 6.9) and then further cycles at higher loads were completed before the panels were loaded to failure. Panel A was tested with protocol 1, panel B was tested with protocol 2, panels C, D and E were tested with protocol 3 and panels from floor F were tested with protocol 4.

Displacement transducers were positioned beneath the centre of each joist to measure the mid-span deflection, and at the ends of joists at locations A, B and C (Figure 7.15). Transducers at A measured the end slip between the top of the joist and underside of the concrete topping ($s_{end}$), transducers at B measured the vertical separation of the topping from the timber joists ($s_{sep}$) and transducers at C measured the vertical separation of the interlayer from the particleboard.
6.3. Analytical Method

The results in this chapter are compared with predictions using the analytical \( \gamma \)-method. Comparisons of the mid-span deflection, end-slip and end separation are made at SLS=2.5kN/m². The \( \gamma \)-method is outlined in Appendix D and the equations for calculating mid-span deflection (equation 6.2), end-slip (equation 6.3) and end-separation (equation 6.4) are reproduced below.

\[
\begin{align*}
  w &= \frac{5ql^4}{384 (EI)_{ef}} \\
  s_{\text{end}} &= \frac{T(x)}{K} \\
  s_{\text{sep}} &= \frac{T(x)}{K} \tan \phi
\end{align*}
\]
Figure 6.10.: Joist end transducer arrangement

\[ T(x) = \frac{\gamma S}{z (EI)_{ef}} a(x) \]  

(6.5)

Where:

- \( w \) is the mid-span deflection of the panel
- \( q \) is the load per metre
- \( l \) is the span of the beam
- \( (EI)_{ef} \) is the panel effective bending stiffness
- \( K \) is the slip modulus of the connectors
- \( T \) is the shear load in a connector
- \( z \) is the distance between the centroids of each part
- \( a \) is the spacing of the connectors
- \( \phi \) is the angle of the shear connector, measured from the horizontal

In addition, an effective width was considered to take account of the effects of shear lag. Shear lag is where the distribution of the stresses is not uniform over the flange of the composite member. In EN1994-1-1 (CEN, 2004c) this is taken into account by calculating an effective width for the flange. According to the design standard the effective width \( (b_{eff}) \) is calculated as:

\[ b_{eff} = b_0 + b_{ei} \]  

(6.6)

where \( b_0 \) is the distance between the centres of the outstand shear connectors and \( b_{ei} \) is the value of the effective width of the concrete flanges, calculated as \( l/8 \). For all of these timber-concrete composite panels \( b_0 \) is equal to 0, whilst \( b_{ei} \) is equal to the geometric width for panels which span 4500mm, and 350mm for panels which span 2800mm.
6.4. Preliminary Test Results

The load-displacement behaviour of panels A and B are presented in Figures 6.11 and 6.12 respectively. The experimental displacement is plotted alongside the theoretical displacement of the panel with full connection, without connection, partial connection (analytical) and without upgrade (experimentally derived elastic bending stiffness of the timber panel).

![Graph showing load-displacement behaviour](image)

For panel A the load-displacement behaviour remained linear until 64.3% of the maximum load was attained. For panel B the load-displacement behaviour remained linear until 25.5% of the maximum load was attained. Thereafter the panel gradually lost composite action until the panel reached 69.1% of its final load where upon there was no composite action remaining. The final 25mm of deflection was achieved without interaction.

Following loss of composite action both panels continued to deflect (panel A 31mm and panel B 24mm) before they both failed through sudden and brittle failure of the joists in combined tension and bending. Cracking of the topping was not observed during the testing of either panel. Upon unloading the panels, no cracking to the surface of panel A was found, whilst a hinge had formed in the topping of panel B beneath one of the spreader beams, where the panel had been loaded. The final failure load was 14.0kN/m² for panel A and 11.4kN/m² for panel B.
6.4.1. Shear Connection Efficiency

The efficiency of each panel’s shear connection system is defined by equation 6.7 (Gutkowski et al. 2008) where $w_N$, $w_I$ and $w_C$ are the theoretical deflection with no connection, experimental deflection and theoretical deflection with a full connection respectively. Connection efficiency was assessed at the SLS=2.5kN/m² and was greatest for panel B, 84.8%, and lowest for panel A, 77.2%. For panel A at SLS=2.5kN/m² the upgrade reduced the mid-span deflection from span/130 to span/406, a 212% increase in bending stiffness. Whilst for panel B at the same load, the upgrade reduced the mid-span deflection from span/117 to span/465, a 298% increase in bending stiffness.

\[
\text{Efficiency} = \frac{w_N - w_I}{w_N - w_C} \tag{6.7}
\]

6.4.2. Analytical and Experimental Correlation

The correlation between the analytical $\gamma$-method prediction and experimental mid-span deflection was comparable to previous literature examples (Fragiacomo & Lukaszewska, 2011; Fragiacomo, 2012; Persaud & Symons, 2005; Yeoh, 2010). For panels A and B the error between analytical and experimental values was 16.1% and 11.1% respectively. For both panels
the analytical approach was conservative, predicting greater deflections than were found experimentally.

Table 6.6.: Comparison between experimental and analytical behaviour at SLS (2.5kN/m²)

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th></th>
<th>B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Exp.</td>
<td>6.89</td>
<td>8.00</td>
<td>Exp.</td>
<td>6.02</td>
</tr>
<tr>
<td>Analyt.</td>
<td>8.00</td>
<td>16.1%</td>
<td>Analyt.</td>
<td>6.69</td>
</tr>
<tr>
<td>Err</td>
<td>16.1%</td>
<td></td>
<td>Err</td>
<td>11.1%</td>
</tr>
</tbody>
</table>

### 6.4.3. Summary

Whilst these test were successful in measuring the load-displacement behaviour of the panels, they have several shortcomings. First, the single joist construction led to the panels rotating at failure, in turn requiring the spreader beams to be tethered to the loading frame for safety. Second, single T-beam panels are impractical for upgrading whilst simply supported, because of their instability. Previous thin topping panel test specimens (Stearn, 2006; Gethin, 2007) have been constructed with three joists but, as the panels always failed due to a combined bending and tension failure of a single joist, it seems wasteful to construct the panels with this many joists. The best compromise solution is to test double T-beams which provide rotational stability, whilst making best use of the available resources.
6.5. Monitoring During Construction

The mid-span deflection of panels C, D and E for 10 days following construction are presented in Figure 6.13, whilst the mid-span deflection of floor F for the first 24 hours and 10 days following construction are presented in Figures 6.14 and 6.15. Each figure also includes the relative humidity and temperature of the laboratory environment where the specimens were monitored. In Figures 6.13, 6.14 and 6.15 the large deflections observed during the first hour are attributed to construction workers standing on the test specimen whilst placing the topping. During monitoring of panels C, D and E there was some minor data loss during the first two hours of data recording, but as the initial deflection and deflection after two hours is known, it is not believed to affect the strength of the conclusions made from the following analysis.

![Figure 6.13: Panels C, D and E mid-span deflection 10 days following construction](image)

Figure 6.13: Panels C, D and E mid-span deflection 10 days following construction

Table 6.7 presents the theoretical and measured initial deflections of the
panels and floor. The average initial deflection of each panel and floor compares well with the theoretical immediate deflection, determined from the weight of the wet screed, the bending stiffness of the timber panel/floor and Equation 6.8:

\[
w = \frac{5ql^4}{384EI}
\]

Where:

- \( q \) is the weight of the wet screed
- \( l \) is the span of the floor
- \( EI \) is the bending stiffness of the timber floor parallel to the direction of the joists.
Furthermore there was a relationship between the bending stiffness of each joist and its mid-span deflection. Those joists with the highest bending stiffness, deflecting the least. Floor F had the least stiff joists placed at the centre of the floor and whilst this to some extent explains the greater deflection at the centre of the floor, when the deflection of each joist is normalised according to the bending stiffness of the joist the trend of greatest deflection at the centre of the floor remains. There may have been some pooling of the wet screed at the centre of the floor to cause this deflection profile.

After the initial deflection of the panels and floor, each was observed to recover (Figure 6.14, 2-4 hours). The joists in Floor F rebounded to an average deflection of 0.3mm, a recovery of 1.4mm. There are two possible explanations for the phenomena.

First, the effect could be due to the topping increasing in temperature, as the hydration of cement is an exothermic reaction. The panels were warm to the touch as the topping hardened, confirming that substantial heat was being generated. The coefficient of linear thermal expansion of concrete is usually twice that of timber (depending on aggregate type and species etc. concrete $=5.8 - 14.0 \times 10^{-6}$ mm/mm°C (Lamond & Pielert, 2006) and timber parallel
As the connectors provide a means by which one material restrains the expansion or contraction of the other, an increase in temperature of the topping causes differential expansion of the topping and timber resulting in an upward deflection of the panel.

The second explanation concerns additives added to this type of cementitious floor screed product, which are designed to overcome, by expansion, the shrinkage that would otherwise occur with a thin cementitious layer. The expansion of the topping due to an additive or an increase in temperature has the same effect, the upward deflection of the panel or floor. However an expansion of an additive would cause a permanent upward deflection, whilst the heat generated by the hydration reaction would cause a temporary upward deflection. As the panels rapidly started to deflect after 4 hours, it seems probable that the majority of the upward deflection is due to thermal expansion of the topping.

Following their initial deflection and recovery the specimens continued to deflect downwards; the start of creep deflection. During the following days the environmental condition of the laboratory did not remain constant and this influenced the mid-span deflection of the specimens. As the relative humidity of the laboratory increased, it promoted an increase in moisture content of the timber, resulting in a swelling of its cellular structure. As the timber joists were restrained by the topping with shear connectors, the expansion caused the downward deflection of the floor. The same effect is observed when the temperature of the laboratory decreased. The different thermal expansion properties of the materials caused greater contraction in the topping than timber and the restraint provided by the timber caused the downward deflection of the specimens.

At 7 days the panels C,D and E and Floor F had average span-deflection ratios of \( \delta/l = 1/566 \), \( \delta/l = 1/683 \), \( \delta/l = 1/608 \) and \( \delta/l = 1/665 \) respectively, which meet the deflection limit suggested by current codes of practice. In the long-term, deflections may exceed this limit due to further creep deflection of the composite but as the initial deflections are small, it seems unnecessary to prop the floor during construction.
6.6. Type 2 Panels Test Results

This section discusses the test results of short-term bending testing of type 2, 4.5m span, panels. The load mid-span deflection behaviour of panels C, D and E are presented in Figures 6.16, 6.17 and 6.18 respectively.

![Figure 6.16: Panel C load vs mid-span deflection](image)

6.6.1. Non-linear Stiffness

As all the panels exhibited some non-linear behaviour, Table 6.8 records the highest recorded effective bending stiffness \((E_{I_{ef,\text{max}}})\), found from the steepest tangent of each load-slip plot, and the effective bending stiffness at the serviceability limit state of 2.5kN/m² \((E_{I_{ef}})\). The ratio between the load at which the maximum effective bending stiffness occurs and the failure load of the panel is defined as:

\[
\frac{F_{E_{I_{ef,\text{max}}}}}{F_{\text{max}}} \quad (6.9)
\]

For panel C the load-displacement behaviour was non-linear throughout the test, with significant stiffening of the panel to approximately half the maximum load where the highest stiffness was recorded, 15% higher than serviceability limit state. Between 50% and 75% of the maximum load
the load-displacement behaviour remained linear. Thereafter the panel lost composite action, with no interaction for the final 15mm of mid-span deflection.

For panel D the non-linear load-displacement behaviour was slightly more pronounced than for panel C. The increase in effective bending stiffness between the serviceability limit state load and 45% of the maximum load where the panel stiffness peaked was 22%. Thereafter the panel gradually lost composite action with no interaction exhibited for the last 20mm of mid-span deflection. Loss of composite action resulted in a reduced effective bending stiffness and greater deformations.

The load-displacement behaviour of panel E exhibited the greatest non-linearity. Between the serviceability limit state and 69% of the maximum load the effective bending stiffness increased by 55%. Thereafter the load-displacement relationship remained linear, displaying no loss of composite action before failure. It is believed that premature failure of one of the joists accounts for the greatest effective bending stiffness being attained at a higher proportion of the load in comparison to panels C and D.

The evidence of non-linear load-mid-span displacement behaviour is corroborated by the load-slip behaviour at the ends of the panels (Figure 6.19). The plots for each panel illustrate similar trends in non-linearity, namely: an
initial low stiffness followed by stiffening to a peak stiffness between 7 and 8 kN/m² and, for panels C and D, gradual loss of composite action before the final 2.0-2.5mm of end-slip without composite action.

### 6.6.2. Failure

Prior to failure, panels C and D demonstrated loss of composite action resulting in a reduced effective bending stiffness and greater mid-span deflection for a given load. Panel C did not reach its ultimate load, the central jack reaching its full extension before either of the joists failed, and the panel was unloaded. However, as the panel had lost all composite action, it is presumed that if sufficient curvature could have been induced into the panel, the ultimate failure would have been a combined tension and bending failure of one of the joists. Final failure of panel D was a tension failure that initiated
on the underside of one of the joists at the location of a prominent knot within
the middle third. The failure mechanism of panel E was also a tension failure
of a single joist at a prominent knot location. However, the failure occurred at
a lower load than expected before composite action between the timber and
topping was lost. The failure loads, or maximum loads achieved, by panels
C, D and E were 15.6, 13.9 and 10.2 kN/m². The factor of safety on the design
load can be calculated in the following manner.

The partial factors for the design loads are $\gamma_g = 1.35$ and $\gamma_q = 1.5$.

The possible characteristic imposed loads are $w_{q,1} = 1.5\text{kN/m}^2$ (residential
(BSI, 2002)) and $w_{q,2} = 2.5\text{kN/m}^2$ (office (BSI, 2002)).

Therefore the design imposed loads are:

$$\gamma_q w_{q,1} = 2.3\text{kN/m}^2$$

$$\gamma_q w_{q,2} = 3.8\text{kN/m}^2$$

To obtain the total design loads the weight of the screed $w_s = 0.44\text{kN/m}^2$ is
added to the values above:

$$\gamma_g w_s + \gamma_q w_{q,1} = 2.8\text{kN/m}^2$$
\[ \gamma_g w_g + \gamma_q w_{q,2} = 4.3kN/m^2 \]

As the weight of the screed is not included in the failure load measured by the load cells, this should be added to the failure load of the joists.

\[ w_{q,\text{fail,mean}} + w_g \]

As there are only two joists for every panels we must assume that the above loads are mean failure loads for the joists not characteristic values. We should therefore adjust the strength of the joists to obtain an approximate design value. In previous research Ceccotti et al. (2006) suggested that the characteristic strength could be assumed as 70% of the mean strength. Adopting this approach allows approximate design values to be derived in the following manner:

\[ f_n = \left( w_{q,\text{fail,mean}} + w_g \right) \cdot \left( \frac{f_k}{f_{\text{mean}}} \right) \cdot \left( \frac{1}{\gamma_m} \right) \]

\[ f_n = \left( w_{q,\text{fail,mean}} + w_g \right) \cdot 0.7 \cdot \left( \frac{1}{1.3} \right) \]

Therefore the factor of safety on the design office loading for panels C, D and E is 2.0, 1.8 and 1.3 respectively. For a resident ail loading the factor of safety on the design load for panels C, D and E is 3.0, 2.7 and 2.0. In all cases the factor of safety on the design load exceeds one and for residential loading all panels exceed a factor of safety of two. In this regard the panels have demonstrated adequate performance for both office and residential loading.

![Figure 6.20: Failure of Panels D (left) and E (right)](image)

The sudden and brittle failure of the joists in panels D and E caused the panels to rotate, to the greatest extent at the centre of the span, inducing tensile stresses into the topside of the panel and leading to cracking. Otherwise
cracking of the topping was not observed during the tests. After unloading the panels the only visible cracking was below the loading points, where hinges had formed (as seen with panel B), and due to the rotation of the panel at failure. No cracking was found at the ends of the panels, where deformation of the shear connectors is greatest. Upon deconstructing the panels there was no evidence of plastic deformation of the screws, only withdrawal from the joists.

Figure 6.21.: Cracking at load points (left-panel C) and hogging cracking induced by rotation of panel at failure (right-panel E)

6.6.3. Shear Connection Efficiency

The efficiency of each panel’s shear connection system at SLS=2.5kN/m² is defined by equation 6.7 and recorded in Table 6.9. Connection efficiency was greatest for panels with highest smeared slip stiffness, as defined by equation 6.10. The percentage difference between the most efficient (panel D) and least efficient (panel C) shear connection system was 20.2%, which was higher than expected, since the smeared slip stiffness of panel D was only 5% higher than panel C, and according to the \( \gamma \)-method does not merit such a difference in behaviour. Consequently, whilst the bending stiffness of panels C and E at SLS were very similar, the bending stiffness of panel D was significantly higher (Table 6.8 and Figures 6.16, 6.17 and 6.18).

\[
k = \frac{K_{0.4}}{a}
\]

(6.10)

Where:

- \( K_{0.4} \) is the slip modulus of the connectors at 40% of their maximum load
- \( a \) is the spacing of the connectors

For panel C at SLS (2.5kN/m²) the upgrade reduced the mid-span deflection from span/219 to span/354 (61% increase in bending stiffness), for panel

134
Table 6.9.: Shear connection efficiency at SLS load of 2.5kN/m²

<table>
<thead>
<tr>
<th></th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Efficiency (%)</td>
<td>49.5</td>
<td>69.7</td>
<td>57.3</td>
</tr>
</tbody>
</table>

D at SLS the upgrade reduced the mid-span deflection from span/199 to span/428 (114% increase in bending stiffness) and for panel E at SLS the upgrade reduced the mid-span deflection from span/189 to span/343 (81% increase in bending stiffness).

### 6.6.4. Analytical and Experimental Correlation

Table 6.10 presents the measured and predicted mid-span deflection, end slip, end separation between the joists and topping at the serviceability limit state load (2.5kN/m²). The mid-span deflection, end slip and vertical separation of timber and topping for each panel with partial connection was calculated using: the material properties presented in Tables 6.1 and 6.2 and 6.3; the geometric properties presented in Figures E.1, of Appendix E, and Equations of Appendix D. The correlation between the analytical $\gamma$-method prediction and experimental mid-span deflection was poor. The agreement for panels C, D and E were -31%, -17% and -29% respectively. In all cases the analytical approach was non-conservative, over predicting the stiffness of all three panels. When predicting the end slip the correlation between analytical and experimental results was worse than for the mid-span deflection. For panels C, D and E the error between analytical and experimental results was -33%, -36% and -31% respectively. As with the mid-span deflection, the analytical method under predicted the end-slip of all the panels. Agreement between the analytical approach and experimental results for the separation of the topping and joists at the SLS was least satisfactory. For panels C, D and E the error was -60%, -55% and -15% respectively.

The disagreement between the predicted and experimental values in these tests is considerable. This is surprising, since there is substantial literature evidence to suggest that the $\gamma$-method is an accurate method of predicting the effective bending stiffness of timber-concrete composite panels including: Ceccotti (2002); Fragiacomo & Lukaszewska (2011); Fragiacomo (2012); Persaud & Symons (2005); Yeoh (2010). Nonetheless there are a number of possible factors which may have contributed to the error which will now be discussed.

The first factor is the non-linear load-slip behaviour of the shear connectors. The $\gamma$-method uses a single slip modulus value to describe the load-slip relationship of the connectors at serviceability and therefore does not account
Table 6.10.: Comparison between experimental and analytical behaviour at SLS (2.5kN/m²)

<table>
<thead>
<tr>
<th></th>
<th>Exp.</th>
<th>Analyt.</th>
<th>Err (%)</th>
<th>Exp.</th>
<th>Analyt.</th>
<th>Err (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm)</td>
<td></td>
<td></td>
<td>(mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>δ</td>
<td>12.7</td>
<td>8.8</td>
<td>-31%</td>
<td>10.5</td>
<td>8.7</td>
<td>-17%</td>
</tr>
<tr>
<td>s&lt;sub&gt;end&lt;/sub&gt;</td>
<td>0.58</td>
<td>0.39</td>
<td>-33%</td>
<td>0.62</td>
<td>0.40</td>
<td>-36%</td>
</tr>
<tr>
<td>s&lt;sub&gt;sep&lt;/sub&gt;</td>
<td>0.97</td>
<td>0.39</td>
<td>-60%</td>
<td>0.62</td>
<td>0.28</td>
<td>-55%</td>
</tr>
</tbody>
</table>

for non-linear behaviour. Fortunately the error due to this factor at SLS is small in these tests; 5-6%.

The second factor is shear deformation of the layers of the beam. An assumption of the γ-method is that the Euler–Bernoulli theory of plane sections of each individual layer remaining plane applies, which in turn means that only bending deformations are considered and the shear deformation of the layers (not the shear connector) are disregarded. This is a particular problem when analysing composite beams with a span/depth ratio less than 10 to 15. In these cases instead of using the Euler-Bernoulli approach (γ-method/partial interaction theory) shear deformations can be accounted for by using Timoshenko Beam theory. However Schnabl et al. (2007) demonstrated that for composite timber-timber beams (E/G ratio greater than 16) the error between the two theories is insignificant. As the span/depth ratio of the 4.5m panels is 21.7 the difference between the theories would be at most 1-2%.

The third factor is vertical separation of the topping from the timber joist. One of the assumptions of the γ-method is that vertical separation of the layers does not take place and consequently the curvature of the layers is equal (Adekola, 1968). Whilst the error between the longitudinal separation of the timber and topping (end slip) is relatively large, for panels C and D there is greater vertical separation than would be expected from withdrawal of the connectors alone. This implies that the layers are separating and the assumption that the parts have equal curvature is violated. Without attempting to modify the existing analytical method to allow for this effect it is difficult to estimate the influence that it has on the result. But considering that very small magnitudes of longitudinal slip causes a large loss in
composite action (Figure 6.19), it would not be surprising if only tenths of millimetres of vertical separation also caused significant loss of composite action.

6.7. Type 3 Panels Test Results

The results of short-term bending tests conducted on panels cut from floor F are presented and discussed in this section. In total four panels were cut from the floor and were labelled according to their joist position in the floor (Figure 6.22), e.g. F1-F2 corresponds to joists 1 and 2. All the panels apart from panel F5-F6 had cracked topping. The cracking of each panel is summarised in Table 6.11 and illustrated in Figure 6.22.

<table>
<thead>
<tr>
<th>Panel ID</th>
<th>Crack Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-F2</td>
<td>One 320mm long crack above joist 2 at one end of the panel, and one crack, curved in plan, which traversed the width of the panel, starting on one edge at 800mm along the length of the panel and reaching the other edge at 1450mm along length of the panel.</td>
</tr>
<tr>
<td>F3-F4</td>
<td>One 400m long crack above joist 3 at one end of the panel, and one crack, curved in plan, which crossed one corner of the panel, starting on one edge at 1450mm along the length of the panel and continuing to where it finished above joist 4.</td>
</tr>
<tr>
<td>F5-F6</td>
<td>No cracking</td>
</tr>
<tr>
<td>F7-F8</td>
<td>One crack starting above joist 7 and moving towards joist 8 by 100mm over its 550mm length</td>
</tr>
</tbody>
</table>

The cracks, caused by accidentally inducing hogging into the floor, visibly extended through the entire depth of the topping. They were square in profile (Figure 6.23) and no attempt was made to precisely measure their width, since they were less than 0.25mm wide. The purpose of the test programme was two-fold, on the one hand to understand how the stiffness of TCC floor panels changes as they undergo cyclic loading within the service range and on the other to establish whether a cracked topping influences the performance of TCC panels.
Figure 6.22.: Floor F crack pattern (all dimensions in mm)

Figure 6.23.: Crack profile
6.7.1. Influence of Cracked Topping

The load mid-span deflection behaviour of the panel is presented in Figures 6.24 and 6.25. To aid clarity only the first cycle of each set of load cycles is plotted and since the measured bending stiffness of the timber joists varied between 9.5 kN/mm² and 15.3 kN/mm² (Table 7.1), the mid-span deflection of each panel has been normalised, according to Equation 6.11, to compare the panel performance. This method of normalisation is appropriate as the density of the timber, and consequently the modulus of elasticity of the timber, was found not to significantly influence the serviceability slip modulus of the connectors in Chapter 4. The theoretical deflection of the normalised panels with full and zero connection are plotted alongside the normalised results.

\[
w_{\text{norm}} = w \cdot \frac{E_{I_{ef}}}{E_{I_{ef,11000}}} \tag{6.11}
\]

Where:

- \( w_{\text{norm}} \) is the normalised deflection
- \( w \) is the experimental mid-span deflection
- \( E_{I_{ef}} \) is the theoretical effective bending stiffness of the panel according to partial interaction theory
- \( E_{I_{ef,11000}} \) is the theoretical effective bending stiffness of the panel if the timber joists had a Modulus of Elasticity of 11,000N/mm².

Figure 6.24 shows that load-displacement behaviour of all the panels is broadly similar and there is no suggestion that the cracking of the topping influenced the performance of the panels. Indeed when studying the mid-span deflection of the panels at a load of 2.5kN/m² it is found that the coefficient of variance of the panels is only 5.4%. Further evidence to support this conclusion is found when the effective bending stiffness of the panels is plotted for load cycles throughout the tests.

6.7.2. In service Loading

Figure 6.26 presents the effective bending stiffness of each panel during the first, second and final cycle of each set of cycles of loading. For example cycle number 22 is the second cycle of set C (set of loading refers to the loading protocol illustrated in Figure 6.9). The plot illustrates that in general there is good agreement between the panels as to how the effective bending stiffness changes during the cycles of loading and that the cracking of the
topping did not influence the performance of the panels. If this conclusion is correct it implies that, the connectors bridged the cracks that were present in the topping at the beginning of the test and as the panels were loaded the cracks closed together allowing compressive forces to be transferred across the cracks.

Figure 6.26 also demonstrates that, as for panels C, D and E the effective bending stiffness of the panels is not constant during the service loading of the panels. As the loading increased during sets A-C and D-F the effective bending stiffness of the panels increased. A similar non-linearity is observed in the load-slip behaviour of the connectors during the pushout tests. For example, the stiffness of the connectors increased by 20%, when the pushout specimens were loaded from 40% to 60% of the maximum connector load. This non-linear behaviour was also evident below 40% of the maximum load (Figure 4.12 in Chapter 4), which accounts for the change in stiffness of the panels. The second cycle of loading of set A has a greater stiffness than cycle one and the increase was greater than for any of the other sets of loading cycles. In comparison the differences between the stiffness of the panel during the second and tenth cycles of loading of sets A-F are within the expected variance.

Following the loading cycles of set C the load was reduced to the load
level of set A (see Figure 6.9). At the same time the bending stiffness of the panels reduced to approximately the same stiffness, as observed during the first cycle of set A. This result is surprising, since it would be expected that the stiffness of the panels would return to the stiffness of cycles 2-10 of set A. The initial increase in stiffness from the first to second cycle is usually attributed to the overcoming of an initial displacement caused by manufacturing inconsistencies, such as how tightly the connectors are fitted (Dias et al., 2010). Following the first cycle of loading of set D the increase in bending stiffness during sets of cycles D-F is more gradual, but more consistent, than sets A-C. However, the stiffness during sets of cycles D and E are lower than sets A and B respectively. Only in set of load cycles F, does the bending stiffness match the bending stiffness, experienced at the same load in previous sets of load cycles.

6.7.3. Shear Connection Efficiency

The efficiency of each panel’s shear connection system is defined by equation 6.7 and recorded in Table 6.9. The percentage difference between the most efficient and least efficient shear connection system was 13.1%. Apart from panel F7-F8 there is very good agreement between the efficiency of each
panel’s shear connection. The efficiency of the connection is not as high as for notched connections (90\% (Lukaszewska et al. 2008)) but higher than for simple nailed connections (35\% (Stearn 2006)). The lower efficiency of screw connections is offset by its simple installation and lower cost as compared to notches.

<table>
<thead>
<tr>
<th>Efficiency (%)</th>
<th>F1-F2</th>
<th>F3-F4</th>
<th>F5-F6</th>
<th>F7-F8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>80.7</td>
<td>75.1</td>
<td>75.8</td>
<td>67.6</td>
</tr>
</tbody>
</table>

6.7.4. Analytical and Experimental Correlation

The mid-span deflection, end slip and vertical separation of timber and topping for each panel with partial connection was calculated using: the material properties presented in Tables 6.1 and 6.2 and 6.3; the geometric properties presented in Figures E.1, of Appendix E, and Equations of Appendix D. As with type 2 panels the prediction of the mid-span deflection does not correspond well with the experimental values. Compared to type 2 span panels, the errors for the mid-span deflection are smaller, between -5\% and -20\%, whereas the end slip errors are greater, between -8\% and -31\%.
Table 6.13.: Comparison between experimental and analytical behaviour at SLS (2.5kN/m²)

<table>
<thead>
<tr>
<th></th>
<th>F1-F2</th>
<th>F3-F4</th>
<th>F5-F6</th>
<th>F7-F8</th>
</tr>
</thead>
<tbody>
<tr>
<td>δ (mm)</td>
<td>4.24</td>
<td>4.04</td>
<td>-5%</td>
<td>5.21</td>
</tr>
<tr>
<td>s_{end} (mm)</td>
<td>0.30</td>
<td>0.25</td>
<td>-17%</td>
<td>0.26</td>
</tr>
<tr>
<td>s_{sep} (mm)</td>
<td>0.39</td>
<td>0.18</td>
<td>-55%</td>
<td>0.33</td>
</tr>
</tbody>
</table>

6.7.5. Non-linear Stiffness

The load-displacement behaviour of these panels displayed significant non-linearity, similar to type 2 panels. The increase between the effective bending stiffness, at SLS (2.5kN/m²), and the peak stiffness was between 36% and 46% and the load at which the peak stiffness occurred was between 38% and 59% of the maximum load of the panel (Table 6.14). Following this peak in stiffness there was a short period of linear behaviour before the panels lost composite action due to withdrawal of the screws from the timber joists. All the panels sustained a period of deflection without composite action apart from panel F3-F4. The magnitude of the mid-span deflection which occurred without composite action before failure of the panel, was determined by the bending strength of the individual joists that failed and was not a function of the topping or connectors. The mid-span deflection attained without composite action before failure ranged between 0mm for F3-F4 to 16mm for F1-F2.

Table 6.14.: Panel effective bending stiffness

<table>
<thead>
<tr>
<th></th>
<th>F1-F2</th>
<th>F3-F4</th>
<th>F5-F6</th>
<th>F7-F8</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS (EI_f) ((10^9 \text{Nmm}^2))</td>
<td>380</td>
<td>305</td>
<td>340</td>
<td>355</td>
</tr>
<tr>
<td>(EI_{f,max}) ((10^9 \text{Nmm}^2))</td>
<td>520</td>
<td>455</td>
<td>480</td>
<td>520</td>
</tr>
<tr>
<td>Percentage Change</td>
<td>36%</td>
<td>49%</td>
<td>40%</td>
<td>46%</td>
</tr>
<tr>
<td>(F_{f,\text{max}} / F_{\text{max}})</td>
<td>0.38</td>
<td>0.38</td>
<td>0.46</td>
<td>0.59</td>
</tr>
</tbody>
</table>
6.7.6. Failure

All four panels failed by tension of a single joist at a prominent knot location (Figure 6.27). As with type 2 panels, the failure was sudden and brittle, causing the panels to rotate at the centre and the topping to crack. As the panels were unstable, the test was stopped and the panels unloaded. Before failure, there was no indication of cracking of the topping, apart from those cracks which were present at the start of the test. The failure loads of panels F1-F2, F3-F4, F5-F6, F7-F8 were 21.7, 18.3, 18.1 and 20.6 kN/m². The factor of safety on the following design loads is calculated by the methods presented in section 6.6.2. If the characteristic imposed load, assuming an office is 2.5 kN/m² (BSI, 2002) and taking the panel with the lowest failure load, the factor of safety on the design load is 2.3. If the panel was part of a floor in a residential building, the characteristic domestic imposed loading would be 1.5 kN/m² and the factor of safety on the panel with the lowest failure load would be 3.69. Assuming that a factor of safety on design loads of two is adequate then these panels have demonstrated adequate performance for both office and residential loading.

![Figure 6.27: Failure of panel F1-F2 (left) and recovery of panel F5-F6 (right)](image)

6.7.7. Ease of Reversibility

At the end of each short-term bending test the panels were dismantled quickly, (less than 15 minutes per panel with three workers) using simple demolition tools. The screws at the end of the panels were examined and it was found that there was no appreciable rotation of the screw heads and minimal damage to the surface of the joist (Figure 6.28). Damage of the joist surface was thought most likely to have occurred during installation of the screws, as the point of the screw bores into the joist, rather than from pullout of the screws. From the point of view of reversibility, the
panels demonstrated good performance, causing minimal damage and a quick deconstruction time.

![Connectors following failure of panel and damage to joists at connector insertion point.](image)

Figure 6.28.: Connectors following failure of panel and damage to joists at connector insertion point.

### 6.8. Concluding Comments

A total of nine panels have been subjected to short-term bending tests. All the panels were tested using six equally spaced line loads which effectively mimic a uniformly distributed load. A series of different loading protocols were used to investigate:

- the increase in panel bending stiffness;
- the linearity of the load-displacement behaviour;
- correlation between the analytical $\gamma$-method presented in Annex B of EN1995-1-1 and experimental results;
- the effects of cycling the load on the effective bending stiffness of the panels;
- the effect of cracked topping on the performance of the panels
- the failure mode.

As a result of the upgrade, all the panels increased in bending stiffness. The upgrade was most efficient for joists with the least depth, increasing the stiffness by up to 298%. For joists 170mm deep an increase of up to 114% in bending stiffness was attained allowing the floor to achieve an instantaneous span/300 deflection at 2.9kN/m². This increase in bending stiffness is sufficient to accommodate change of use from residential (1.5kN/m² imposed load) to office occupancy (2.5kN/m² imposed load).

A significant non-linear load-mid-span deflection relationship was observed during the tests. In a large proportion of the tests the panels stiffened between
the serviceability load (2.5 kN/m²) and 40-60% of the maximum load. Panels with connectors spaced according to the shear distribution displayed no greater non-linearity than the panels with evenly spaced connectors.

Correlation between the popular, analytical, $\gamma$-method predictions and experimental results were reasonable with respect to SLS deflection but poor with regards to end slip. Error between the analytical and experimental mid-span deflection ranged between 5% (panel F1-F2) and 31% (panel C). Conversely to the results of type 2 and type 3 panels, the preliminary tests demonstrated conservative agreement between theory and experiment 16% and 11% error.

For type 3 panels the load was cycled sixty times and served to confirm the non-linear behaviour observed in type 2 panels. The testing method also highlighted that the effective bending stiffness of the panels deteriorated when the load is cycled at a load level below the maximum load level previously experienced by the panel. However the nature of the test setup (hydraulic hand pump) precluded a longer test duration with greater number of cycles and consequently it is not properly understood how the panels will behave after thousands of cycles of loading for example. In this respect the conclusions are limited in their scope and application. Future work should seek to improve the understanding of how the bending stiffness of panels changes during in service loading.

The panels demonstrated robustness, since cracks in the topping were found not to affect the short-term bending performance of the panels throughout the duration of the test. Panels that were cracked performed equally as well as those without cracking.

At higher loads the connectors withdrew from the joists, causing greater interfacial slip and mid-span deflection. Depending on the strength of the individual joists, the panels deflected between 0 and 25 mm without composite action before failure. Final failure was of the joists by combined tension and bending following the loss of composite action. Although the topping cracked at failure it is thought to be a function of the test rather than a behaviour of the system. No cracking was observed in the topping prior to failure.
7. | Floor Tests

The vibration and flexural testing of timber floors upgraded with a thin cementitious topping are documented in this chapter.

Complete timber-concrete composite floor tests are rarely conducted because most research programmes focus on the strength and stiffness behaviour of the composite systems, which can be evaluated by testing panels (Fragiacomo (2012); Mertens et al. (2007) are notable exceptions). Whilst panels are a cost and time effective method of investigating some aspects of composite behaviour, complete floors provide the opportunity to study load sharing effects and dynamic behaviour. As the dimensions and edge support conditions of the floor influence its vibration response, the vibration testing of panels cannot be a substitute for the testing of floors.

The results of two floor tests are presented in this chapter, a preliminary test floor spanning 2.8m and 3.2m wide (Floor F) and a full scale floor spanning 4.57m and 4.4m wide, tested at the Building Research Establishment (BRE floor). Floor F was tested dynamically before being cut into panels, each comprising of two joists, which were subjected to short-term bending tests to failure. The short-term bending test results along with the characterisation of the joists are reported in Chapter 6 alongside the other panel testing.

The objectives of the vibration testing programme were to:

- evaluate the change in vibration performance with the addition of the topping upgrade with respect to modal frequencies, damping ratios and accelerations;
- compare the vibration performance of the timber and TCC floors using EN1995-1-1 design criteria.

The objectives of the static testing programme were to:

- assess how the topping changed the load sharing characteristics between joists;
- assess the behaviour of the floor at serviceability during repeated cycles of loading;
- compare serviceability performance with the analytical $\gamma$-method predictions;
• compare floor short-term bending performance with previous panel testing

The chapter is structured with the following sections: in section 7.1 the BRE floor construction is described; in section 7.2 the experimental and analytical procedures are described; in section 7.3 the monitoring of the floor during the upgrade is reported; in section 7.4 the vibration testing of Floor F is reported; in section 7.5 the vibration testing of the BRE floor is presented; in section 7.5 the elastic testing of the BRE floor is presented; in section 7.7 the short-term bending test of the BRE floor to failure is reported and in section 7.8 some concluding comments are provided.

7.1. Floor Construction

Prior to the construction of the BRE test floor a preliminary floor was constructed and tested (Floor F). Following dynamic testing of this floor four panels were cut from the floor and subjected to short-term bending tests. The details of the short-term bending tests and the floor construction are presented in Chapter 6. This section discusses the construction of the BRE test floor.

C16 graded timber joists, 44mm wide and 195mm deep were subjected to an elastic 4-point bending test (Figure 7.1) to establish the global modulus of elasticity of each joist. Loads of 0.21 kN were applied at third points, whilst the deflections at the mid-span and loading points were measured visually, before and after the loads were applied, using dial gauges. The values are listed in Table 7.1 alongside the global modulus of elasticity and the coefficient of variation. BS EN 338:2009 provides a mean modulus of elasticity parallel to the grain of 8kN/mm² and a characteristic 5% modulus of elasticity parallel to the grain of 5.4kN/mm², which this sample of joists exceed. The mean density and moisture content were established from six specimens, sampled form a single joist following the failure test; BS EN 13183-1:2002 was used to determine the moisture content. The mean density was 398.8kg/m³ and the coefficient of variation was 5%, whilst the mean moisture content was 15.8% with a coefficient of variation of 0.5%. The joists were distributed so that the stiffest joists were at the centre of the floor and numbered according to their location within the floor.

The mean flexural and compressive strength of the topping was established from 40 x 40 x 160mm prisms which were stored in the same environment as the test floors whilst they cured. They were tested according to BS EN 1015-11 1999 ; in flexure the specimens were loaded at a constant displacement
rate of 0.2mm/min and in compression at 0.5mm/min. The mean values are reported in Table 7.2. The proprietary topping was delivered as dry ingredients, which were sieved, to establish that the maximum aggregate size was between 2 mm and 4 mm. The density of the topping was 2200kg/m³ and for the purposes of the analytical study the modulus of elasticity was assumed to be that measured from the cylinder tests reported in Chapter 6. The mass per unit area of the timber floor components are listed in Table 7.3.

Table 7.2.: Topping mean flexural and compressive strength

<table>
<thead>
<tr>
<th>Mean¹ Flexural Strength (N/mm²)</th>
<th>Mean² Compressive Strength (N/mm²)</th>
<th>CV (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast 1 8.1</td>
<td>56.1</td>
<td>6</td>
</tr>
<tr>
<td>Cast 2 8.6</td>
<td>49.9</td>
<td>6</td>
</tr>
</tbody>
</table>

¹mean of three specimens ²mean of six specimens

Figure 7.1.: Four-point bending elastic characterisation of timber joists

The construction sequence of the floor was broadly similar to the panel construction described in Chapter 6. In addition to the previous construction,
Table 7.3.: Mass of the timber floor components

<table>
<thead>
<tr>
<th>Description</th>
<th>Mass (kg/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18mm Thick Particleboard</td>
<td>7.2</td>
</tr>
<tr>
<td>195x44mm joists @400mm c/c</td>
<td>9.2</td>
</tr>
<tr>
<td>Plasterboard</td>
<td>8.3</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>24.7</strong></td>
</tr>
</tbody>
</table>

12mm thick plasterboard was fixed to the underside of the floor with 4mm diameter drywall fixings spaced at 400mm centres. Noggins were fixed between joists at the centre and ends of the joists to provide additional transverse stiffness to the floor and supplementary locations for fixing the particleboard deck and plasterboard soffit. A plan and elevation of the BRE floor can be found in Appendix E.

The topping was mixed in a 150 ltr capacity pan mixer, mixing 150 ml of water for every 1kg of dry ingredients. It was then placed on the floor to a depth of approximately 19.5mm using formwork and timber battens as guides. The topping was levelled by agitating the wet screed by hand and then trowelling to a level finish. Unlike the construction of the panels, a plastic sheet was not used to help retain moisture following placing of the topping. Whilst the topping was placed the temperature and relative humidity of the lab were 17.4°C and 54% respectively. The topping was allowed to cure for ten days before vibration testing commenced.

Figure 7.2.: Upgrade process: Floor prepared with plastic sheet laid, screw heads protruding and topping placement against battens
7.2. Methodology

7.2.1. Construction Monitoring

Following placement of the topping, the mid-span deflection of each floor joist and the strains of joists seven and ten were monitored for ten days, during which time the temperature and relative humidity of the laboratory were recorded. TML strain gauges were attached at the top, centre and bottom of the joist (Figure 7.3 for positions) to measure the longitudinal strains of joists seven and ten. The strain gauges were 60mm long, wire strain gauges with a 2% strain limit. Each joist was prepared by sanding the surface of the joist before fixing the gauges to the joists using a single component, centrality, adhesive.

![Figure 7.3: Longitudinal strain gauge positions: top, centre and bottom of joist](image)

The purpose of monitoring the floor during and after the construction period was to investigate the consequences of unpropped construction. It was of particular interest to find the initial deflection and the deflection ten days after construction, which would give an indication of the necessity of propping to minimise long-term deflections. During construction, the floor was supported at the joist ends but was unsupported along the other edges. Draw-wire transducers, accurate to 0.1mm, positioned at the centre of each joist, measured the deflection of the floor, whilst temperature and relative humidity sensors monitored the environmental conditions of the laboratory. The results are presented in section 7.3.
7.2.2. Vibration Tests

Several methods were used to excite the floors, impact hammer excitation, revolving mass excitation and a mass release method. Each method will be briefly discussed in the following sections.

7.2.2.1. Impact Hammer Excitation

Impact hammer excitation is commonly used to excite floors, as it is one of the quickest methods of capturing the vibration response of floors (Ohlsson, 1982; Bernard, 2008; Ghafar et al., 2010; Lukaszewska & Fragiacomo, 2010; Rijal et al., 2010). In these tests, a hammer with a load cell was used to strike the floor (Figure 7.4) and the resulting accelerations were measured at discrete points using accelerometers. The positions of the accelerometers and locations at which the floor was struck are illustrated in Figure E.8, in Appendix E.

![Figure 7.4.: Impact hammer, hammer tips and amplifier](image)

The frequency spectrum of the force input was determined by a combination of the stiffness of the hammer tip and the surface being struck. A relatively low stiffness material was required to create a broader force input peak, to translate to a lower frequency input; more akin to footfall and more likely to excite the low frequency response of the timber floors.

To assess which combination of hammer tip and striking surface material was most appropriate, four hammer tips: hard (H), medium (M), soft (S)
and super soft (SS), and three strike surfaces: bare floor (1), rubber pad (2) and foam (3), were compared. Figures 7.5 and 7.6 provide a time domain comparison of the strike surface and hammer tip. Figure 7.5 shows that surface type 3 provided the broadest force input whilst the other surfaces provided very similar force inputs. The hammer tip (Figure 7.6) had no effect on the breadth of the force input peak; the change in magnitude of the force input was not influenced by the hammer tip, rather it was caused by the application of the force by the operator.

![Figure 7.5: Strike surface comparison: supersoft tip](image1)

![Figure 7.6: Hammer tip comparison: foam surface](image2)

By their very nature, impulse force inputs are short in duration and a sampling rate higher than is necessary to study the frequencies of interest, is required. Coupled with a short duration of data capture, problems of
insufficient resolution in the frequency domain can arise when the data is transformed. To assess the best compromise between a sampling rate high enough to capture the force input and a rate slow enough to have sufficient resolution in the frequency domain, hammer impacts sampled at 500Hz and 2000Hz were investigated. Figure 7.7 shows that increasing the sampling rate from 500 Hz to 2000 Hz increased the number of data points capturing the force input, providing greater clarity and breadth to the force input than at 500 Hz. However, following further studies it was decided that sampling data at 1000 Hz was the best compromise.

![Figure 7.7.: Sampling rate comparison: for supersoft tip on bare floor](image)

This problem of insufficient resolution can be confounded by the effects of electrical noise and the limitations of instruments at small accelerations, which can also contribute to a poor quality frequency response function (FRF). These effects were reduced by averaging the FRFs of six individual impacts at the same location. Conducting multiple hammer strikes was considered necessary but reduced the speed of the test.

Applying the impact to the floor required a skilled operator to avoid double impacts. If a double impact did occur then the resulting Fourier transforms of the impact superimposed on one another could cancel each other out, causing significant error in the mobility at the frequencies of coincidence (ISO, 1994). To avoid this, each impact was individually assessed and records with double impacts were discarded.
7.2.2.2. Revolving Mass Excitation

There are multiple continuous excitation methods available for exciting floors, the excitation method available for the floor test was a revolving mass shaker (Figure 7.8). The shaker consisted of a step motor and a shaft driving two rotating masses, one clockwise and the other counter clockwise. The resultant of the two forces was sinusoidal, the amplitude of which was calculated by Equation 7.1:

\[ F = mr\omega^2 \]  \hspace{1cm} (7.1)

where:
- \( F \) is the load amplitude.
- \( m \) is the mass of the revolving masses.
- \( r \) is the lever arm, the perpendicular distance between the centroid of the masses and the centre of the drive shaft.
- \( \omega \) is the angular frequency.

The frequency of the sinusoidal force input was controlled by a stepping signal or a controller set at a discrete frequency. This type of shaker can be used to perform the well known sine sweep procedure. In this procedure the frequency of the force input is increased or decreased at a constant rate giving equal weighting to each frequency. As the forcing frequency coincides with the modal frequencies of the floor, the floor resonates. The resulting acceleration response is shorter and sampled a lower rate than with an impact hammer, resulting in a more accurate FRF. However the amount of data collected is usually large, requiring substantial memory which is not always
available. This is because a slow sweep rate is required to avoid distorting the FRF, affecting the frequency and damping ratio. The suggested maximum linear sweep rate through a resonance (ISO, 1990) is:

\[ s_{\text{max}} \leq 216 f^2 \zeta^2 \]  

(7.2)

where:

- \( s_{\text{max}} \) is the maximum sweep rate (Hz/min).
- \( f \) is the resonant frequency.
- \( \zeta \) is the equivalent viscous damping ratio.

The exciter was driven by a stepping motor through a drive shaft and uses several drive belts to rotate the masses. Unfortunately the exciter was limited in the frequency of excitation that it could apply, since the drive belt slipped between 36 Hz and 40 Hz. Another concern was that the force input was proportional to the square of the forcing frequency and consequently more than one sine sweep, with different sized masses, would be required to capture the response of the floor without causing irreparable damage from resonance. Another disadvantage of this method was that the exciter has to be attached directly to the floor, which gave the floor additional mass. This was of particular concern as:

1. timber floors have relatively little mass so the mass of the shaker would provide significant inertia, influencing the response.
2. the test programme was concerned with the change in performance between two floor systems with very different masses. The influence the mass of the shaker exerted on the upgraded floor would be less than for the timber floor.

### 7.2.2.3. Mass Release Excitation

The mass release excitation method was arguably the simplest test method available. A mass was hung from the underside of the floor by a wire and when the wire was cut the floor rebounded and vibrated freely. As with the impact hammer test, the resulting vibration response was of a short duration, but unlike the impact hammer test, mathematically the force remains applied once the wire is cut. The transform of this force input was very different to an impact force input as it increased linearly with frequency. This has the effect of amplifying the response at the higher frequencies in the FRF, which was undesirable. Care was taken to avoid inadvertently applying a force to the floor when cutting the wire suspending the mass.
7.2.2.4. Experimental Method

For Floor F the mass release method was chosen, whilst for the BRE floor the impact hammer method was used. The modal frequencies of both floors were identified from the frequency response function (FRF), whilst the mode shape of each modal frequency was identified separately using a revolving mass exciter.

During the testing of floor F, data were sampled at 1000 Hz and the mass was released from a slightly off-centre position. Accelerometers, placed at the predicted locations of anti-nodes, measured the resulting accelerations. During the BRE floor test, data were sampled at 1000 Hz so as to capture the force input which was applied at three positions on the floor, designed to excite all modes below 40 Hz. Accelerometers, placed at the predicted location of nodes and anti-nodes of modes below 40 Hz, measured the resulting accelerations. The positions at which the impacts were applied and the locations at which accelerometers were placed are illustrated in Figure E.8 of Appendix E.

In both floor tests the modal frequencies were identified from the FRF, whilst the mode shape of each modal frequency was identified separately using the revolving mass exciter. The frequency of the revolving mass exciter force input was set to the modal frequency of interest and whilst one accelerometer’s position was fixed, a second was used to identify the magnitude and phase of the accelerations at 35 locations on the floor. With the relative phase and amplitude of the accelerometers established at each location, the resulting 'mesh' of measurements was used to describe the mode shape of each modal frequency.
Two support conditions were considered; simply supported on two sides and simply supported on four sides. The pinned supports alongside the outside joists were created by fixing a second joist along the length of the outside joist which in turn was restrained from moving vertically at third points by a bolted connection (Figure 7.10).

7.2.2.5. Analysis Methodology

A FRF is a transfer function, which is a ratio of inputs (force) to outputs (acceleration, velocity or displacement). FRFs were constructed for each measured response by transforming both the input and the output to the frequency domain using the fast Fourier transform method. The modal frequencies and modal damping ratios were extracted from the FRFs using the single degree of freedom (SDOF) circle-fit method. At resonance the plot of the imaginary part of the FRF against the real part of the FRF (Nyquist plot, Figure 7.11) tends towards a circle, from which the modal parameters are found. The type of damping being evaluated determines the type of FRF needed for analysis (Ewins, 1984), for viscous damping the mobility plot is used.

The SDOF circle fit analysis procedure followed the basic sequence below:
Figure 7.11.: Nyquist plot at a modal frequency

- The points to be analysed for each modal frequency were selected by hand.

- A circle was fitted to the selected data using a least-squares method and the radius and coordinates of the centre of the circle were identified.

- Each modal frequency was found by identifying the maximum 'sweep rate' around the circle. The angles between successive data points, formed from radial lines from the centre of the circle to the data points, were found and the location of the maximum 'sweep rate' (the least dense distribution of data points around the circle) corresponding to the modal frequency ($\omega_n, 7.11$) was identified.

- The viscous damping ratio of each mode was identified by considering the angles, $\theta_x$ and $\theta_y$, between data points, $\omega_x$ and $\omega_y$, either side of the modal frequency (7.11). With this information the viscous damping ratios were calculated using the following equation:
\[ \zeta = \frac{\left(\omega_y^2 - \omega_x^2\right)}{2\omega_n \left\{ \omega_y \tan \left(\frac{\theta_y}{2}\right) + \omega_x \tan \left(\frac{\theta_x}{2}\right) \right\}} \] (7.3)

If \( \theta_x = \theta_y = 90^\circ \), equation 7.3 reduces to the popular half-power bandwidth method equation:

\[ \zeta = \frac{\omega_y - \omega_x}{2\omega_n} \] (7.4)

- The process of comparing the angle between points either side of the modal frequency was repeated and a mean value obtained.

By utilising this technique it is assumed that a resonant frequency is dominated by one particular mode. However, the other modes do exert an effect on the circle, but only to displace it from its original position as the combined effect is represented as a constant offset applied equally to all points around the circle (Ewins, 1984). Nonetheless when modes have little separation then the method becomes less accurate as there is insufficient differentiation between modes in the FRF and consequently an inadequate number of data points to form a circle.

### 7.2.3. Elastic Tests

The elastic stiffness of the timber floors were assessed for both support conditions by applying a 0.6 kN/m line load across the centre of the floor. The load was achieved by placing 25kg bags at 400mm intervals (Figure 7.12). In addition, the stiffness of the BRE timber and upgraded floor due to a 1kN point load was assessed by placing four 25kg bags at the centre of the floor. In each test the mid-span deflection of every joist was measured using draw-wire transducers, with a precision of 0.1mm.

![Figure 7.12.: Line load and central 1 kN load](image)
7.2.4. Short-term Bending Tests to Failure

Following the completion of the vibration tests, the floor was subjected to short-term bending tests. The floor was loaded with nine hydraulic jacks across six equally spaced spreader beams, approximating a uniformly distributed load (Figure 7.13). The advantages and disadvantages of this loading scheme are discussed in Chapter 6. The loading protocol is illustrated in Figure 7.14 and consisted of four sets of three subsets of five cycles of loading, 60 cycles in total. At the peak and trough of each load cycle the load was held for 30 seconds. The protocol was finished by loading the floor to 70% of the maximum load and then unloading to 10% of the maximum load before loading to failure.

Figure 7.13.: Loading apparatus and test arrangement

Figure 7.14.: Loading protocol: short term bending test
As with the panel tests a pinned-pinned support condition was used rather than a pinned-roller support.

Draw-wire transducers were positioned beneath the centre of each joist to measure the mid-span deflection. Displacement transducers were located at the ends of joists seven, ten and twelve at positions A, B and C (Figure 7.15). Transducers at A measured the end slip between the top of the joist and underside of the concrete topping ($s_{end}$), transducers at B measured the vertical separation of the topping from the timber joists ($s_{sep}$) and transducers at C measured the vertical separation of the interlayer from the chipboard.

![Figure 7.15.: Joist end transducer arrangement](image)

The results have been compared with predictions using the analytical $\gamma$-method. Comparisons of the mid-span deflection, end-slip and end separation are made at SLS=2.5kN/m². The $\gamma$-method is outlined in Appendix D and the equations for calculating mid-span deflection (equation 7.5), end-slip (equation 7.6) and end-separation (equation 7.7) are reproduced below.

\[
 w = \frac{5ql^4}{384 (EI)_{ef}} \quad (7.5)
\]

\[
 s_{end} = \frac{T(x)}{K} \quad (7.6)
\]

\[
 s_{sep} = \frac{T(x)}{K} \tan \phi \quad (7.7)
\]

\[
 T(x) = \frac{\gamma S}{z (EI)_{ef}} a(x) \quad (7.8)
\]
Where:

\( w \) is the mid-span deflection of the panel

\( q \) is the load per metre

\( l \) is the span of the beam

\( (EI)_{ef} \) is the panel effective bending stiffness

\( K \) is the slip modulus of the connectors

\( T \) is the shear load in a connector

\( z \) is the distance between the centroids of each part

\( a \) is the spacing of the connectors

\( \phi \) is the angle of the shear connector, measured from the horizontal

The slip modulus of the shear connectors used in the analysis was established from pushout tests as 4335N/mm.
7.3. Construction Monitoring

The strains of joist seven, for 24 hours following construction are presented in Figure 7.16, whilst the strains of joists seven and ten, for ten days following construction are presented in Figures 7.17 and 7.18 respectively. Each figure also includes the relative humidity and temperature of the laboratory environment where the floor was monitored. In Figures 7.16, 7.17 and 7.18 the noise observed during the first 120 minutes is attributed to construction workers standing on the test specimen whilst placing the topping. In these figures shortening of the joist (compression) is measured as a negative change in strain whilst elongation of the joist (tension) is measured as positive change in strain. Note that no offsetting of the initial strains has been made.

As with the panel monitoring in Chapter 6, it was found that the addition of wet screed caused the floor to instantaneously deflect, with an increase in compressive strains at the top of the timber joists and an increase in tensile strains at the bottom of the joist. Preceding this all the gauges indicated a small increase in compressive strain, perhaps indicating hogging of the floor due to sequentially placing topping from joist 1 to joist 12. Following the deflection of the floor due to the weight of the wet screed the floor rebounded, reversing the strains applied due to the wet topping. The possible reasons for this repeatedly observed phenomena are discussed in Chapter 6.

Following the instantaneous deflection of the floor, the floor underwent creep deflection. The deflection profile of the floor at three, seven and ten days is presented in Figure 7.19. The most striking feature is the non-symmetrical nature of the deflection profile. The joists are distributed so that the floor stiffness is approximately symmetrical therefore it can be only concluded that this aspect of the deflection profile is due to the combination of the sequential placement of the topping, from 0mm to 4400mm and the topping hardening at x=0mm before the topping at x=4400mm was placed. The highest mid-span deflection at ten days was 8.4 mm at joist four, a span to deflection ratio of 544.

This span to deflection ratio was almost four times higher than found by Fragiacomo (2012) for an unpropped floor at ten days (span to deflection ratio of 1910), although in that example the supports were built in, providing greater fixity, reducing the theoretical deflection by a factor of five for fully fixed supports compared to pinned supports. Having applied an imposed load of 2.5kN/m² to propped and unpropped floors Fragiacomo (2012) concluded that the beneficial reductions in deflection (instantaneous and creep) did not outweigh the inconvenience of propping during construction. As the support conditions in this test are pinned, rather than as found in
practice, it seems reasonable to conclude that on the basis of realistic support conditions, propping should be avoided if the ceiling finishes below the upgraded floor can tolerate the increased deflection.

### 7.3.1. Creep Calculation

As suggested above the thin topping upgrade could be applied without propping the floor. This is because the additional mass of the topping is relatively small compared to conventional, thick topping upgrades, which should lead to lower long-term creep. The extent of this advantage is assessed in this section by undertaking some basic calculations. A simple method known as the Effective Modulus Method has been used to evaluate the creep deflection of the BRE floor upgraded with the 19.5mm topping and a 50mm thick topping. Although this method is simplistic, in that it does not take into account the shrinkage of concrete or changing environmental conditions, it is suitable for providing a basic comparison and an overview of the method, outlined in Ceccotti (1995), is now provided.

All the components in the floor (topping, timber and connection) exhibit creep phenomena. In particular the creep behaviour of the timber and connection depend on the mechano-sorptive effect. To calculate the final
deflection of the floor an effective modulus of elasticity is used for each material and the stiffness of the connection reduced to account for the creep phenomena. These effective values are then used in conjunction with the $\gamma$-method to calculate the deflections. The following equations outline how the effective material properties are calculated.

\[ E_{c,\text{eff}} = \frac{E_c}{1+\phi_c}, \quad E_{t,\text{eff}} = \frac{E_t}{1+\phi_t}, \quad K_{s,\text{eff}} = \frac{K_s}{1+\phi_s} \]

$\phi_c$, $\phi_t$ and $\phi_s$ are the creep coefficients of the concrete, timber and connection respectively. Ceccotti (1995) suggests the values listed in Table 7.4 as suitable creep coefficients for floors in service class 1.

<table>
<thead>
<tr>
<th>Creep Coefficient</th>
<th>Permanent</th>
<th>Medium-term</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_c$</td>
<td>2.25</td>
<td>1.35</td>
</tr>
<tr>
<td>$\phi_t$</td>
<td>0.6</td>
<td>0.25</td>
</tr>
<tr>
<td>$\phi_s$</td>
<td>0.6</td>
<td>0.25</td>
</tr>
</tbody>
</table>

The values in Table 7.5, measured from the cross-section of the BRE floor, are used in the creep calculations.
Two topping thicknesses, 19.5mm and 50mm thick, two loading conditions, self-weight of the floor (permanent) and an imposed load of 2.5kN/m² (medium-term), and the propped condition of the floor during construction have been considered. The self-weight of the upgraded floor was calculated as 0.65kN/m² and 1.3kN/m² for 19.5mm and 50mm thick toppings. For the unpropped floor, the final deflection due to the permanent load was calculated by assuming that the topping did not contribute to the stiffness of the floor, whereas the final deflection due to medium-term load was calculated using an effective bending stiffness reduced by the effective modulus method. The final creep deflection \( w_{\text{final}} \) for each scenario is presented in Table 7.6 alongside the predicted instantaneous deflection of the timber floor due to the addition of the topping \( w_{\text{inst}} \).

When the floor is unpropped, the instantaneous deflection is 115% higher for the 50mm thick topping, than the 19.5mm thick topping, this is in proportion
to the additional load of the topping. In the case of the unpropped floor, the final creep deflection of the floor due to both permanent and medium-term loads is 36% higher for the floor with the 50mm thick topping than the floor with a 19.5mm topping. This is despite the deflection of the floor with the thicker topping being 35% less due to medium-term load than the floor with the thinner topping. The allowable total creep deflection recommended by EN1995-1-1 varies between span/150 and span/300. The limit chosen by the engineer will depend on whether brittle finishes are present. Presuming that brittle finishes are not present then the limiting deflection could be span/150 (30.5mm). In this case the final deflection of the floor with the 19.5mm thick

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**Figure 7.19.: Floor deflection profile at three, seven and ten days following construction**

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**Table 7.6.: Calculated instantaneous and creep deflections**

<table>
<thead>
<tr>
<th>Topping Thickness</th>
<th>$w_{\text{inst}}$ (mm)</th>
<th>$w_{\text{final}}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unpropped P</td>
<td>Unpropped M</td>
</tr>
<tr>
<td>19.5mm</td>
<td>7.8</td>
<td>14.1</td>
</tr>
<tr>
<td>50mm</td>
<td>16.8</td>
<td>28.4</td>
</tr>
</tbody>
</table>

$^P$ and $^M$ refer to permanent and medium-term load.
topping would be acceptable whereas the final deflection of the floor with a 50mm thick topping would be unacceptable.

In the case of the propped floor, the final deflection of the floor, having removed the props, would be 20% lower with a 50mm thick topping than a 19.5mm thick topping. This is because the final deflection due to the medium-term load is more significant than the final deflection due to the permanent load. As the effective bending stiffness of the floor is greater with the thicker topping, the deflection due to the medium-term load is lower. These results demonstrate that it is possible to upgrade floors with a 19.5mm thick topping without propping but that if upgraded with a thicker topping, propping is required.
7.4. Floor F Vibration Test Results

The results of the vibration testing of Floor F are presented in this section. Five test arrangements were analysed; timber and upgraded floors supported on two and four sides, and the floor with reduced breadth (2.4m x 2.8m) consisting of joists numbered three to eight, supported on four sides. Table 7.7 presents the modal frequencies and modal damping ratios for the various test arrangements. A re-analysis of the experimental data, since the first publication of this work (Skinner et al., 2013), with more advanced methods has found lower modal damping ratios for the first modal frequency than were previously reported. During analysis of the data, the mass release method of excitation, was found to be ineffective for exciting all of the modal frequencies below 40 Hz with sufficient energy to provide a reliable, measurable response. This method of excitation was best suited to exciting the fundamental frequency, rather than higher order modes. Some higher order modes which were expected, were absent from the measured response and those that were detectable, had their modal frequency and corresponding damping ratio measured with less than satisfactory precision, compared to the first mode.

7.4.1. Modal Frequencies

First order modal frequencies for the floors supported on four sides were predicted using equation 7.9:

\[
f_{1,j} = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}} \cdot \sqrt{1 + \left[ 2j^2 \left( \frac{l}{b} \right)^2 + j^4 \left( \frac{l}{b} \right)^4 \right] \cdot \frac{(EI)_b}{(EI)_l}}
\]  

(7.9)

Where:

- \( f_{i,j} \) is the modal frequency under consideration.
- \( j \) is the mode number perpendicular to the direction of the joists.
- \( l \) is the floor span.
- \( b \) is the floor breadth.
- \( m \) is the mass per unit area of the floor.
- \((EI)_l\) is the bending stiffness parallel to the direction of the joists (calculated).
- \((EI)_b\) is the bending stiffness perpendicular to the direction of the joists (calculated).
Table 7.7.: Modal frequencies and damping ratios Floor F

<table>
<thead>
<tr>
<th>Description</th>
<th>$f_{1,1}$ (Hz)</th>
<th>$\zeta_{1,1}$ (%)</th>
<th>$f_{1,2}$ (Hz)</th>
<th>$\zeta_{1,2}$ (%)</th>
<th>$f_{1,3}$ (Hz)</th>
<th>$\zeta_{1,3}$ (%)</th>
<th>$f_{1,4}$ (Hz)</th>
<th>$\zeta_{1,4}$ (%)</th>
<th>$f_{1,5}$ (Hz)</th>
<th>$\zeta_{1,5}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber supported 2 sides</td>
<td>20.4</td>
<td>1.8</td>
<td>24.1</td>
<td>1.7</td>
<td>26.9</td>
<td>2.0</td>
<td>34.3</td>
<td>3.3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Timber supported 4 sides</td>
<td>21.5</td>
<td>1.8</td>
<td>22.7</td>
<td>0.7</td>
<td>23.4</td>
<td>0.7</td>
<td>27.8</td>
<td>2.7</td>
<td>36.1</td>
<td>2.4</td>
</tr>
<tr>
<td>TCC supported 2 sides</td>
<td>18.1</td>
<td>2.4</td>
<td>23.1</td>
<td>1.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
<td>20.0</td>
<td>1.5</td>
<td>25.9</td>
<td>1.3</td>
<td>32.5</td>
<td>2.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>TCC (6 joists) supported 4 sides</td>
<td>22.0</td>
<td>1.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 7.8.: Comparison between predicted and experimental modal frequencies Floor F

<table>
<thead>
<tr>
<th>Description</th>
<th>$f_{1,1}$ (Hz)</th>
<th>$f_{1,2}$ (Hz)</th>
<th>$f_{1,3}$ (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber supported 4 sides</td>
<td>21.5</td>
<td>22.7</td>
<td>23.4</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
<td>20</td>
<td>25.9</td>
<td>32.5</td>
</tr>
<tr>
<td>$\Delta f_{1,j}$ (%)</td>
<td>-7</td>
<td>14</td>
<td>39</td>
</tr>
</tbody>
</table>

171
To calculate the bending stiffness of the floor, parallel to the direction of the joists, $K_e$ (7580kN/mm), was used in conjunction with the $\gamma$-method.

The upgrade caused the fundamental frequency to decrease by 11.3% and 7.0% when supported on two and four sides respectively. Whereas it was predicted that the frequency would increase by 21% when supported on four sides (Table 7.8). It may be that cracking of the topping (documented in Chapter 6) contributed to this unexpected result, although no comparison data are available to confirm this hypothesis. Reducing the breadth of the floor caused the frequency of the first mode to increase by 10% whilst higher modal frequencies were not detectable.

### 7.4.2. Modal Damping Ratios

When the floor was supported on four sides, the upgrade had a minor effect on the modal damping ratio; a reduction in the equivalent viscous damping of 0.3%. Whereas when supported on two sides the equivalent viscous damping ratio increased by 0.6%. For the second modal frequency addition of the topping increased the modal damping ratios regardless of the support condition. Chui (1986a) reported that timber floors supported on four sides had higher damping ratios than those supported on two. Contrary to those findings, these results found that the timber and TCC floor had higher damping ratios when only simply supported at the joist ends.

Apart from one modal frequency ($f_{1,1}$ supported on four sides) the modal damping ratios increased with the addition of the topping. Relative movement between floorboards and joists, support conditions and joints are all considered as locations of energy dissipation in timber structures (Chui, 1986b; Chui & Smith, 1989; Polensek, 1975). The upgraded floor dissipated additional energy to the timber floor which indicates new locations of energy dissipation. Locations and mechanisms by which a TCC floor could dissipate energy, in addition to those in a timber floor include:

- the topping material damping;
- within the shear connector joints through frictional losses;
- at the interface between the topping and existing floor through friction, if there is appreciable movement.

### 7.4.3. Influence of Floor Breadth

Reducing the breadth of the floor from eight joists to six, caused the first modal frequency of the TCC floor to increase by 10% and reduced the
corresponding equivalent viscous damping ratio by 0.1\% (Table 7.7). Modal frequencies other than the first mode were not excited during the test, two fewer modes than when the floor had eight joists.

7.4.4. Conformity to EN1995-1-1

Figure 7.20 illustrates conformity for the timber and TCC floor support on four sides. The figure is a graphical representation of the following vibration design criteria from EN1995-1-1:

\[
v \leq b (f_1 \zeta - 1)
\]  

(7.10)

where:

- \(v\) is the unit impulse velocity response, the maximum initial velocity caused by an ideal unit impulse (1 Ns). Components above 40Hz are disregarded;
- \(b\) is a limiting value given in the UK National annex (BSI, 2006);
- \(f_1\) is the first modal frequency;
- \(\zeta\) is the modal damping ratio.

To assess conformity to EN1995-1-1 the unit velocity impulse response is assumed to be comparable to the peak accelerance. The peak accelerance is the maximum acceleration of the vibration response divided by the peak force from the force input. In Figure 7.20 the line related to the modal damping ratio is the boundary between unacceptable (above) and acceptable (below) performance.

Figure 7.20 shows that the upgrade significantly improves the performance of the floor with respect to vibration response. The additional inertia, provided by the upgrade, is the largest contributor to the overall improvement in the vibration response. Whilst both floors conform to EN1995-1-1 the improvement in response from the upgrade is so large, that according to EN1995-1-1, even lightly damped floors (less than 1\%) would have acceptable performance. This is advantageous as damping in timber structures is often provided by non-structural elements such as furniture and partitions (Rijal et al., 2010).

7.5. BRE Floor Vibration Test Results

The results of the dynamic testing of the BRE test floor are reported and discussed in this section. Figure 7.21 provides an example of a force and
acceleration record in the time domain of a single hammer input at the centre of the upgraded floor, whilst supported on four sides. Although the acceleration response in this example is approximately two seconds in duration, the record has been shortened to half a second to allow better appreciation of this initial input and response. Figure 7.22 provides an example of a mobility frequency response function (FRF) measured at the centre of the timber and upgraded floors, from a force input applied to the centre of the floor. Each plot is an average of six successive inputs; averaging of multiple impacts reduced experimental errors such as electrical and instrumental noise. To aid comparison, the scales of the axes have been made equal.

7.5.1. Structure Linearity

The theory which has been used to analyse the experimental data relies on the assumption that the floor behaves in a linear manner. In practice most structures exhibit some non-linear behaviour. Ewins (1984) suggests that signs of non-linear behaviour include unrepeatable tests, natural frequencies which vary with magnitude and FRF plots which are distorted. Figure 7.23 presents the mobility FRF from two separate impacts of different magnitudes, with the response measured at the same location. Whilst there appears to be some minor distortion of the FRF at 20-25Hz, further inspection of FRF’s from other impacts, with magnitudes between these extremes, showed that
Figure 7.21.: Example force input and acceleration response (time domain)

Figure 7.22.: FRF: Mobility plot for four side supported timber and TCC floors measured at the centre of the floor
the resolution of the transform function was more critical than any non-linear effects.

![Graph](image)

Figure 7.23.: Linearity check

### 7.5.2. Modal Frequencies

Table 7.9 presents the modal frequencies, modal damping ratios and instantaneous deflections under a point load for the timber floor and upgraded floor with 2 sided and 4 sided support. The percentage change from upgrade of each first order modal frequency and first order modal damping ratio are presented alongside the percentage change in instantaneous deflection in Figure 7.24. Based on literature precedent (Chui, 1986b; Bainbridge & Mettem, 1997; Weckendorf, 2009; Mertens et al., 2007) it would be expected that the topping upgrade would cause the higher modal frequencies to separate, the separation becoming greater as the mode number increased. When the floor was supported on four sides the higher order modes were separated with the upgrade but this did not happen when the floor was supported on two sides. In this support condition the first and second modal frequencies increased by less than 2% before the third and fourth modes decreased in frequency by 14% and 9%. For the timber floor the support conditions had a similar effect, for example the first, second, fourth and fifth modal frequencies were 4%, 10%, 3% and 26% higher when supported on four sides compared to two sides. These corroborate findings by Chui (1986a) which also showed that timber floors supported on four sides had greater mode separation than those supported on two sides. The total number of modal frequencies below 40Hz, which are those assumed to contribute to the
perceptible portion of the vibration response, decreased from seven to five when the timber floor was supported on four sides instead of two. When the upgraded floor was supported on four sides the number of modal frequencies below 40Hz decreased from five to four.

Modal frequencies for floors supported on four sides were predicted using equation 7.9. For the TCC floor, the correlation between the change in predicted modal frequencies and experimentally observed change in modal frequencies was 5%, 5% and 18% for modes $f_{1,1}$, $f_{1,2}$ and $f_{1,3}$ respectively. In all cases the beneficial separation of the higher order modes was overestimated and the effect became worse at higher modes. The main cause of the inaccuracy was the underestimation of the stiffness of the timber floor perpendicular to the direction of the joists, although the prediction of the stiffness of the TCC floor perpendicular to the direction of the joists was also underestimated. For example, the error between experimental and analytical values for mode three was 44% for the timber and TCC floors. This method was inadequate for predicting the frequency of higher order modes because estimating the stiffness perpendicular to the direction of the joists was too inaccurate. However, without a better estimation of the bending stiffness of the floor in this direction, other methods are unlikely to be any more accurate.
Table 7.9.: Modal frequencies (below 40 Hz) and damping ratios

<table>
<thead>
<tr>
<th>Description</th>
<th>f&lt;sub&gt;1,1&lt;/sub&gt; (Hz)</th>
<th>ζ&lt;sub&gt;1,1&lt;/sub&gt; (%)</th>
<th>f&lt;sub&gt;1,2&lt;/sub&gt; (Hz)</th>
<th>ζ&lt;sub&gt;1,2&lt;/sub&gt; (%)</th>
<th>f&lt;sub&gt;1,3&lt;/sub&gt; (Hz)</th>
<th>ζ&lt;sub&gt;1,3&lt;/sub&gt; (%)</th>
<th>f&lt;sub&gt;1,4&lt;/sub&gt; (Hz)</th>
<th>ζ&lt;sub&gt;1,4&lt;/sub&gt; (%)</th>
<th>f&lt;sub&gt;1,5&lt;/sub&gt; (Hz)</th>
<th>ζ&lt;sub&gt;1,5&lt;/sub&gt; (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber supported 2 sides</td>
<td>12.1</td>
<td>1.5</td>
<td>13.9</td>
<td>2.9</td>
<td>20.9</td>
<td>2.6</td>
<td>24.5</td>
<td>4.4</td>
<td>26.4</td>
<td>4.7</td>
</tr>
<tr>
<td>TCC supported 2 sides</td>
<td>12.3</td>
<td>1.2</td>
<td>14.0</td>
<td>1.8</td>
<td>17.9</td>
<td>2.3</td>
<td>24.0</td>
<td>1.3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Timber supported 4 sides</td>
<td>12.6</td>
<td>2.4</td>
<td>15.3</td>
<td>3.4</td>
<td>20.4</td>
<td>2.5</td>
<td>25.3</td>
<td>4.6</td>
<td>30.9</td>
<td>3.1</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
<td>13.1</td>
<td>2.9</td>
<td>17.0</td>
<td>1.3</td>
<td>23.6</td>
<td>1.4</td>
<td>-</td>
<td>-</td>
<td>34.1</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Table 7.10.: Comparison between predicted and experimental modal frequencies

<table>
<thead>
<tr>
<th>Description</th>
<th>f&lt;sub&gt;1,1&lt;/sub&gt; (Hz)</th>
<th>f&lt;sub&gt;1,2&lt;/sub&gt; (Hz)</th>
<th>f&lt;sub&gt;1,3&lt;/sub&gt; (Hz)</th>
<th>f&lt;sub&gt;1,4&lt;/sub&gt; (Hz)</th>
<th>f&lt;sub&gt;1,5&lt;/sub&gt; (Hz)</th>
<th>Δf&lt;sub&gt;1,j&lt;/sub&gt; (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber supported 4 sides</td>
<td>12.6</td>
<td>11.1</td>
<td>15.3</td>
<td>11.4</td>
<td>20.4</td>
<td>12.4</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
<td>13.1</td>
<td>12.2</td>
<td>17.3</td>
<td>13.3</td>
<td>23.6</td>
<td>16.7</td>
</tr>
<tr>
<td>∆f&lt;sub&gt;1,j&lt;/sub&gt; (%)</td>
<td>9</td>
<td>11</td>
<td>16</td>
<td>16</td>
<td>34</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 7.9: Modal frequencies (below 40 Hz) and damping ratios

Table 7.10: Comparison between predicted and experimental modal frequencies
7.5.3. Modal Damping Ratios

Apart from one modal frequency \((f_{1,1} \text{ supported on four sides})\) the modal damping ratios decreased with the addition of the topping by between 13\% and 72\% (Figure 7.24). This is the opposite finding to Floor F, where the modal damping ratios increased due to the upgrade. Previous testing (Chui & Smith, 1989) has shown that the support conditions of timber structures have a large influence on their damping properties and is likely to be a contributing factor to the different observations in this testing.

As with the BRE floor, Ghafar et al. (2008) reported a reduction in the modal damping ratio of a timber panel with the addition of a concrete topping, whereas Ceccotti (1995) suggested that damping ratios of TCC floors should be higher than timber floors, at 2\%. Although the absolute amount of energy that an upgraded floor dissipates is greater than a timber floor (Ghafar et al., 2008), the critical damping also increases due to the upgrade (for this floor approx. 180\%, Equation 7.11). As damping is the energy dissipated as a proportion of the critical damping, the damping ratio of an upgraded floor will reduce if the energy dissipated does not increase by a greater proportion than the square root of the product of the change in stiffness and mass of the floor.

\[
c_r = 2m\omega = 2\sqrt{km}
\]

(7.11)

Nonetheless, the modal damping ratios of the TCC floor, which range between 1.2\% and 2.3\% (Table 7.9), are within the 1\% to 7.5\% range reported in the literature (Ghafar et al., 2008; Rijal et al., 2011; Fragiagomo & Lukaszewska, 2011; Mertens et al., 2007).

7.5.4. Mode Shapes

The first three first-order modes for the TCC floor supported on two and four sides are illustrated in Figure 7.25. The figure presents the elevation of the mode shape viewed in the direction parallel to the span of the joists.

Movement at the unsupported edges of the two sided floor was considerable for all modes, whereas when the floor was supported on four sides there was no appreciable movement when the floor vibrated in the first mode. As the mode number increased there was an increasing amount of movement at the supported edge and at the third mode there was no appreciable difference in mode shape between the floors supported on two or four sides. In contrast, timber floors with two and four side restraint tested by Weckendorf (2009)
demonstrated no appreciable movement at the restrained edges, parallel to the direction of the joists, for all modes of vibration. It is probable that for the current study, the reason that movement increased at the restrained edge as the modal frequency increased is that the force input of the revolving exciter increased by a power of two with the modal frequency (Equation 7.1). By the third mode the edge supports could no longer restrain the floor from moving as the force input was almost four times as large as at the first mode.

Figure 7.25.: TCC floor: shapes of modes 1,1 to 1,3

(a) two-side support  (b) four-side support
7.5.5. Conformity to EN1995-1-1

Figure 7.26 illustrates conformity for the timber and TCC floor supported on four sides. The figure is a graphical representation of the vibration design criteria from EN1995-1-1 (Equation 7.10). Whilst this method provides one measure of the change in performance its weakness is in not being able to adequately allow for the change in perception due to the separation of higher order modes. To assess conformity to EN1995-1-1 the unit velocity impulse response is assumed to be comparable to the peak accelerance. The peak accelerance (Table 7.11), is the maximum acceleration of the vibration response divided by the peak force from the force input. It was found that peak accelerance was not dependant on the support conditions and the change in peak accelerance was proportional to the mass per unit area; the mass of the structure increased by 152% whilst the peak accelerance decreased by 59.1%.

In Figure 7.26 the line related to the modal damping ratio is the boundary between unacceptable (above) and acceptable (below) performance. EN1995-1-1 suggests that the modal damping ratio of all modes is 1%, or in the UK 2%. However in these tests the modal damping ratios varied according to mode and support condition and previous testing has also shown a breadth of results and dependency on support and joint fixity (Chui & Smith, 1989; Ghafar et al., 2008; Rijal et al., 2011; Fragiacomo & Lukaszewska, 2011; Mertens et al., 2007). For comparative purposes the modal damping ratio of the first mode was assumed to be most critical and was used to assess conformity.

<table>
<thead>
<tr>
<th>Table 7.11.: Peak accelerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>peak accelerance</td>
</tr>
<tr>
<td>(mm/s²/N)</td>
</tr>
<tr>
<td>Timber supported 2 sides</td>
</tr>
<tr>
<td>Timber supported 4 sides</td>
</tr>
<tr>
<td>TCC supported 2 sides</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
</tr>
</tbody>
</table>

The floors compared in Figure 7.26 are the timber and TCC floors supported on all four sides. Figure 7.26 shows that the upgrade significantly improves the performance of the floor with respect to vibration response. The additional inertia, provided by the upgrade, is the largest contributor to the overall improvement in the vibration response reducing the peak accelerance by 60%. In contrast the first modal frequency only increased by 4%. Whilst the timber floor supported on four side does not conform to EN1995-1-1 the upgraded floor does conform. The upgrade in general permits acceptable
performance, according to EN1995-1-1, for lightly damped floors (less than 1%). This is advantageous as damping in timber structures is often provided by non-structural elements such as furniture and partitions (Rjal et al., 2010).

In comparison to the only other known vibration testing of complete TCC floors (Mertens et al., 2007), the peak accelerance recorded for these floors was 330% higher. This is a direct consequence of a thinner topping, which leads to a floor which is less stiff and provides less inertia. However, these tests show that a thin topping upgrade is capable of delivering sufficient improvement in performance to overcome vibration problems in timber floors. A reduction in the first modal frequency is avoided by using a thin topping, knowing that even a thin topping will increase the inertia of the floor by a multiple of two or three. Whilst the first modal frequency reported by Mertens et al. (2007) was 5% lower for the TCC floor than the timber floor, in these test the thin topping increased the first modal frequency by 4%. This was as predicted, using the methods from Chapter 3.

Thin upgrades are best suited to floors spanning less than 6m. At spans greater than six metres, existing joist depths are likely to increase, reducing the effectiveness of a thin topping (see Chapter 3 section 3.5.2) as the increase in stiffness would be smaller. Whilst the increase in bending stiffness would be smaller, the change in mass would be similar to a shorter span floor, which could lead to a resonant response, unless the shear connection was
very effective, since long span timber floors are more likely to have a first modal frequency below 8Hz.

7.6. BRE Floor Elastic Test Results

7.6.1. Point Load Deflection

In addition to criteria regarding the velocity response to a unit impulse, EN1995-1-1 also requires floors to be sufficiently stiff from a 1kN central point load. The instantaneous deflection from a central 1 kN point load can be predicted by Equation 7.12 from clause 2.7.2 of the UK N.A. to EN1995-1-1, where $a$, the limiting deflection, is 1.55mm. The values as predicted by Equation 7.12 are reported alongside the experimental values in Table 7.10.

$$w = \frac{1000k_{\text{dist}}l_{\text{eq}}^{3}k_{\text{amp}}}{48(EI)_{\text{joist}}} \leq a$$

(7.12)

where

- $w$ is the instantaneous deflection from a central 1 kN point load.
- $k_{\text{dist}}$ is the proportion of load acting on a single joist.
- $l_{\text{eq}}$ is the equivalent floor span.
- $k_{\text{amp}}$ is amplification factor to account for shear deflections in solid timber joists.
- $(EI)_{\text{joist}}$ is the bending stiffness of a joist.

<table>
<thead>
<tr>
<th>Description</th>
<th>$w$ (mm)</th>
<th>Exp.</th>
<th>Analyt.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber supported 2 sides</td>
<td>1.50</td>
<td>1.55</td>
<td>1.55</td>
</tr>
<tr>
<td>Timber supported 4 sides</td>
<td>1.55</td>
<td>1.55</td>
<td>1.55</td>
</tr>
<tr>
<td>TCC supported 2 sides</td>
<td>0.44</td>
<td>0.44</td>
<td>0.44</td>
</tr>
<tr>
<td>TCC supported 4 sides</td>
<td>0.41</td>
<td>0.41</td>
<td>0.41</td>
</tr>
</tbody>
</table>
The correlation is better for the timber-concrete floor than the timber floor and the experimental timber floor deflection equals the limiting deflection whilst the predicted deflection exceeds it. The edge restraints were found to have no appreciable effect on the central deflection of the timber or TCC floor due to a 1 kN point load which was also observed by Weckendorf (2009).

7.6.2. Load Sharing

For both support scenarios the instantaneous deflection of the floor was reduced by over 70% due to the stiffening effect of the upgrade. In addition to the increase in bending stiffness the increase in load sharing between adjacent joists is a significant contributing factor to the reduction in deflection from a central point load. The maximum instantaneous deflection of the floors supported on four sides expressed as a proportion of the deflection of the adjacent joist is a measure of the load sharing. Load sharing was found to increase by 25% due to the topping upgrade.
7.7. Short-term Bending Test Results

The test results from the short-term bending testing of the BRE floor are presented in this section. Figures 7.29 and 7.30 present the load-mid-span deflection behaviour of joists seven and ten whilst Figure 7.32 presents the load-end slip at joists seven and ten. A comparison between theoretical and experimental values at serviceability limit state (SLS=2.5kN/m²) are presented in Table 7.13.

7.7.1. Uniformity of load

To present these results, the nine load cells have been assumed to act uniformly over the floor throughout the test, so that the load records could be averaged to provide a load per unit area. To check the validity of this assumption, the deviation of each load cell from the mean value was inspected for the whole test. It was found that when the floor was loaded the deviation was less than 5% of the mean value but when unloaded between cycles the deviation from the mean could exceed 40% (Figure 7.27). However the distribution of loads as the floor was unloaded was not of interest and it was considered satisfactory to average loads in this manner.

![Figure 7.27: Deviation of load cells (where $x$ is the load cell value and $\bar{x}$ is the average value of all nine load cells)](image)

Figure 7.27.: Deviation of load cells (where $x$ is the load cell value and $\bar{x}$ is the average value of all nine load cells)
7.7.2. In service Loading

The stiffness of the floor at joists seven and ten versus cycle number is presented in Figure 7.28. The stiffness of each cycle was evaluated by measuring the secant of the load-displacement plot between the minimum and maximum load of the cycle. As with the panels (presented in Chapter 6), the effective bending stiffness of the floor was not constant during the service loading of the panels. It is believed that this phenomena was a function of the non-linear load-slip behaviour of the shear connectors (see Chapter 4 section 4.3.6).

Unlike the panels tested from Floor F, the behaviour of joists seven and ten are not as well aligned. Whilst the change in bending stiffness of joist ten with cycle number is similar to the panels from Floor F, the change in bending stiffness of joist seven is different. For example, the bending stiffness of joist ten decreases as the load decreases between sets of cycles (cycles 15 to 16, 30 to 31 and 45 to 46) whereas the bending stiffness of joist seven increases as the load decreases between sets of cycles. Between cycles five and sixty, the difference in bending stiffness is up to 28%, but this is occurring at the lowest loads, as the load increases, the behaviour of the two joists tends towards uniformity. Consequently the different behaviour could be caused by either loads being unevenly applied, (small differences have a greater proportionate effect at low loads) or frictional stick in the measuring devices (draw-wire transducers) or test floor, also more significant at low loads. Whilst different behaviour was observed between the two joists, no measurable loss of bending stiffness was observed between cycles 5 and 60. The apparent loss in bending stiffness of panels from Floor F (see Chapter 6 section 6.7.2) was not observed for these joists. Whilst these results provide an indication of long-term performance which suggests that the bending stiffness of an upgraded floor will not deteriorate when service loads are less than 40% of the failure load, further testing with thousands of cycles are required to determine true long-term durability.

7.7.3. Non-linear behaviour

Figures 7.29 and 7.30 present the load-mid-span deflection of joists seven and ten alongside the theoretical deflections for zero, full and partial composite action as predicted by the $\gamma$-method. Whilst the stiffness of joist ten increased to 55% of the floor’s maximum load ($F_{\text{max}} = 9.05 \text{ kN/m}^2$), joist seven only increased in stiffness to 40% of the maximum load.
7.7.4. Analytical and Experimental Correlation

Table 7.13 presents the measured and predicted mid-span deflection, end slip, end separation between the joists and topping at SLS (2.5kN/m²) for joists seven and ten. The deflection profile of the floor at its mid-span at SLS is presented in Figure 7.31 alongside the analytical deflection profile. Analytical (predicted) results were calculated using the $\gamma$-method presented in Appendix D. The correlation between the analytical $\gamma$-method prediction and experimental mid-span deflection was comparable to the panels taken from Floor F (see Chapter 6, section 6.7.4) but unsatisfactory compared to previous research where disagreement was less than 5% (Fragiacomo & Lukaszewska, 2011; Persaud & Symons, 2005). The agreement for joist seven was -26.8% and for joist ten, -21.2%. For the other joists, whilst the correlation was mixed, in general the analytical method was non-conservative and over-predicted the stiffness of the panels (see Figure 7.31). At the outermost joists the correlation was closest, but these joists only had to support half the floor area of the other joists, so a lower deflection would be expected. The deflection of joist four, in particular, did not follow the expected floor deflection profile; either the distribution of the load was uneven across the floor or the bending stiffness of the joist was inaccurately characterised. Uneven application of load was likely as the surface of the topping was uneven, preventing even application of the load, which would cause local stress concentrations.
Correlation between analytical and measured end slip was better than for the panel tests reported in Chapter 6. For joists seven and ten the error between the analytical and the experimental result was -16% and -13% respectively. This was better than the panels from floor F and panels C, D and E, which had errors between -18% and -31%. Unlike the mid-span deflection the analytical method over predicted the end-slip of the joists. Agreement between the analytical approach and experimental results for the separation of the topping and joists at the SLS was mixed. For joist seven the error was 47% whereas for joist ten the error was -57%. Vertical separation, which was greater than expected, indicates unequal curvature of the topping and timber, a violation of an assumption of the γ-method.

Table 7.13.: Comparison between experimental and analytical behaviour at SLS (2.5kN/m²)

<table>
<thead>
<tr>
<th></th>
<th>Joist 7</th>
<th>Joist 10</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exp.</td>
<td>Analyt.</td>
</tr>
<tr>
<td>δ(mm)</td>
<td>9.19</td>
<td>7.68</td>
</tr>
<tr>
<td>seend(mm)</td>
<td>0.29</td>
<td>0.4</td>
</tr>
<tr>
<td>sep(mm)</td>
<td>0.19</td>
<td>0.28</td>
</tr>
</tbody>
</table>
7.7.5. Joist end-slip

Figure 7.32 presents the load-end slip behaviour at the ends of joists seven and ten. As was expected from the load-mid-span deflection plots (Figures 7.29 and 7.30), the end-slip at the joists was broadly linear to 55% of the maximum load of the floor before a greater rate of end-slip signalled a gradual loss in composite action between 55% and 75% of the maximum load of the floor. The magnitude of the end-slip at failure was not necessarily equal at both ends of the joist (joist ten), but was dependent on the curvature of the joist.

Figure 7.33 presents the load-end slip behaviour at one end of joists seven and ten with the load-slip behaviour of a pushout specimen. To compare the behaviour side by side, the load for both the joist end-slip and pushout specimen were expressed as a proportion of their maximum loads ($F_{\text{max}}$). Below 30% of $F_{\text{max}}$, the end-slip behaviour was similar to the load-slip behaviour of a single connector. However, thereafter the load-slip behaviour of a single connector changed, it became stiffer than the connectors at the end of the joists. Towards $F_{\text{max}}$, the load-slip behaviour of the pushout specimen plateaued, whereas the end-slip behaviour of joists seven and ten was stiffer, as the load was redistributed to adjacent connectors nearer the centre of the joist. Consequently the loss of composite action was gradual, rather than the
behaviour rapidly plateauing. In turn the loss load mid-span deflection of the panels cannot be well approximated as a bi-linear relationship, as would be possible if the loads did not redistribute between connectors, instead loss of composite action is more gradual. This behaviour is not particularly ductile but it is preferable to the linear behaviour to failure of a timber floor without upgrade.

Figure 7.31.: Mid-span deflection profile at SLS (2.5kN/m²)
Figure 7.32.: Load vs end slip: joists seven and ten

Figure 7.33.: Joist end slip and connector load-slip
7.7.6. Shear Connection Efficiency

At SLS=2.5kN/m² the efficiency of the shear connection, as defined by Gutkowski et al. (2008), was 67% and 76% for joists seven and ten respectively. This is comparable with the findings of the panel testing but is less than would be expected for a composite system with stiff connections such as notches. For notches, the shear connector efficiency could be as high as 90% (Lukaszewska et al., 2008). Whereas the screw connectors, in conjunction with the topping, increased the bending stiffness of the floor by 113% (at joist seven), a connector with 90% efficiency would have achieved a 258% increase. However, as discussed in Chapter 4, notches are impractical for upgrading existing floors and cause greater damage to the existing floor than screw connections. In any case, a 113% increase in stiffness is sufficient, (ignoring strength for now), to allow an increase in load through change of use from residential, 1.5 kn/m² imposed load, to office, 2.5 kN/m² imposed load. Therefore additional structural efficiency and increased bending stiffness are not necessarily required.

7.7.7. Failure

The deflection profile of the floor at 5%, 10%, 20%, 40%, 60%, 80% and 95% of $F_{\text{max}}$ (9.05kN/m²) is presented in Figure 7.34. Throughout the test, joist four undergoes greater deflection relative to the other joists in the floor. As previously discussed, this is most likely due to an uneven application of load to the surface of the floor from the spreader beam, caused by rises and troughs in the finish of the floor. In turn this would have allowed a proportion of the spreader beam to be in contact with the floor leading to stress concentrations. At 40% of the maximum load the deflection of joist four was 14% higher than joist five, by 80% of the maximum load the deflection of joist four was only 8% higher than joist five; a small redistribution of load away from joist four. By 95% of the maximum load joists four and seven had deflected disproportionately compared to the other joists reaching displacements of 79mm and 80mm respectively. Between 3.5kN/m² and 7.5kN/m² joist seven lost all composite action, then joist ten lost all composite action by 8.0kN/m². Before failure of joist four, at 9.05kN/m², joist seven underwent approximately 15mm of deflection without composite action, whereas joist ten underwent approximately 11mm of deflection without composite action.

Final failure of the floor was, first joist four, followed by joist seven, in tension within the middle third of the span, at prominent knot locations on the tension side of the joists (Figure 7.35). At this point the test was finished.
and the panel unloaded as further displacement of the floor would only have caused sequential failure of each remaining joist. As the floor had lost composite action at failure, the strength of the existing floor had not been increased but instead reduced by the weight of the topping. Consequently the upgrade only bettered the serviceability performance; stiffening the floor and improving its vibration response.

![Diagram](image)

Figure 7.34.: Mid-span deflection profile at 5%, 10%, 20%, 40%, 60%, 80% and 95% of $F_{\text{max}}$

![Image](image)

Figure 7.35.: Failure of joist four, initiating at a prominent knot location on tension side of the joist

The failure load of the floor was 9.05 kN/m². The factor of safety on the design load can be calculated in the following manner.

The partial factors for the design loads are $\gamma_g = 1.35$ and $\gamma_q = 1.5$. 

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The possible characteristic imposed loads are \( w_{q,1} = 1.5\text{kN/m}^2 \) (residential (BSI, 2002)) and \( w_{q,2} = 2.5\text{kN/m}^2 \) (office (BSI, 2002)).

Therefore the design imposed loads are:

\[
\gamma_q w_{q,1} = 2.3\text{kN/m}^2 \\
\gamma_q w_{q,2} = 3.8\text{kN/m}^2
\]

To obtain the total design loads the weight of the screed \( w_g = 0.44\text{kN/m}^2 \) is added to the values above:

\[
\gamma_g w_g + \gamma_q w_{q,1} = 2.8\text{kN/m}^2 \\
\gamma_g w_g + \gamma_q w_{q,2} = 4.3\text{kN/m}^2
\]

As the weight of the screed is not included in the failure load measured by the load cells, this should be added to the failure load of the joists.

\[
w_{q,\text{fail,mean}} + w_g
\]

As this is a small sample of joists we must assume that the above loads are mean failure loads for the joists not characteristic values. We should therefore adjust the strength of the joists to obtain an approximate design value. In previous research Ceccotti et al. (2006) suggested that the characteristic strength could be assumed as 70% of the mean strength. Adopting this approach allows approximate design values to be derived in the following manner:

\[
f_n = \left( w_{q,\text{fail,mean}} + w_g \right) \cdot \left( \frac{f_k}{f_{\text{mean}}} \right) \cdot \left( \frac{1}{\gamma_m} \right)
\]

\[
f_n = \left( w_{q,\text{fail,mean}} + w_g \right) \cdot 0.7 \cdot \left( \frac{1}{1.3} \right)
\]

Therefore the factor of safety on the design office loading is 1.2. For a residential loading the factor of safety on the design load is 1.8. In all cases the factor of safety on the design load exceeds one. In this regard the panels have demonstrated adequate performance for both office and residential loading. However it is clear that proper assessment of the existing structure should be made before an upgrade is applied to decide whether the floor requires strengthening as well as stiffening. Whilst the upgrade does reduce the
strength capacity of the floor, the reduction is small as the mass of the topping is less than 5% of the failure load.

Figure 7.36 illustrates a plan view of the topping crack pattern following the test. The numbers refer to the loads at which the cracks had propagated with F denoting cracks that were found following failure of the floor. The lowest load at which cracks were observed was 6.3 kN/m², 70% of the maximum load. The application of the loading was unlikely to be even, due to the rises and troughs in the floor and cracking of the topping was not prevalent during testing of the panels. It is therefore likely that a good proportion of the cracking before failure is attributable to localised hogging induced by uneven application of load. Whilst in practice it is unlikely that a loading pattern sufficient to cause tension cracks to appear, would take place, this phenomena would merit further investigation. Cracking found after failure of the floor was most likely to have been caused during failure due to the disparity in deflections between joists. From this point of view the floor demonstrated that the upgrade solution has good robustness during and beyond SLS.

![Figure 7.36: Topping crack pattern (all loads in kN/m², F denoted cracks found following failure of the floor)](image-url)
7.8. Concluding Comments

This chapter has reported the testing of two floors. Both floors were subjected to dynamic loads, whilst one floor was subjected to short-term bending loads to investigate:

- the increase in floor bending stiffness,
- the correlation with the experimental deflections at SLS and the analytical $\gamma$-method presented in Annex B of EN1995-1-1,
- the effects of cycling the load on the effective bending stiffness of the panels,
- the change in load sharing characteristics,
- the change in dynamic performance,
- the dynamic performance with respect to EN1995-1-1 guidelines.

As with the panels the topping was found to increase the bending stiffness of the floor. At SLS (2.5 kN/m²) joist seven had increased in effective stiffness by 113% (the mean increase in effective bending stiffness for the middle eight joists was 111%) allowing the floor to achieve a span/300 deflection at 2.9kN/m².

Correlation between the popular, analytical, $\gamma$-method predictions and experimental results was less than satisfactory with respect to SLS mid-span deflection and end slip. The error between the analytical and experimental mid-span deflection was between 16% and 13% for joists seven and ten. For end-slip, the error between the analytical and experimental results was between 35% and 38% for joists seven and ten.

Applying cycles of loading in the service range resulted in no measurable deterioration in effective bending stiffness of joists seven and ten. This counters the initial conclusions made from the panel testing findings (Chapter 6) and provided further evidence of the robustness of the upgrade system. However the nature of the test setup (hydraulic hand pump) precluded a longer test duration with a greater number of cycles and consequently it is not properly understood how the panels will behave after thousands of cycles of loading. In this respect the conclusions are limited in their scope and application.

The topping upgrade was found to increase the load sharing effects between joists. Using the maximum deflection under a central 1 kN load, expressed as a proportion of the adjacent joist as a measure of load sharing, the upgrade was found to increase the load sharing between joists by 25%, thereby allowing design loads acting on single joists to be reduced.
The addition of the topping was found to: reduce the number of first order modal frequencies below 40 Hz from five to three, increase the frequency of the first mode by 4%, increase the separation of higher first order modal frequencies when the floor was supported on four sides, and reduce the damping ratio. Whilst the error between the predicted change in frequency of the first mode and the actual change in frequency was small, 5%, predicting the change in frequency of higher order modes, using the equation of vibration for orthotropic plates simply supported on four sides, was an imprecise approach. The error increased with mode number and was due to the inadequate estimation of the stiffness of timber and TCC floors perpendicular to the direction of the joists.

Contrary to previous reports, which suggested that damping ratios increase from approximately 1% for timber floors, to 2% for TCC floors (Ceccotti, 1995), in these tests the modal damping ratios decreased with addition of the topping.

The effectiveness of the upgrade with respect to changing the vibration response was assessed using EN1995-1-1 design guidelines. It was found that the upgrade significantly improved the response of the floor, mostly due to the reduced peak acceleration of the floor from an impact, and allowed even lightly damped floors (less than 1%) to be acceptable under EN1995-1-1 guidelines; which is advantageous as damping can be highly variable.
8. Conclusions and Further Work

The aim of this research study has been to develop thin (20mm thick) topping timber-concrete composite (TCC) upgrades to improve the serviceability performance of timber floors. In particular, to understand how the addition of a topping improves the bending stiffness and transient vibration response of timber floors.

In Chapter 3, an understanding of how the thickness of a topping upgrade influences the change in stiffness and transient vibration response of a timber floor was developed. Previous approaches to determining the topping depth were either, to create a balanced section, with timber and topping contributing equally to the bending stiffness of the composite (Van der Linden, 1999), or to create a composite section with the neutral axis at the interface between the parts, thereby ensuring that the concrete resisted compression and the timber resisted tension (Yeoh et al., 2011b). However, neither approach is necessarily suitable for upgrading a timber floor, since topping depths from either of these approaches deliver at least four-fold increases in bending stiffness, in excess of what is required for most upgrades. In addition, both approaches fail to acknowledge that timber performs well in compression. The new approach developed in this study determines the topping depth on the basis of the requirement to increase bending stiffness and improve vibration performance. Using this method showed that the bending stiffness of a TCC does not increase linearly with topping depth. As the topping depth increases, the upgrade becomes less effective at increasing the bending stiffness of the composite. Furthermore, even thin toppings increase the inertia of a timber floor by several times almost guaranteeing an improvement in transient vibration response. Consequently, to improve vibration response, the topping should be optimised to increase the first modal frequency to avoid a resonant response.

In Chapter 4, inclined screw connectors were identified as the most appropriate existing shear connectors for a thin topping application. Screw connectors were characterised using pushout specimens subjected to static load. Utilising a factorial design approach, the testing also established how the stiffness and strength of screw connectors were influenced by the
thickness of the topping, screw inclination and timber density. Whilst the depth of the topping affected the stiffness of the connectors, the inclination of the screws was more influential. Although the strength of the connectors was influenced by all three factors, the topping depth was most influential. Further experimental studies demonstrated that zones of low stiffness beneath the screw heads, did not influence the strength or stiffness of the connectors. From this point of view the connection system is robust.

In Chapter 5, a method was devised for testing pushout specimens under low amplitude cyclic loads, to establish the dynamic stiffness and energy dissipation properties of the shear connectors. Specimens were tested in two independent systems, which demonstrated that the stiffness and energy dissipation results from the tests were unreliable and non-replicable. This was because the testing system exerted significant influence on the stiffness and energy dissipation of the connectors. This method needs further work to demonstrate its reliability before characterisation of connectors can take place.

In Chapter 6, nine panels were subjected to short-term bending tests. Panels were loaded with six equally spaced line loads, simulating a uniformly distributed load, whilst mid-span deflection and end slip were measured with displacement transducers. Depending on the depth of the joists, the addition of the topping increased the bending stiffness of the panels by between 114% (170mm deep joists) and 298% (95mm deep joists). This increase in bending stiffness is sufficient to accommodate change of use from residential (1.5kN/m² imposed load) to office occupancy (2.5kN/m² imposed load). Correlation between the experimental and analytical (γ-method) mid-span deflections at SLS (2.5kN/m²) was non-conservative, errors were between 5% and 31%. Vertical separation of the topping and timber panel is suggested as the source of the error and further work on this aspect would provide a meaningful contribution to TCC research. The final series of tests compared panels with and without cracks in the topping. Cracking of the topping was found not to influence the bending stiffness or strength of the panels, demonstrating the robustness of the upgrade system and providing reassurance to potential practitioners.

A short-term bending test of a floor to collapse was completed at the Building Research Establishment and is presented in Chapter 7. The floor was loaded with six equally spaced line loads, simulating a uniformly distributed load, whilst mid-spa deflections and end slip were measured with draw wire transducers and displacement transducers. Sixty cycles of load were applied to the floor, increasing in amplitude up to 40% of the floor’s maximum load. Since the bending stiffness of the floor did not deteriorate throughout the test,
further evidence was provided for the robustness of the composite. Prior to failure, the floor demonstrated progressive loss of composite action, leading to a loss of bending stiffness. Consequently, the strength of the floor was not improved by the upgrade, rather the possible imposed load that could be applied to the floor was decreased by the weight of the topping upgrade. The factor of safety on design loads for an office environment (2.5kNm²) was only 1.2, due to the strength of the timber joists. Before upgrading a floor using this technique, the existing floor joists should be surveyed and strength graded to ensure that appropriate factors of safety on design loads are available. Elastic point load tests at the centre of the floor, showed that the topping upgrade improved the load sharing between adjacent joists by 25%, thereby allowing design loads acting on single joists to be reduced.

Vibration testing of two floors was completed and reported in Chapter 7. Modal frequencies and modal damping ratios were identified from the frequency response functions. Small increases in the first modal frequency were achieved, with 76% connector efficiency, an improvement on previous testing where the first modal frequency decreased with the addition of the topping (Ghafar et al., 2008). Contrary to previous reports, which suggested that damping ratios increase from approximately 1% for timber floors, to 2% for TCC floors (Ceccotti, 1995), in these tests the modal damping ratios decreased with addition of the topping. The upgrade was found to improve the transient vibration response of the timber floor, mainly due to an increase in the inertia of the floor. Performance of the upgraded floor far exceeded conformity to EN1995-1-1 (CEN, 2004a) criteria; the upgraded floors would have conformed even if they were lightly damped (less than 1% equivalent viscous damping ratio). Modal frequencies were predicted according to the equation for calculating the modal frequencies of an orthotropic plate simply supported on four sides. Apart from the first modal frequency, the prediction of higher first order modal frequencies was inaccurate. The inaccuracy occurred because the stiffness perpendicular to the direction of the joists could not be accurately estimated.

8.1. Main Conclusions

For refurbishing timber floors, a 20mm thick topping was found to sufficiently increase the bending stiffness and improve the transient vibration response. The stiffness of the screw connectors was influenced by the thickness of the topping and the inclination of the screws. During the short-term bending tests, the gamma method provided a non-conservative prediction of composite bending stiffness. In the majority of cases the modal
frequencies of the floors tested increased after upgrade, whilst the damping ratios decreased. The upgrade system was shown to be robust as cracking of the topping did not influence the short-term bending performance of panels.

8.2. Further Work

Throughout all the tests, the topping performed well, not displaying any shrinkage cracking and possessing sufficient compressive strength and stiffness to provide an effective upgrade. However, the topping used was a screed normally used to provide a wearing surface to protect concrete floors against industrial vehicles. Optimisation of the topping for this new application would make the solution more cost effective and more appropriate for upgrading timber floors.

Two practicalities of using the upgrade method on site should be resolved before the solution is applied to an existing floor. First, existing timber floors suffer permanent creep deflection, which if left uncorrected would lead to a thicker screed at the centre of the floor than at the perimeter. To overcome this scenario the floor would have to be levelled prior to construction, either on the floorboard side, with a new deck, or by propping the underside of the floor to remove the creep deflection prior to placing the topping. Research is required to understand how these interventions would influence the effectiveness of the upgrade. Second, for existing floors, fixing inclined screws into the joists near the joist ends is impractical because there is insufficient space to manipulate an electric screwdriver. It is important to resolve this problem because the shear connectors at the ends of the joist are subjected to the greatest loads and deformations. An existing solution, is to fix the screws vertically (Fragiacomo, 2012), but this work has shown that, for thin toppings, these connectors fail undesirably, cracking the topping. Alternative options which merit investigation, are plug connectors at the ends of the joists or omitting connectors entirely at the ends of the joists.

The strength of the shear connectors was low compared to other connection reported in the literature. To prevent the connections failing by pullout of the screw from the topping, the pointside penetration of the screws into the joists was reduced. Further pushout testing should be conducted with 100mm and 120mm long screws, to find out whether the connection strength can be increased, whilst avoiding a pullout failure from the topping.

Throughout the testing of the panels and floors, the $\gamma$-method was consistently inaccurate, over predicting the bending stiffness at SLS. Further work should address whether the inaccuracy is due to the vertical separation of
the topping and timber. If this is the case, additional work should consider whether adapting the partial interaction theory or using finite element methods is the most suitable/easily applied method for predicting the SLS performance of thin topping TCC floors.

Whilst the first modal frequencies could be predicted, the modal frequencies of higher order frequencies were not accurately predicted. Further work to predict these frequencies is necessary to advance TCC vibration research.

Establishing the dynamic stiffness and energy dissipation of connectors from pushout specimens subjected to low amplitude cyclic loads was found to be unreliable and non-replicable. The testing system was found to influence both properties of the connector. To address this, future investigations should aim to: select the most appropriate type of test specimen (symmetrical, asymmetrical or pure shear) for the tests; identify methods of limiting energy loss within the testing system and identify non-contact methods for measuring displacements.

More general issues which have not been addressed in this study but merit attention include: acoustic; fire and creep performance of thin topping TCCs. Increased knowledge in these areas would enhance the commercial potential of this upgrade solution.

With this further work in mind, thin topping TCC upgrades offer a practical and effective solution to building practitioners, for improving the serviceability performance of existing timber floors.
References


A. | Basis of Vibrating Systems

To assist the reader to understand the basics of vibrating structures an introduction to vibrating systems is presented in this appendix. This includes an introduction to sinusoidal vibration and the equation of motion for undamped and damped vibrating systems. Although real structures have multiple degrees of freedom (MDOF) this introduction is limited to single degree of freedom systems (SDOF) because MDOF systems can be represented as a superposition of several SDOF system (Ewins, 1984). In depth discussions with regard to MDOF systems are given by Pain (2005), Thorby (2008) and Timoshenko & Young (1955).

A.1. Sinusoidal Vibration

Sinusoidal vibration is an alternative term for simple harmonic motion, describing the oscillatory behaviour of a body displaced from its equilibrium position. As the body moves away from its equilibrium position a restoring force acts to move the body back to equilibrium (Figure A.1). The restoring force is described by Hooke’s law (Equation A.1), whilst Newton’s second law defines the body’s rate of change in velocity (acceleration) (Equation A.2).

\[ F = -kx(t) \]  \hspace{1cm} (A.1)

\[ F = m\ddot{x}(t) \]  \hspace{1cm} (A.2)

Figure A.1.: A spring-mass system
Where:

\( F \) is the force acting on the body.

\( k \) is the stiffness of the body.

\( m \) is the mass of the body.

\( x \) is the displacement of the body.

\( \ddot{x} \) is the acceleration of the body.

If the motion is free and undamped then no energy is lost from the system and Equation A.1 equals Equation A.2, forming Equation A.3, the equation of motion.

\[
m\ddot{x} (t) + kx (t) = 0
\]  \hspace{1cm} (A.3)

The undamped, angular frequency of the body, \( \omega_n \), described by Equation A.4, allows an alternative form of the equation of motion, Equation A.5, to be expressed.

\[
\omega_n = \sqrt{\frac{k}{m}} \text{[rad/s]}
\]  \hspace{1cm} (A.4)

\[
\ddot{x} (t) = -\omega^2 x (t)
\]  \hspace{1cm} (A.5)

A solution to the equation of motion is (Pain (2005)):

\[
x (t) = X \sin (\omega t + \varphi) = A_1 \sin \omega t + A_2 \cos \omega t
\]  \hspace{1cm} (A.6)

Where:

\[
A_1 = X \cos \varphi
\]

\[
A_2 = X \sin \varphi
\]

\[
X = \sqrt{A_1^2 + A_2^2}
\]  \hspace{1cm} (A.7)

Where:

\( X \) is the amplitude of the oscillation.

\( \varphi \) is the phase angle (rad).
The relationship between displacement, velocity and acceleration are best illustrated by Figure A.2 where the phase angle is zero. Displacement, velocity and acceleration are plotted against time for a body oscillating in simple harmonic motion. Velocity is $\pi/2$ radians out of phase with displacement and lags acceleration by $\pi/2$ radians.

![Figure A.2: Variation of displacement, velocity and acceleration with time](image)

### A.2. Damped Oscillations

The derivation presented thus far only considers undamped, free oscillations, however in a real system energy is dissipated leading to the decay of vibrations. Damping mechanisms in a structure might include: damping inherent within the materials, air resistance and frictional losses within joints. As energy is dissipated from an oscillating body (Figure A.3) the accelerations, velocities and displacements reduce. The resistive damping force, dissipating energy, always opposes the direction of motion and is proportional to the magnitude of the velocity (Pain (2005)).

\[
F = -c \dot{x}(t)
\]

(A.8)

Where:

$c$ is the damping coefficient.
Therefore the equation of motion for a damped oscillating body is:

\[ m \ddot{x}(t) + c \dot{x}(t) + k x(t) = 0 \]  \hspace{1cm} (A.9)

Equations A.10 (a), (b) and (c) are assumed to be the solutions for the damped equation of motion (Pain (2005)). By substituting the solutions into the damped equation of motion and dividing through by \( Ce^{\lambda t} \) the characteristic equation, Equation A.11, is found. Subsequently the characteristic equation is solved with the quadratic formula to give Equation A.12.

\[ x = Ce^{\lambda t} \hspace{1cm} (a), \hspace{1cm} \dot{x} = \lambda Ce^{\lambda t} \hspace{1cm} (b), \hspace{1cm} \ddot{x} = \lambda^2 Ce^{\lambda t} \hspace{1cm} (c) \]  \hspace{1cm} (A.10)

\[ m\lambda + c\lambda + k = 0 \]  \hspace{1cm} (A.11)

\[ \lambda = -\frac{c}{2m} \pm \sqrt{\left( \frac{c}{2m} \right)^2 - \frac{k}{m}} \]  \hspace{1cm} (A.12)

Considering the square root term of Equation A.12 there are three possible descriptions of a damped system:

- Underdamped, the situation for most real structures, the acceleration decays over more than one oscillation when:

\[ \left( \frac{c}{2m} \right) < \frac{k}{m} \]  \hspace{1cm} (A.13)

- Overdamped, the motion decays exponentially within one cycle and occurs when:

\[ \left( \frac{c}{2m} \right) > \frac{k}{m} \]  \hspace{1cm} (A.14)
Critically damped, when the square root element of Equation A.12 is zero and the damping coefficient, $c$, for this special case is called the critical damping, $c_r$. Thorby (2008) describes critical damping as “the boundary between oscillatory and non-oscillatory behaviour”.

$$c_r = 2m\omega = 2\sqrt{km} \quad \text{(A.15)}$$

The most commonly used measure of damping, the equivalent viscous linear damping ratio, is the energy dissipation expressed as a proportion of the critical damping.

$$\zeta = \frac{c}{c_r} \quad \text{(A.16)}$$

Other means of quantifying linear damping include: the logarithmic decrement; the hysteretic damping coefficient (also known as the structural damping coefficient); the specific damping capacity; the number of half cycles to half amplitude and the loss coefficient (Thorby, 2008).

Equations A.10 and A.12 will now be rewritten in terms of the undamped natural frequency, $\omega_n$, and the equivalent viscous damping ratio.

$$\dot{x}(t) + 2\zeta\omega_n\dot{x}(t) + \omega_n^2 x(t) = 0 \quad \text{(A.17)}$$

$$\lambda = -\zeta\omega_n \pm \sqrt{\zeta^2 - 1} \quad \text{(A.18)}$$

As $i = \sqrt{-1}$, Equation A.18 can be rewritten as:

$$\lambda = -\zeta\omega_n \pm i\omega_n \sqrt{1 - \zeta^2} \quad \text{(A.19)}$$

For practical situations where a system is underdamped we take the following roots (Thorby, 2008):

$$\lambda = -\zeta\omega_n \pm i\omega_d \quad \text{(A.20)}$$

Where:

$$\omega_d = \omega_n \sqrt{1 - \zeta^2} \quad \text{(A.21)}$$

$\omega_d$ is the damped natural frequency and is lower than the undamped natural frequency, $\omega_n$. However, because the damping in structures is low, the difference is small and can be ignored (Thorby, 2008; Weckendorf, 2009).
A.3. Resonant and Transient Responses

Resonance is the magnification of a vibration’s amplitude, caused when the frequency of the free vibration of the system coincides with the frequency of a continuous disturbing force (Timoshenko & Young, 1955). Whereas, a transient response is characterised by a series of impacts, each followed by a separate decay. Human footfall, the force most likely to vibrate a residential timber floor, can be approximated as a series of impulses, which if spaced at the natural frequency, 1st, 2nd or 3rd harmonic of the floor, could cause the floor to resonate. However, a transient response is more likely to occur than a resonant response since timber floors generally have a natural frequency above 8Hz (Johnson, 1994); 8Hz is approximately the 3rd harmonic frequency of footsteps and marks the divide between where resonant and impulsive responses are observed. Indeed EN1995-1-1 (CEN, 2004a) only provides clauses on vibration design for timber floors which have a natural frequency above 8Hz. In EN1995-1-1 an acceptable vibration response is evaluated on the basis of the floor’s predicted response to a unit impulse velocity.
B. Partial Interaction

Partial interaction theory was first derived by Stüssi (1947). The following derivation and the symbols used have been drawn from work presented by Yam (1980). It allows the prediction of deflections and stresses in beams made of two components with a connection which resists interfacial slip between the components. The connection joining the components together can be described as either ‘full’, when it is completely rigid and does not deform, ‘zero’, when there is no connection, or ‘partial’, when the stiffness of the connection lies somewhere between zero and fully rigid.

B.1. Full Interaction

Although all connectors are not perfectly rigid leading to only partial interaction, it is worth first considering full composite action. When the connection is perfectly rigid there is no interfacial slip and consequently the strain difference due to slip is zero and the strain distribution of the composite is linear with depth (Figure B.1).

![Figure B.1.: Full interaction: strain distribution](image)

The tension in the timber beam is:

\[ T = kE_2 A_2 \left( h_1 + \frac{h_2}{2} - n \right) \]  

(B.1)

Which is equal to the compressive force in the topping:

\[ C = kE_1 A_1 \left( n - \frac{h_1}{2} \right) \]  

(B.2)
The neutral axis depth is found by equating the compressive force in the topping and the tensile force in the timber:

\[ n = \frac{h_1}{2} + \left( \frac{h_1 + h_2}{2} \right) \left( \frac{E_2 A_2}{E_1 A_1 + E_2 A_2} \right) \]  

(B.3)

The bending moment (Equation B.4) is obtained by summing the zero interaction component and the resistance provided by the couple provided by the tensile and compressive forces. Equation B.4 can be rewritten to include \( \alpha \), the composite stiffness factor.

\[ M = k \sum EI + C \left( \frac{d_1 + d_2}{2} \right) = k \sum EI (1 + \alpha) \]  

(B.4)

Substituting equation B.3 into equation B.2 to eliminate \( n \) from \( C \) and then substitute the result into equation B.4 to solve for \( \alpha \) (Equation B.5).

\[ \alpha = \frac{E_1 A_1 E_2 A_2}{E_1 A_1 + E_2 A_2} \cdot \frac{(h_1 + h_2)^2}{4 (E_1 I_1 + E_2 I_2)} \]  

(B.5)

### B.2. Partial Interaction

If there is no vertical or horizontal separation of the components then there is full composite interaction. However all connectors exhibit some flexibility rendering only partial interaction possible. Yam (1980) highlights that uplift, vertical separation, is likely to have a negligible effect on the behaviour of a composite, therefore it is ignored and only slip, horizontal separation, is considered. By assuming that there is no vertical separation of the elements it is implied that the curvature of both elements are equal.

Slip of the connectors causes a strain difference, \( e \) (Figure B.2), at the interface between the timber and concrete that means that the strain distribution cannot be determined using Equations B.1 and B.2.

Using the additional notation in Figure B.2 the tension and compressive forces and the strain difference are now written as:

\[ C = k E_1 A_1 \left( n_1 - \frac{h_1}{2} \right) \]  

(B.6)

\[ T = k E_2 A_2 \left( \frac{h_2}{2} - n_2 \right) \]  

(B.7)

\[ e = k \left( h_1 + n_2 - n_1 \right) \]  

(B.8)
Equate equations B.6 and B.7 to form B.9 and rearrange equation B.8 in terms of $n_2$.

$$E_1 A_1 n_1 + E_2 A_2 n_2 = E_1 A_1 h_1/2 + E_2 A_2 h_2/2$$  \[(B.9)\]

$$n_2 = \frac{e}{k} + n_1 - h_1$$  \[(B.10)\]

Substitute equation B.9 into B.10 and rearrange to find $n_1$.

$$n_1 = \frac{1}{E_1 A_1 + E_2 A_2} \left[ E_2 A_2 \left( \frac{h_2}{2} + h_1 \right) - E_2 A_2 \left( \frac{e}{k} \right) + E_1 A_1 \left( \frac{h_1}{2} \right) \right]$$  \[(B.11)\]

$n_1$ can now be eliminated from equation B.6 by substitution of equation B.11 to form equation B.12.

$$C = k \cdot \frac{E_1 A_1 E_2 A_2}{E_1 A_1 + E_2 A_2} \cdot \left[ \left( \frac{h_2}{2} + \frac{h_1}{2} \right) - \left( \frac{e}{k} \right) \right]$$  \[(B.12)\]

Finally $M$ can be expressed without $k$ by rearranging equation B.12 in terms of $k$ and substituting into Equation B.4.

$$M = \frac{(h_1 + h_2) (1 + \alpha)}{2 \alpha} C + \frac{2 \sum EI}{h_1 + h_2} e$$  \[(B.13)\]

### B.3. interface Shear Flow

The interface shear flow is at its maximum when the connection is fully rigid and reducing to zero without connection is described by Figure B.3 where force equilibrium of an element of a composite beam is considered.

Equilibrium gives

$$qd x + dC = 0$$  \[(B.14)\]
Figure B.3.: Equilibrium of an element

\[ q = - \frac{dC}{dx} = 0 \] (B.15)

The slip stiffness of the connector is defined as:

\[ K = \frac{qa}{s} \] (B.16)

Which leads to a definition of the strain difference between the components.

\[ \epsilon = \frac{ds}{dx} = \frac{a}{K} \cdot \frac{dq}{dx} = \frac{a}{K} \left( - \frac{d^2C}{dx^2} \right) \] (B.17)

**B.4. General Differential Equation**

Finally the governing differential equation can be defined by substituting equation B.17 into equation B.13. Equation B.18 is independent of loading and support conditions.

\[ \frac{2a \sum EI}{K (h_1 + h_2)} \cdot \frac{d^2C}{dx^2} - \frac{(h_1 + h_2) (1 + \alpha)}{2\alpha} C + M = 0 \] (B.18)
This Appendix solves the general governing differential equation for partial interaction for the specific case of 3-point bending.

C.1. Solving the Governing Differential Equation

The governing second order differential equation has the following solution:

\[ C = \beta P \left( x - \frac{\sinh \psi x}{\psi \cosh \psi l/2} \right) \]  \hspace{1cm} (C.1)

where:

\[ \frac{d^2 C}{dx^2} - \psi^2 C + \psi^2 \beta Px = 0 \]  \hspace{1cm} (C.2)

\[ \psi^2 = \frac{(h_1 + h_2)^2}{4 \sum EI} \cdot \frac{K (1 + \alpha)}{a \alpha} \]  \hspace{1cm} (C.3)

\[ \beta = \frac{\alpha}{(d_1 + d_2)(1 + \alpha)} \]  \hspace{1cm} (C.4)

C.2. Mid-Span Deflection

Since C is now known the curvature of the beam can be integrated twice to find the deflection of the beam at any position along its length.

\[ w = \frac{Px}{4 \sum EI} \left[ \left( \frac{x^2}{6} - \frac{l^2}{8} \right) + \beta (h_1 + h_2) \left( \frac{\sinh \psi x}{x \psi^3 \cosh \psi l/2} + \frac{l^2}{8} - \frac{1}{\psi^2} - \frac{x^2}{6} \right) \right] \]  \hspace{1cm} (C.5)

When \( x = l/2 \) equation C.5 reduces neatly to equation C.6.

\[ w = -\frac{Pl^3}{48 \sum EI} + \frac{Pl \beta (h_1 + h_2)}{4 \sum EI} \left( \frac{\sinh \psi x}{x \psi^3 \cosh \psi l/2} + \frac{l^2}{12} - \frac{1}{\psi^2} \right) \]  \hspace{1cm} (C.6)
Equations C.6, C.3 and B.5 can also be modified to allow for the inclusion of an interlayer.

\[ \alpha = \frac{E_1 A_1 E_2 A_2}{E_1 A_1 + E_2 A_2} \cdot \frac{(h_1 + h_2 + 2t)^2}{4 (E_1 I_1 + E_2 I_2)} \]  
(C.7)

\[ \psi^2 = \frac{(h_1 + h_2 + 2t)^2}{4 \sum EI} \cdot \frac{K (1 + \alpha)}{a \alpha} \]  
(C.8)

\[ w = -\frac{P l^3}{48 \sum EI} + \frac{P l \beta (h_1 + h_2 + 2t)}{4 \sum EI} \left( \frac{\sinh \psi x}{x \psi^3 \cosh \psi l/2} + \frac{l^2}{12} - \frac{1}{\psi^2} \right) \]  
(C.9)

C.3. Shear Flow

By differentiating Equation C.1 the shear flow at the interface between the timber and concrete is found.

\[ q = -\frac{dC}{dx} = -\beta P \left( 1 - \frac{\cosh \psi x}{\cosh \psi l/2} \right) \]  
(C.10)

C.4. End Slip

The slip at the interface is derived by considering Equation C.10, the spacing of the connectors and the slip modulus of the connectors.

\[ s = \frac{qa}{K} = -\beta a P \frac{1 - \cosh \psi x}{\cosh \psi l/2} \]  
(C.11)
D. | Gamma Method

In this Appendix the equations that underpin the method for predicting the effective bending stiffness of mechanically jointed beams are presented. It is an exact solution for the equations of partial interaction for beams to subjected to loads that give a sinusoidal or parabolic bending moment (Persaud & Symons, 2005). This method is presented in Annex B of Eurocode 5 (CEN, 2004a) and is often referred to as the gamma method or γ-method.

D.1. Assumptions

The following assumptions are made:

- the beams are simply supported with span \( l \),
- the individual parts are full length across the span,
- the individual parts are connected together by connectors with slip modulus \( K \).
- the spacing between the connectors is constant or varies uniformly according to the shear flow between the parts.
- the bending moment acting on the beam varies sinusoidally or parabolically.

D.2. Effective Bending Stiffness

Deflections are calculated from an effective bending stiffness \((EI)^e\).

\[
(EI)^e = E_1 I_1 + E_2 I_2 + \gamma S \tag{D.1}
\]

With:

\[
S = \frac{E_1 A_1 E_2 A_2 z^2}{E_1 A_1 + E_2 A_2} \tag{D.2}
\]

Where:
$E_i$ is the modulus of elasticity (topping is denoted by $i=1$ and timber $i=2$)

$l_i$ is the second moment of area

$A_i$ is the cross-sectional area

$z$ is the distance between the centroids of each part

The shear bond coefficient, $\gamma_i$, is expressed as:

$$\gamma_i = \frac{1}{1 + \left(\frac{\pi^2S}{Kl^2z^2}\right)}$$

With:

$$k = \frac{K}{a}$$

Where:

$a$ denotes the spacing of the connectors

$K$ denotes the slip modulus of the connectors

$l$ denotes the span of the beam

For beams with connectors spaced in proportion to the shear force along the beam an effective spacing can be used:

$$a = a_{ef} = 0.75a_{min} + 0.25a_{max}$$

Where $a_{max} \leq 4a_{min}$.

The effective bending stiffness can be used to calculate the mid-span deflection of a uniformly loaded beam (Equation D.6), the shear load in a connector (Equation D.7), the longitudinal end slip (Equation D.8) and the vertical separation of timber and topping (Equation D.9):

$$w = \frac{5ql^4}{384 (EI)_{ef}}$$

$$T (x) = \frac{\gamma S}{z (EI)_{ef}} a (x)$$

$$s_{end} = \frac{T (x)}{K}$$

$$s_{sep} = \frac{T (x)}{K} tan \phi$$

Where:

$T$ is the shear load in a connector

$\phi$ in the angle of the shear connector, measured from the horizontal
E. Panel and Floor, Plans, Sections and Elevations

E.1. Panels A and B

Figure E.1.: Panels A and B elevations and section (all dimensions in mm)
E.2. Panels C, D and E

Figure E.2.: Panels C, D and E section (all dimensions in mm)
Figure E.3.: Panels C, D and E elevation (all dimensions in mm)
E.3. Floor F

Figure E.4.: Floor F plan and elevation (all dimensions in mm)
E.4. Panels taken from Floor F

Figure E.5.: Floor F panels section (all dimensions in mm)
**E.5. BRE Floor**

Figure E.6.: BRE Floor plan (all dimensions in mm)
Figure E.7.: BRE Floor elevation (all dimensions in mm)

LEGEND:

- **EXCITATION LOCATION + ACC.**
- **ACCELEROMETER**

Figure E.8.: BRE Floor accelerometer and impact location plan (all dimensions in mm)