Zainal Abidin Akasah, David Yeoh' Lam Kah Leong/ International Journal of Engineering Research and Applications (IJERA) ISSN: 2248-9622 www.ijera.com
Vol. 2, Issue6, November- December 2012, pp.1337-1355

Analytical Assessment for Comparisons of Triple-T Precast-Concrete-Timber Composite Floor using Gamma method

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Abstract

Concrete-timber composite floor is a construction technique combining two materials, concrete and timber, connected together using some form of interlayer shear connectors either positioned discretely or continuously along the span of the floor. The interlayer connections provide the desired composite action, where the degree of this composite action is governed by the types of connection system used in the concrete-timber composite system. This analytical method. The analytical method here refers to a design procedure known as Gamma Method recommended in the Eurocode 5 Annex B for bending stiffness, $(EI)_{eff}$. This quantity obtained experimentally and analytically was compared and the reliability of Gamma method was investigated.
concrete-timber composite system. In this method, the specific quantity to express the structural performance of the composite system is the effective paper presents the preliminary study on the structural performance of a proposed triple-T precast concrete-timber composite floor subjected to collapse load. The structural performance derived experimentally through four-point bending collapse test was compared to the same performance established using

I. Introduction

The basic concept of concrete-timber composite (CTC) floor system is to exploit the advantages of timber and concrete in a system as a whole. Basically, timber and concrete are effective in resisting tension stress and compressive stress, respectively. Due to the advantages provided by this system, application of reinforcement bars can be neglected, where this material can be replaced with timber. At the same time, the thickness of the concrete slab can be reduced greatly up to 65% of the typical slab thickness. The concrete-timber system has been developed and researched since the past 50 years. This further study of this system was induced by the wide usage of timber members in the European countries. The interest in the system has grown resulting in the construction of bridges (United States, New Zealand, Australia, Switzerland, Austria, and Scandinavian countries), upgrading of timber floors (Europe), and construction of new buildings [1]. The floors designed at earlier times did not comply with current regulations and failed to meet the requirements of building physics with regard to sound insulation and fire resistance [2]. Composite floors can easily withstand design live loads higher than the standard 1.92 kN/m^2 (40 lb/ft^2) [3].

II. Significance

Initially, the applications of CTC were originally developed for bridges and upgrading existing timber floors mainly as a refurbishment and maintenance technique. The CTC system is popularized in the refurbishment of old historical buildings in different European cities such as Leipzig in Germany during in the late 19th century [4]. Use of wood replaces non-renewable resource-based concrete and steel rebar with a managed renewable resource. The cost benefit of a solid wood–concrete floor system is evidenced by successful commercial projects, which have occurred in Europe. Among them are a 4-storey school building and a 5-storey apartment building constructed in Europe [5]. Sometime a lot of maintenance also has to be considered as this procedure should follow accordingly either using concrete or timber [34], [35], [36] and [37].
Basically, in a timber-concrete composite structure the concrete topping mainly resists compression, while the timber joist resists tension and bending, and the connection system transmits the shear forces between the two components. Advantages over wooden floors include the increased load-carrying capacity, higher stiffness (which leads to reductions in deflections and susceptibility to vibrations), improved acoustic and thermal properties, and higher fire resistance. There are also some advantages relative to normal reinforced concrete floors, notably cracks in the tensile region of reinforced concrete slabs may promote penetration of moisture and corrosion of the steel re-bars. Further, the lower part of a concrete slab (40–60% of the depth) is generally ineffective since it is cracked and thus non-resistant. By replacing that part with a resistant solid wooden deck, the overall depth of the concrete slab can be reduced by about 50% and, thus, the self-weight of the structure can be markedly decreased [6].
In terms of cost, the CTC system is a much economical method as compared to the cast-in-situ concrete slabs. The high cost of cast-in-situ concrete slabs, mainly due to the cost of transporting fresh concrete and the use of props, formwork such as planks/particleboard or plywood sheets as composite components, which further increases the self-weight of the structure, and use of separating layer-foil

between the concrete and timber to prevent the timber coming into contact with wet concrete [7].

III. Objectives and Scope

The main objectives of conducting the experiment and analysis regarding on the performance of the triple-T-beam precast-CTC floor system are as follows: (1) To investigate the structural behavior; (2) To derive and compare the experimental and analytical effective bending stiffness quantity at ULS and SLS; and (3) To investigate the structural performance at ULS and SLS.

The scope of the study was limited to a several limitations. Experimental refers to four-point bending test to collapse and analytical refers to the design method known as the gamma method. The timber sizes available from the local suppliers offer nominal sizes. The nominal sizes of the timbers used were of 50 mm x 100 mm (≈ 2" x 4"), with 3100 mm of span. The self-compacting concrete of Grade 30 was used. The insufficient number of strain gauges limited the researching ability to investigate the stress block pattern of the composite's cross-section. The limited number of linear variable diode transducers (LVDTs) only allowed for the mid-span defection and the overall relative slip of the specimens to be investigated. The experimental performance of the concrete-timber composite system was done in Universiti Tun Hussein Onn Malaysia, Batu Pahat, Johor Darul Takzim, Malaysia.

IV. Construction Technique and Concept

For this construction technique to be efficient, three fundamental design criteria must be satisfied: (1) the neutral axis of the composite cross-section should be located near the timber-concrete interface in order to ensure both components act efficiently with concrete purely compressed and, therefore, uncracked and the timber mostly subjected to tensile stresses; (2) the connection system must be strong and stiff enough to transfer the design shear force and provide an effective composite action; and (3) the timber part (joist/beam or solid deck made from individual planks nailed together on the edge) must be strong enough to resist bending and tension induced by gravity loads applied on the composite beam. Awareness and familiarity with the behavior and design methods of CTC are important for this type of construction to permeate into the building industry [8].

Pre-drilling holes of timber elements up to 60% of the nail diameter was adopted in all specimens of joint to avoid splitting when driving the nails [9].

The notch of the system has to be carefully constructed to ensure that higher stiffness and strength of the interlayer connection can be achieved. Gutkowski et al. (2008) performed a collapse test under 4-point bending of a multiple timber-concrete layered beams connected with notch shear/key anchor details, and the composite efficiency reported to be ranging from 54.9 to 77%. The composite efficiency obtained is relatively low resulting in low performance of the system. This is due to the poor construction of the notch connections [10].

Lukaszewska et al. [11] conducted a short-term collapse test on a total of five 4.8 m span full scale T-section TCC floors with 1600 × 60 mm concrete slab and triple 90 × 270 mm glulam joists under a 4-point bending with the concrete slab of specimens was prefabricated off-site with mounted connectors. Three specimens had lag screws surrounded by steel pipes whilst two specimens had metal plates nailed to the glulam joists. It is reported that from the tests conducted, the composite action is at only 60% and 30% achieved in the beams with lag screws and metal plates, respectively [11]. Yeoh [8] stated that the use of a notched connection together with the steel pipe and lag screw is a possible way suggested by the authors of improving the connection efficiency.

The concept of concrete-timber composite (CTC) is actually the exploitation of the advantages of the timber and concrete's characteristics. Timber is known for its high resistivity against tensile forces (deflection), induced by gravity loads, whereas concrete has high resistivity against high compressive force. Through this exploitation, theoretically, the span of the composite could be lengthen or span till up to ±15m, depending on the degree of composite action between the two constituents. The effectiveness of the composite action is governed by the connections between them. The types of connection applied will vary the result of the product. Furthermore, the thickness of the concrete slab could be reduced to 65 mm thick, or lesser, again, dependent on the composite action. Further reduction of slab thickness would result in the concrete usage to be reduced as well.

Generally known, usage of cementitious material produces carbon dioxide gas that causes the greenhouse effect. This approach of implementing CTC in construction would be sustainable to the environment. Plus, timber is carbon-neutral. Hence, this will give CTC a place in becoming one of the important criteria of the Green Building Index.

V. Gamma Method

There are no particular design standards or specifications available yet by the Malaysia Standard (MS) or the *Jabatan Kerja Raya* (JKR) Malaysia for CTC system to date. Hence, the design of CTC system in this study was in accordance to the Gamma method given in the Annex B of Eurocode 5 (EC5).

According to the Gamma method, an effective bending stiffness, $(EI)_{eff}$, given by Eq. (1), is used to account for the flexibility of the timber-concrete shear connection. A reduction factor γ, which ranges from 0 for no composite action between the timber and concrete interlayers to 1 for fully composite action (and rigid connection), is used to evaluate the effective bending stiffness:

$$(EI)_{ef}=E_1I_1+E_2I_2+\gamma_1E_1A_1a_1^2+\gamma_2E_2A_2a_2^2 \qquad (1)$$

where subscripts 1 and 2 refer to concrete and timber elements, respectively; E is the Young's modulus of the material; A and I are the area and the second moment of area of the element cross-section; a is the distance from the centroid of the element to the neutral axis of the composite section; and γ is the shear connection reduction factor.

Using the effective bending stiffness, the maximum stresses in bending, tension and compression for both the timber and concrete elements, and the shear force in the connection can then be calculated. In Eq. (1), γ_1 is calculated from Eq. (2) and γ_2 is taken as one:

$$\gamma_1=\frac{1}{1+\frac{\mu^2E_1A_1s_{eff}}{Kl^2}} \qquad (2)$$

$$\gamma_2=1 \qquad (3)$$

where s_{eff} is the effective spacing of the connectors assumed as smeared along the span of the floor beam, and is the total of $0.75s_{min}$ and $0.25s_{max}$, where s_{min} and s_{max}, are the minimum and maximum spacing of connection, respectively (Fig. 1); l is the span of the TCC floor beam; and K is the slip modulus of the connector. For verifications at ULS and SLS, different values of slip moduli, K_u and K_s, are used, defined by Eq. (4) and Eq. (5), respectively. Such a difference between K_u and K_s arise from the shear force-relative slip relationship of the connection, which is generally nonlinear. These stiffness properties of connector are evaluated through experimental push-out shear test carried out as recommended in EN26891:

$$K_u=\frac{0.6F_m}{v_{0.6}} \qquad (4)$$

$$K_s=\frac{0.4F_m}{v_{0.4}} \qquad (5)$$

where F_m is the mean shear strength obtained from a push-out test, $v_{0.4}$ and $v_{0.6}$ are the slips at the concrete-timber interface under a load of 40% and 60% of the mean shear strength F_m, respectively.

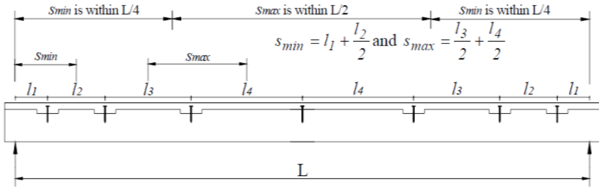

Fig. 1. Typical TCC beam showing indicative spacing of notched connection for the definition of s_{min} and s_{max} [8]

VI. Loading Regime

It is important to eliminate any internal friction that exists between the concrete and the connectors in order to obtain pure connection strength in the floor system, and to ensure any initial slip that takes place in the connection does not affect the overall result. Hence, the European Standards has procedure in terms of loading regime to eliminate the stated friction force from the connectors.

The loading procedure shall be as shown in Fig. 2 and the idealized load-deformation curve is shown in Fig. 3.

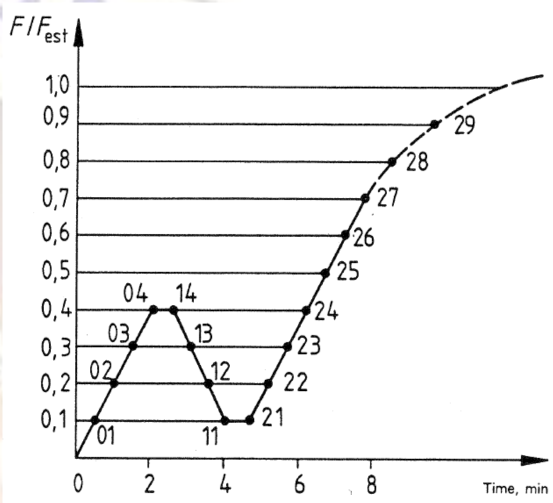

Fig. 2. Loading procedure [12]

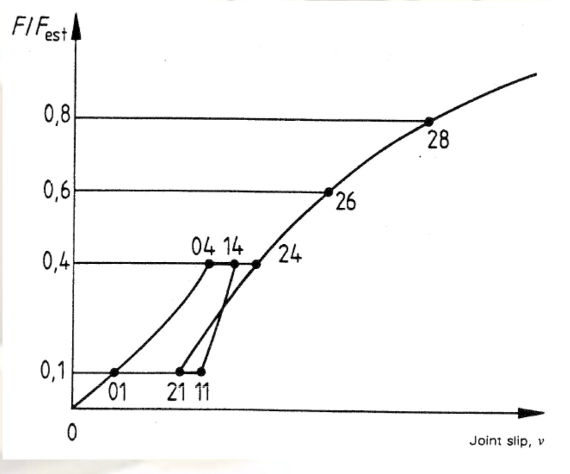

Fig. 3. Idealized load-deformation curve and measurements [12]

Before the loading is applied to the specimen, the maximum load, F_{est}, has to be estimated. F_{est} is estimated on the basis of experience, calculation or preliminary tests, and should be adjusted as required [12].

The loading shall be applied up to 0.4 F_{est} and maintained for 30 s. The load shall then be reduced to 0.1 F_{est} and maintained for 30 s. Thereafter the load shall be increased until the ultimate load or slip of 15 mm is reached.

VII. Interlayer System of Concrete-Timber Composite

VII.1. Partially Composite Action

According to Gutkowski et al. [5], the behavior of a layered composite cross section is bounded by two stiffness limits. The upper limit is that of "full composite action". The cross section has a single neutral axis and the two flexural strains are identical at the interface. The transformed section method applies for analysis. The lower limit is that of "full non-composite action", whereby no horizontal force is transferred between the two layers by either mechanical bond or friction. The layers have individual neutral axes and discontinuous flexural strain at the interface. When mechanical fasteners or adhesives interconnect the layers, flexure causes the layers to experience some horizontal motion (slip) at the interface. The behavior is referred to as "partially composite action". The single neutral axis splits and as slip between the layers increases the two neutral axes move farther and farther apart. Slip reduces the efficiency of the cross section below the levels of strength and stiffness present in a nonslip situation. The degree of composite behavior is function of the interlayer force developed by interconnection.
Therefore, it is to be claimed that the CTC flooring system in this study is of a partially composite action due to the difference of material types. However, in this thesis, the 4-connector CTC slab system resembled as the partial composite action, also, a study on high composite action, the 10-connector slab system, was carried out to investigate its behavior subjected under 4-point bending in terms of the economics of the construction.

VII.2. Connectors of the System

Basically, the composite action of the CTC flooring or decking system strongly depends on the connections between the two different constituents in order to provide a higher resistance towards deflection, hence, increasing the span of the member. For this thesis, the notches used were of a symmetric "V" shape. The notches were cut into shape on the timber joists perpendicular to the span of the system to provide the required composite action, and high-strength nails of Ø12 mm in diameter and 100 mm in length were driven and embedded half-length into the notches.
O'Neill [13] claimed that the mentioned type of connection by right is one of the best connections for the system with respect to strength and stiffness performance. A triangular shaped notch also performed almost as well as a rectangular notch, thus making it one of the more viable options as it is much easier to manufacture [13].
Anchorage capacities in timber are dependent on the dimensions of the screws, nails, or notches and the material properties of the timber and anchor. Design equations for shearing off or pulling out anchors exist, as well as equations for the shear failure of concrete notches [3].
The relative slip between the concrete slab and the timber beam can be prevented by direct bearing of the concrete within the notch on the timber of the beam, leading to a strong and stiff connection.
Yeoh et al. [2] has conducted experiments using different notch geometries (rectangular, triangular, inverted trapezoidal) have been used with different timber materials (i.e., sawn timber, Glulam, and LVL), with or without reinforcement. Fig. 4 illustrates the notches cut in the timber joist and reinforced with a steel screw.

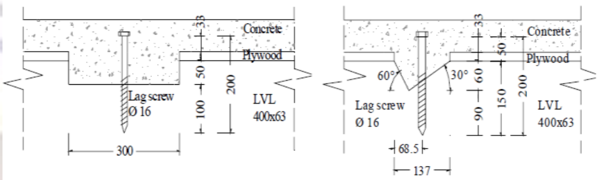

Fig. 4. Details of the rectangular (left) and triangular (right) notched connections tested at the University of Canterbury (dimensions are in mm) [14]

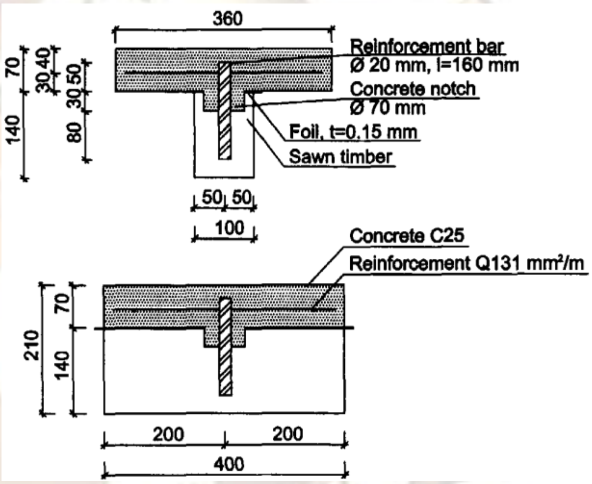

Fig. 5. Concrete notch with reinforcement [15]

As shown above in Fig. 5, the notches cut in the timber beam and reinforced with a steel screw or dowel, is by right the far most effective connection for CTC with respect to the strength and stiffness (slip moduli) although it may not be altogether economical if the notches had to be cut manually [15]-[18].
Yeoh et al. [19] stated that the length of the notch, the presence of a lag screw and its depth of penetration into the LVL, were found to be the most important factors affecting the performance of the connection.
It was found that the notch length affects the strength and stiffness of the connection while the lag screw provides ductility and improves the post-peak behavior [19].
Alternatively, mechanical connectors such as nailplates (Figure 2.15) that do not require any cutting in the timber can be used [19],[20].
According to Balogh and Gutkowski [21] and Kuhlmann and Michelfelder [22], these connectors

(i.e, nailplates) were found to be efficient in strength and stiffness although significantly less than a notched connection. An important difference between mechanical and notch connections is that in the first case the slip modulus largely depends upon the flexibility of the fastener and the timber in contact with the fasteners; in the second case, conversely, the slip modulus mostly depends on the stiffness of the wood in the inclined surface of the notch, and also on the stiffness of the concrete inside the notch.

Basically, the connectors in a CTC are usually positioned along the beam according to the shear force distribution so that they are concentrated near the supports where the internal shear force is higher and spaced out gradually into the span as the shear force reduces to zero in the middle for a simply supported beam subjected to a uniformly distributed load.

Gutkowski et al. [10] argued that the structural efficiency of a CTC highly depends on the stiffness of the interlayer connection; a connection system that results in high composite action allows a significant reduction of the beam depth and longer span length when compared with a non-composite system.

VII.3. Load-Slip Behavior of the Interlayer

Ordinary mechanical connectors alone, such as nails, screws, etc., can be very inadequate for the interlayer shear forces involved in the layered mixed material systems. This was demonstrated for ordinary nails via laboratory tests of T-beams comprised of a concrete flange; dimension lumber stem, and nailed interlayer connection, with only 10 – 20% efficiency was observed for ordinary nails [23].

Gutkowsi et al. [5] claimed that in wood-wood-layered systems connected by mechanical fasteners or glue (no notch), the load versus interlayer slip behavior is nonlinear (Fig. 6).

From Fig. 6, for the notched shear key anchor connection, the load versus interlayer slip behavior is linear up to initial brittle failure in either layered material. Then it drops off to much lower residual resistance, due to the connector subsequently resisting shear. The pilot study carried out by Thompson [24], showed the notched shear key anchors had a higher slip modulus in comparison to two other details based on mechanical connectors that transfer shears.

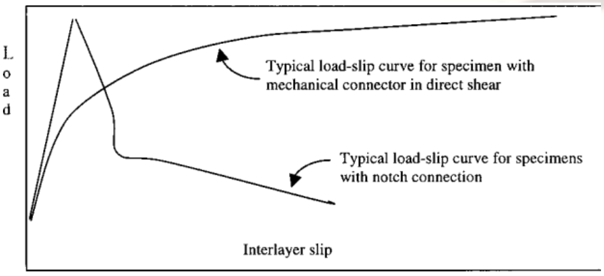

Fig. 6. Typical load-slip curves [5]

The slip modulus is used to characterize the flexibility of a connection. It is defined by the ratio between deformation and force and is influenced by the type of connector and its characteristics, for example size, strength and the fitting accuracy. The slip modulus of a timber concrete composite system is required for many design methods and can be determined from experimental tests according to ISO 6891 [25]. This method was not used for the current tests.

It has been experimentally proven by past researches and development that the stiffness of inclined screws used as shear connectors in concrete and timber composite (CTC) floors is much efficient in their performance behavior. Screws inclined in the direction of slip have been shown to provide a higher stiffness (slip modulus) than vertically placed screws. An increased slip modulus per screw enhances the effective flexural stiffness of a partially composite action of the timber and concrete combination, or, alternatively, allows the same flexural stiffness to be achieved with fewer screws.

VIII. Behavior of Concrete-Timber Composite Subjected to Loading

VIII.1. Loading at Ultimate Limit State and Serviceability Limit State

Determination of loading at ultimate limit state (ULS) and serviceability limit state (SLS) in the short term is essential to interpolate the corresponding slip moduli. The slip modulus is responsible to determine the reduction factor for the concrete constituent, γ_1, which eventually affects the final outcome of the effective bending stiffness, $(EI)_{eff}$.

According to Cecotti et al. [26], the load at ULS is estimated using the formula $2P_u = (f_d/f_m) \times 2P_c \times k_{mod}$ for beams with fracture tensile failure (in the case of well-designed beams) or $2P_u = (R_k/R_m) \times 2P_c \times k_{mod}/\gamma_m$ for beams with connection shear failure (in the case of under-designed beams). The SLS load is estimated by $2P_s = 2P_u/\gamma_Q$.

VIII.2. Short-Term Loading

In order to account for CTC slab's bending stiffness by computation using the γ-method, the connectors slip modulus needs to be verified from the short-term "push-out" tests. All shear tests were arranged as "push-out-tests" with specimens consisting of a centrically located timber part and two bordering concrete layers. Short-term shear tests have been performed to determine the stiffness and the ultimate load of each joint at the initial state. Collapse tests are important to quantify the actual composite action of the system, the load-bearing capacity and the failure mechanisms. There is in general a close relationship between the collapse load and the failure mechanism, and the type of connection system.

A push-out test of the connection should always precede a beam collapse test in order to obtain important information on the mechanical properties of the connection [14].

Yeoh [14] summarized the state-of-the-art about short-term collapse test of CTC in recent years in Table I.

TABLE I
SUMMARY OF THE STATE-OF-THE-ART ABOUT SHORT-TERM COLLAPSE TEST OF CTC IN RECENT YEARS [14]

Reference	Test description	Remarks
Grantham et al. [27]	Existing timber floor in a full-scale light-frame multi storey platform building converted to a TCC floor using SFS connectors	Long-term test of 34 days under 2.5 kN/m^2 live load with deflection limit of span/333 met and structural collapse at 11.9 times the design imposed load.
Persaud and Symons [28]	7.3 m span 2 m wide T-beam built from 160 x 405 mm glulam with 10 vertical lag screw connectors tested under 3-point bending. Collapse load was 173 kN with mid-span deflection 74.9 mm and maximum end slip 5.7mm.	The composite system was more than 3 times stiffer than and almost twice as strong as the one without composite action. Experimental results were compared to 3 different prediction methods: rigorous elastic solution, gamma method and elasto-plastic method. Gamma method was found to overestimate the experimental ultimate strength by 20% while elasto-plastic method showed the closest estimate.
Clouston et al. [29]	Solid glulam deck-concrete system, 10 m long 960 mm wide and 340 mm deep with 3 rows of continuous steel mesh along span, each 1 m long tested under 4-point bending. Ultimate failure occurred at 291 kN with a maximum deflection of over 80 mm.	Near full composite action of the system was reported in the conclusion with a 97% effective bending stiffness with respect to that of a full composite system. Using the gamma method, the failure load was estimated as 312 kN compared to 315 kN for full composite action which is just 1% less.
Ceccotti et al. [26]	Double 6 m span, 1.5 m wide, built from two 125×500 mm glulam T-beam, with 18 corrugated rebars glued to each beam with epoxy resin. Beam was twice loaded and unloaded prior to 4-point bending collapse test after a 5-year long-term monitoring. Collapse load was 2P = 500 kN with a 33.2 mm and 2.47 mm of maximum deflection and end slip, respectively.	Beam collapsed at 2.44 times the service design load due to tension failure in timber with a very stiff behaviour. Composite efficiency of 87 to 93% was reported. No significant plasticization of connection system reported. Experimental results were compared to analytical solutions using the gamma method with the connection secant stiffness at $K_{0.4}$, $K_{0.6}$ and $K_{0.8}$ corresponding to service, ultimate and collapse level, respectively obtained from push-out test. The collapse load was better approximated using $K_{0.8}$.
Gutkowski et al. [10]	Multiple timber-concrete layered beams connected with notch shear/key anchor details, each with a clear span of 3.51 m, were tested to collapse under 4-point bending.	The composite efficiency reported ranged from 54.9 to 77%. The failures were characterized by flexural tensile in the wood. Poor construction of the notch connections resulted in low performance of the system.
Lukaszewska et al. [11]	Five 4.8 m span full scale T-section TCC floors with 1600×60 mm concrete slab and triple 90×270 mm glulam joists tested to failure in 4-point bending. The concrete slab of specimens was prefabricated off-site with mounted connectors. Three specimens had lag screws surrounded by steel pipes whilst two specimens had metal plates nailed to the glulam joists.	Composite action of only 60% and 30% achieved in the beams with lag screws and metal plates, respectively. The use of a notched connection together with the steel pipe and lag screw is a possible way suggested by the authors of improving the connection efficiency.
Yeoh et al. [14],[30]	11 LVL-concrete composite T-beams with 8 and 10 m span, 600 and 1200 mm widths, and notched coach screw and toothed metal plate connections were tested to failure under 4-point bending. Low shrinkage and normal concrete was used.	6 beams were well-designed and 5 beams were under-designed for the targeted non-structural permanent load of 1 kN/m^2 and imposed load of 3 kN/m^2. Composite action of 87.6 to 99.23% at SLS was achieved. Well-designed and under-designed beams collapsed at a range of 2.29-2.91 and 1.17-2.31

Reference	Test description	Remarks
		times the ultimate design load, respectively.

IX. Review of Lightweight Concrete Application in Concrete-Timber Composite Floor System

The traditional practice of lightweight construction is that it is crucial to minimize the dead load on floors in order to use wood frames. The application of lightweight concrete provides an interesting variation in the technology of CTCs. The dead load of a CTC's floor can be reduced 15% by using a lightweight concrete with a density of 1.6 kN/m^3 (100 lb/ft^3) instead of a normal weight concrete of 2.3 kN/m^3 (145 lb/ft^3).

However, the failure mechanisms in lightweight concrete for the wide range of differently shaped connectors have not been fully investigated. The application of lightweight concrete requires an evaluation of the potential of the connectors with regard to the anchorage in concrete.

According to Steinberg et al. [3], the failure mode of a wood connection while using normal weight concrete is predominantly due to shearing off or withdrawal of the connector from the wood. This type of failure can be controlled by altering the dimensions of the screws. In the tests performed as part of this study, only a failure of the concrete occurred.

Further stated by Steinberg et al. [3], timber-lightweight concrete composite structures are affected by the modulus of elasticity of the lightweight concrete which leads to a lower effective bending stiffness of the structure. Consequently, the connectors have to be positioned at a closer spacing in lightweight concrete compared to normal weight concrete. The design, therefore, depends on the compromise between the higher cost due to the use of lightweight concrete and the closer setting of the connectors, and the reduction in permanent load.

Koh et al. [31] tested 12 push-out specimens built from lightweight foamed concrete and Malaysian hardwood connected using different types of nails. Yeoh [14] claimed that higher grade of lightweight concrete was recommended in order to fully exploit the efficiency of the connection.

X. Advantages of the Concrete-Timber Composite System

A typical concrete slab would require reinforcement bars for it to sustain the tensile stress experienced. The cost of the typical member itself is high or costly due to the high market price of steel, what more if a longer span is to be constructed. In addition to the disadvantages, a conservative design thickness of a slab would be ranging from 150 – 200 mm, but with concrete-timber composite system, the slab thickness can be reduced to 65mm, which is a reduction of about 65% of the typical thickness.

For timber, where traditional light timber frame floors alone may suffer from excessive deflection, susceptibility to vibrations, insufficient acoustic separation, and low fire resistance.

Again, all of these problems can be resolved by using concrete-timber composite floor system. There are many advantages of this floor system over timber-only or reinforced concrete floors.

Cost advantages in comparison to reinforced concrete floors are based on the use of the timber both as permanent formwork and as a construction element, the opportunity to use the space under the floor earlier, and the option to leave the ceilings with the generally same rustic appearance for renovation projects.

From an ecological standpoint, TCCs are superior to reinforced concrete floors [32].

According to Yeoh et al. [2], for new buildings, by connecting an upper concrete slab to timber joists and beams it is possible to: (1) significantly increase the stiffness compared to timber-only floors; (2) considerably improve the acoustic separation; and (3) increase the thermal mass, important to reduce the energy consumption needed to heat and cool the building.

Furthermore, Yeoh et al. [2] mentioned that, on the other hand, by replacing the lower part of a reinforced concrete section, ineffective due to the cracking induced by tensile stresses, with timber joists or a solid timber deck, it is possible to achieve the following advantages: (1) rapid erection of the timber part, particularly if prefabricated off-site, due to its lower weight, with a function of permanent formwork for the concrete topping; (2) reduced load imposed on foundation; (3) reduced mass and, hence, seismic action; (4) possibility to use the timber as a decorative ceiling lining; (5) low embodied energy; and (6) reduced CO^2 emissions since timber is carbon-neutral. For refurbishment of old buildings, the following advantages can be obtained by connecting a concrete topping of about 50 mm to the existing timber floor: (1) increased stiffness and load-bearing capacity; (2) preservation of historical buildings for future generation; and (3) better seismic performance due to the improved diaphragm action. TCC floors are significantly lighter and more economical when compared to their counterparts, reinforced concrete and steel-concrete composite floors, which are characterized by non-regenerative manufacturing process with high energy demand and high emission of carbon dioxide.

XI. Research Program

Basically, the purpose of this study is to investigate the experimental performance of a triple-T concrete-timber composite floor system under sheltered condition. The nominal size of each timber joist/beam is 50 mm x 100 mm, with a span of 3100 mm. The thickness of the slab used was 65 mm. The concrete slabs were reinforced both ways with 205 mm x 205 mm BRC 'A' steel mesh in order to control the cracking. There were a total of four (4) tests that are conducted with conditions such as follows:

1) Specimen A – Using only timbers as the beams to support the loading subjected on them (triple beam);
2) Specimen B – Triple-T precast-concrete-timber composite floor system without any composite action (no connection system);
3) Specimen C – Triple-T precast-concrete-timber composite floor system with partial composite action having 4 connections on each timber; and
4) Specimen D – Triple-T precast-concrete-timber composite floor system with high composite action having 10 connections on each timber.

Experimental refers to 4-point bending test to collapse by means of loading by the Magnus frame (Fig. 7) and analytical refers to the design method known as the Gamma method, which is recommended in the Eurocode 5 Annex B for the design of concrete-timber composite system.

Fig. 7. The Magnus frame with the 4-connector CTC slab system specimen placed on it

Fig. 8 summarizes the research methodology, in achieving the objectives of the study, into a form of flowchart.

Fig. 8. Flowchart of the entire research methodology

XI.1. Timber Notch Construction

Three timbers of length 3100 mm with actual cross-section of 45 mm (width) by 95 mm (depth) were used as the tensional resistance for the slabs. Each and every timber had notches, except for Specimen A and B (no connection system). Each notch was of a symmetrical V-shape and has dimensions of 150 mm (base) and 30 mm (height). High-strength nails of 100 mm in length were used as the connector for the system. The nails were embedded at half-length into the timber notches. The timbers were pre-drilled at about 60% of the nail's diameter before the nails were embedded into the timbers. The positions of the notches were calculated using Microsoft Excel spreadsheet. Basically, the notches were positioned at the centroid of the shear force obtained from the shear force diagram of the concrete-timber composite system. Table II summarizes the position of notches for half of the timber span from the point of support. Fig. 9 illustrates the configuration of notch connection system with dimensions displayed, whereas Fig. 10 shows the notches from a close-up view.

TABLE II
NOTCHES POSITION FOR ONE-HALF OF THE TOTAL SPAN FROM THE POINT OF SUPPORT

No. of notches on one-half of total span	Distance of notches according to the position from the point of support (m)				
	1st	2nd	3rd	4th	5th
2	0.212	0.766	-	-	-
5	0.077	0.240	0.430	0.667	1.018

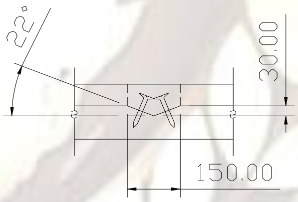

Fig. 9. Geometry of the V-shape notch

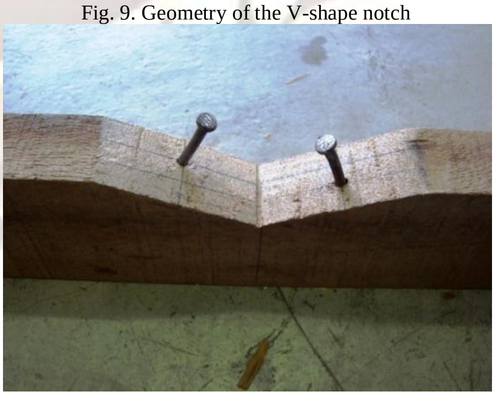

Fig. 10. The timber notch from a close-up view

XI.2. Precast Self-Compacting Concrete

A total of three precast concrete slabs were casted on site, for specimens B, C and D, respectively. The type of concrete used for the casting of the slabs is by using the mix design of foamed concrete, or also known as the self-compacting concrete. The grade of the concrete determined from the concrete cube test was averaged at 30 MPa. The slump test for the self-compacting concrete was recorded at 240 mm (Fig. 11). The mix compositions in ration for 1 m^3 of concrete are as follows in Table III.

Fig. 11. Slump test on SCC

TABLE III
COMPOSITION AND PROPERTIES OF THE GRADE 30 SELF-COMPACTING CONCRETE MIXTURE PER METER CUBIC

Mix proportions, kg/m^3	
Ordinary Portland cement (OPC)	400
Coarse aggregate	928
Fly ash	900
Fiber glass	1
Fluid volume, L/m^3	
Water	21.5
Superplasticizer	4
Water-proofer	30
Properties	
Slump, mm	240
28-day compressive strength, MPa	30

The slabs are of 65 mm thick. The 6 mm diameter BRC 'A' of 205 mm x 205 mm grid size was used as crack control which is positioned approximately 30 mm from the bottom of the slab before the casting session being carried out. An approximate of 30 mm (in height) spacers were used to lift and hold the BRC in position in order to provide the adequate concrete cover.
Polystyrene blocks measuring at approximate of the timber notch dimension at plan view, and the height was approximately 70 mm, were used as moulds to form connection pockets on the concrete slabs for the purpose of aligning the connectors on the timbers with the slabs (Fig. 12).

Fig. 12. Polystyrene blocks as the mould to form connection pockets on the concrete slabs

The pockets were formed as holes to fit along with the timber notches. The concrete slabs were left to undergo the curing process for full 28-day duration (Fig. 13). Formworks were removed before the positioning of the slabs on the timber beams.

Fig. 13. Concrete slabs left to cure for 28 days from the day of concrete casting session with the polystyrene blocks resting in place

XI.3. Grouting of Connection Pockets

The 65 mm concrete slabs were lifted and placed on their respective timber beams with the connection pockets positioned and aligned correctly with the notches on the timbers. Before the attachment of the concrete slab and timbers, Plywood was cut to pieces accordingly and nailed to the sides of every of the timber notches in order to avoid leakage of grout material during grouting (Fig. 14). The pockets of the slabs were grouted with high strength epoxy resin compound, the Sikadur-330.

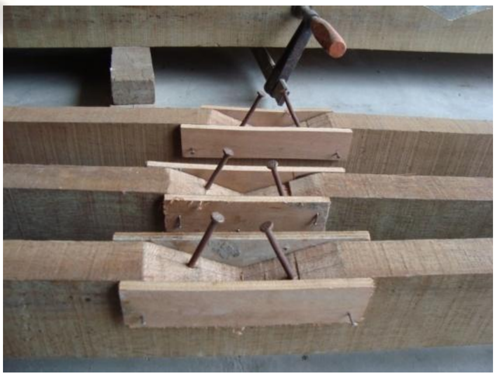

Fig. 14. Plywood pieces nailed to the sides of the timber notches

Fig. 15 shows the attachment of the concrete slab to the timber joists with one of the holes still exposed.

Fig. 15. The holes of the concrete slab aligned accordingly along the timber notches with the shear connectors still visible to sight from the side view

Sikadur-330 compound is a 2-part epoxy impregnation resin, namely Part A – resin and B – hardener. Part A has a white pigmentation, whereas Part B has grey pigmentation. Both parts were mixed with a proportion of 4:1 by weight (i.e., Part A: Part B). The mixture forms a compound of light grey in color. All dust, debris, loose and friable materials were completely removed from all the contact surfaces before pouring the Sikadur-330 mixture. The mixed was then poured into each of the holes and was left to cure. The leakage of grout was safely guarded by the plywood pieces. The room temperature of the laboratory at the time of curing of the Sikadur-330 was recorded at 33°C. Fig. 16 shows the condition of the hardened epoxy resin fitted in one of the connection pockets.

Fig. 16. Hardened composition of Sikadur-330 from plan view

From the prescription manual of Sikadur-330, it takes 2 days to achieve full cure at a temperature of 35°C. The total mass of the specimens was resulted at an approximate of 220 kg each.

XI.4. Test Setup

The specimens were subjected under 4-point bending to failure, by means of the Magnus frame. The loading points were placed at an approximate of 1/3 and 2/3 of the total effective span, 2900 mm, configuring a 4-point bending test on the systems, which are exactly on the points of 967 mm from the points of supports. The supports were of a pin and roller type.

Triple-I steel loading beam was used to give the desired configuration of loading. Load cell of 500 kN capacity was used to record the loading subjected by the 1000 kN capacity actuator on it and then, to the slab via two point loads as mentioned. Fig. 17 illustrates the overall configuration of the test program.

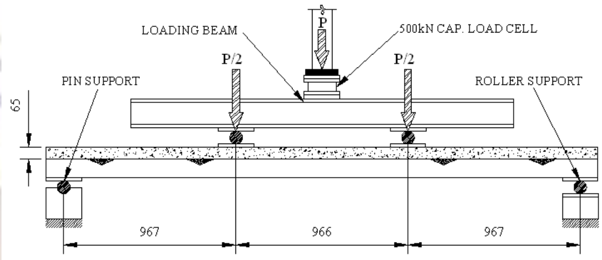

Fig. 17. Overall test configuration

The loading procedure used in this study was as accordance to the EN 26891 [12]. Before loading the specimens, the maximum loading was estimated, F_{est}, for every test, as in Table IV below

TABLE IV
F_{est} FOR EACH TEST SPECIMEN

Test Specimen	F_{est} (kN)
No-connector	30
4-connector	75
10-connector	150

XI.5. Analytical Study Program

The analytical study on the bending stiffness for every specimen was computed by applying the Gamma method as expressed in Chapter V. The analytical bending stiffness, $(EI)_{theo}$ was compared to the experimental bending stiffness, $(EI)_{expt}$. The comparison is to check on the reliability of the Gamma method relevant to the actual condition. An Excel spreadsheet was used as an aid to compute for the analytical values.

The values of SLS and ULS slip moduli, $K_{0.4}$ and $K_{0.6}$, obtained by Ooi [33] was used in the computation procedure. Ooi conducted push-out tests for the connection similar to that of the CTC floor system studied in this thesis. The slip moduli at SLS and ULS of the tests are 2.29 kN/mm and 2.10 kN/mm [33].

The analytical bending stiffness value computed by applying Gamma method was a representative for a single-T-section. The value was multiplied by 3 in accordance to the triple-T-section.

XI.6. Summary of the Methodology

The structural performance derived experimentally through four-point bending collapse test will be compared to the same performance established using analytical method. The analytical method here refers to a design procedure known as Gamma Method. In this method, the specific quantity to express the structural performance of the composite system is the effective bending stiffness, $(EI)_{eff}$. This quantity obtained experimentally and analytically was compared.

Finally, the structural performance of the floor system was investigated by means of comparing the experimental-analytical values obtained for the quantities of effective bending stiffness and also the safe imposed load capacity. The comparisons were in relation to the effective spacing, s_{eff}, of the connectors.

XII. The Structural Behavior of Concrete-Timber Composite

XII.1. Relationship between Load and Mid-Span Deflection

The number of connectors available on the composite slab also indicates the degree of composition for the connection. Here, evaluating in terms of the degree of composition, the 10 connectors being the fully composite element, followed by the 4 connectors as partially composite, and the no connector as having no composite action. In other words, the no connector composite slab acted as the control of the tested specimens, in terms of having no connection system to produce the composite action as claimed in the literatures.

From the results plotted as in Fig. 18, it is clear that the composite slab with 10 connectors on each timber beam has the ability to sustain the highest load, by means of 4-point bending, which was recorded at 113.26 kN, with a maximum mid-span loading of 38.70 mm, before the point of failure. The magnitude of load being able to withstand by the composite slab having 4 connectors was recorded at 61.04 kN having a 33.80 mm mid-span deflection. The control specimen, having no connection system, recorded a load value of 23.38 kN and a maximum mid-span deflection of 36.43 mm.

For the 4-connector composite slab, there was a strength recovery from the load drop after being recorded at the point of 60.20 kN, as observed from the load-deflection plots. After the recovery, the load increased to P_{max} which was at 61.04 kN, and failed eventually. At the point of drop, a "thump" was heard, and that situation occurred was due to the failure of the outer connectors. The recovery of strength relates to the load that was equally sustained by all the connectors initially, began to transfer to the inner connection, which was towards the mid-span.

By computation, as compared to the no-connector slab system, the percentage of reduction in mid-span deflection for the 4-connector CTC slab system is 7.23%, whereas for the 10-connector CTC system, the computed reduction in percentage is 9.20%. Based on the computed percentages, the 10-connector slab only exhibited a difference of 1.97% from the 4-connector. The computation explains that as the number of connector increases, it would be expected that the trend would end up being a plateau. In other words, there would not be any differences in the control of mid-span deflection after reaching the optimum point of number of connectors.

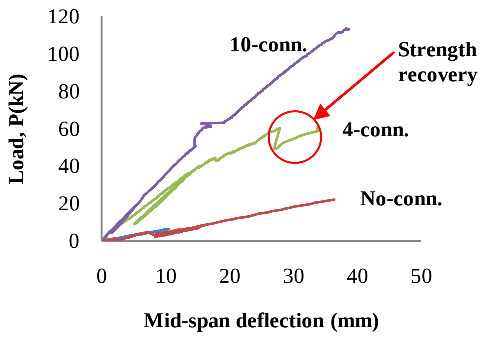

Fig. 18. Experimental load-deflection plots of the tested specimens

XII.2. Relationship between Load and Relative Slip

The relative slip in this study was accounted as an average for the whole specimen, instead of investigating the slip of every connection of the slabs. Based on the plots as in Fig. 19, the degree of composition of the CTC slab controls the relative slip at the interlayer in a much controlled fashion. For the 10-connector slab specimen, also as the specimen having full composite action had a maximum relative slip of 2.97 mm at the event of maximum loading (i.e., the point of failure of the slab).

For the 4-connector slab, after the relative slip being recorded at 0.84 mm (i.e., the highest relative slip value recorded for this slab), the curve deferred to an unexpected route. This situation may be due to the technical malfunction of the LVDTs responsible for the recording the progress of relative slip at the moment. By right, the curve was expected to increase up till the point of failure, which was at 61.04 kN.

Judging by the gradient of the 4-connector and 10-connector specimens, both curves relatively had almost the same gradient until the point where the 4-connector slab's curve deferred off the expected route. This could be argued that the connection system of the mentioned specimens had the same degree of composite action based on their control over the relative slips, respectively, regardless of their number of connectors found on the system.

As for the no-connector slab, the curve of the load-relative slip plot showed irregularity of the variables' relationship. The occurrence of this irregularity can be concluded as the shear connectors responsible for the tying of concrete and timber was not available for the no-connector slab. Both of the constituents can be regarded as independent to one another, instead of working along as a composite.

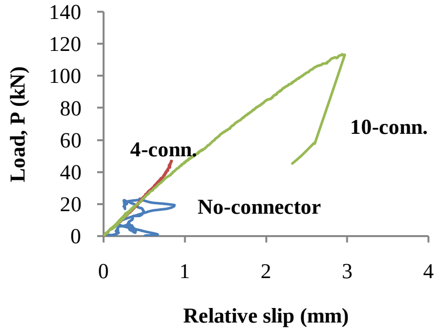

Fig. 19. Experimental load-relative slip plots of the tested specimens

XIII. The Structural Performance of Concrete-Timber Composite

XIII.1. Experimental Bending Stiffness and Safe Imposed Load at Ultimate Limit State and Serviceability Limit State

The concern in this research study is only in the case of under-designed-beams. The connections of the floor system failed due to bending before the timber joists. Hence, the formula from Chapter VII was adjusted accordingly to adapt to the study, where the load at ULS was defined as $P_u = (R_k/R_m) \times P_c \times k_{mod}/\gamma_m$ (in the case of under-designed beams); and load at SLS was defined as $P_s = P_u/\gamma_Q$. Based on Eurocode 5, the load duration modification factor, k_{mod} = 0.8 (where imposed load refer to an office building, for which a medium term load duration was considered); connection strength characteristic/mean ratio, R_k/R_m = 0.83; partial factor for connection, γ_m = 1.25; partial factor for variable action, γ_Q = 1.5; and P_c is the failure load.

Generally, the experimental bending stiffness, $(EI)_{expt}$ was computed by applying the formula of $EI = P_s a/24\delta_s.(3l^2 - 4a^2)$, where a = 967 mm; δ_s is the deflection corresponding to P_s; and l = 2900 mm.

The experimental safe imposed load, Q_{expt}, was computed to display the structural performance of the studied CTC floor system at ULS and SLS, with the actual design imposed load of 3 kN/m^2 from BS8110: Part 2: 1985 was used as the border of acceptance. The basic formula used was derived from BS8110: Part 2: 1985, which is $Q_{expt,u} = (P_u - 1.4G_k)/(1.6A)$ and $Q_{expt,s} = (P_s - 1.4G_k)/(1.6A)$, where $Q_{expt,u}$ and $Q_{expt,s}$ are the experimental safe imposed load at ULS and SLS; A is the slab's surface area; and G_k denotes the self-weight of the CTC floor. Tables V and VI summarize the experimental bending stiffness and safe imposed load capacity at ULS and SLS, respectively.

TABLE V
THE EXPERIMENTAL BENDING STIFFNESS AND SAFE IMPOSED LOAD AT ULS

No. of connector	s_{eff} (mm)	P_c (kN)	P_u (kN)	δ_u (mm)	$(EI)_{expt,u}$ (x10^{11} Nmm^2)	$Q_{expt,u}$ (kN/m^2)
Nil	-	23.38	12.42	21.92	4.91	4.71
4	522	60.20	31.98	12.47	22.20	14.57
10	217	113.26	60.16	15.87	32.80	28.78

TABLE VI
THE EXPERIMENTAL BENDING STIFFNESS AND SAFE IMPOSED LOAD AT SLS

No. of connector	s_{eff} (mm)	P_u (kN)	P_s (kN)	δ_s (mm)	$(EI)_{expt,s}$ (x10^{11} Nmm^2)	$Q_{expt,s}$ (kN/m^2)
Nil	-	12.42	8.28	16.19	4.43	2.63
4	522	31.98	21.32	9.03	20.40	9.20
10	217	60.16	40.11	11.29	30.80	18.67

XIII.2. Experimental Bending Stiffness and Safe Imposed Load at ULS and SLS

The effective analytical or theoretical bending stiffness, $(EI)_{theo}$, was determined by applying the Gamma method as per Chapter V. The mean of Young's modulus of concrete and *Kapur* species timber, 25000 MPa and 13700 MPa (MS544: Part 2: 2001), respectively, was used. The deflection values at ULS and SLS were extracted from the experimental and are assumed as fixed variables in order to compute for the analytical safe imposed load, Q_{theo}. Ultimately, the experimental and analytical values were compared and summarized in 4.3.3. Ooi [34] conducted a series of push-out tests on the type of connection used in this triple-T precast-CTC floor system. Tables VII and VIII summarize the experimental bending stiffness and safe imposed load capacity at ULS and SLS, respectively.

TABLE VII
THE ANALYTICAL BENDING STIFFNESS AND SAFE IMPOSED LOAD AT ULS

No. of connector	s_{eff} (mm)	$\gamma_{1,u}$	P_u (kN)	δ_u (mm)	$(EI)_{theo,u}$ (x10^{11} Nmm^2)	$Q_{theo,u}$ (kN/m^2)
Nil	-	-	21.15	21.92	8.64	9.11
4	522	0.016	12.16	12.47	8.73	4.58
10	217	0.037	15.63	15.87	8.82	6.33

TABLE VIII
THE ANALYTICAL BENDING STIFFNESS AND SAFE IMPOSED LOAD AT SLS

No. of connector	s_{eff} (mm)	$\gamma_{1,s}$	P_s (kN)	δ_s (mm)	$(EI)_{theo,s}$ (x10^{11} Nmm^2)	$Q_{theo,s}$ (kN/m^2)
Nil	-	-	15.62	16.19	8.64	6.33
4	522	0.017	8.80	9.03	8.73	2.89
10	217	0.040	11.12	11.29	8.82	4.06

XIII.3. Comparisons among the Experimental-Analytical Values at Ultimate Limit State and Serviceability Limit State

The comparisons of the effective bending stiffness values and the safe imposed load capacities, experimentally and analytically, are summarized in Fig. 20-23.

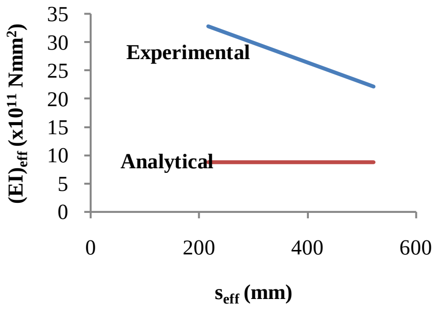

Fig. 20. Experimental-analytical comparisons of the effective bending stiffness, $(EI)_{eff}$, at ULS

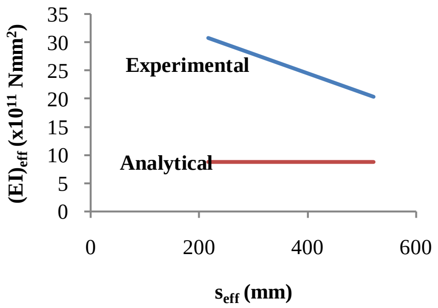

Fig. 21. Experimental-analytical comparisons of the effective bending stiffness, $(EI)_{eff}$, at SLS

At the same time, the effect of varying the effective spacing, s_{eff}, over the bending stiffness and imposed load capacity could be observed.

Important observations to be pointed out from Fig. 20 and 21 are:

(1) The experimental bending stiffness, $(EI)_{expt}$, exhibits a significant change of value with the transition of effective spacing value. However, the trend of the analytical bending stiffness, $(EI)_{theo}$, is almost constant across the effective spacing values.

(2) Analytically, the Gamma method underestimated the effective bending stiffness, which the experimental bending stiffness is at least 2 times larger than the analytical bending stiffness.

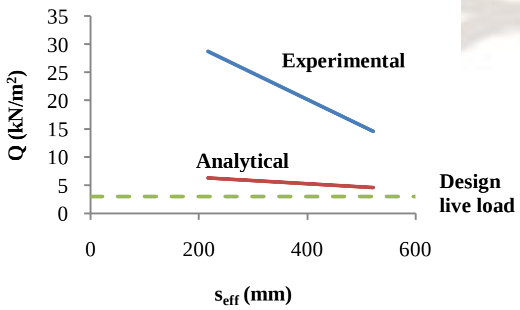

Fig. 22. Experimental-analytical comparisons of the safe imposed load, Q, at ULS

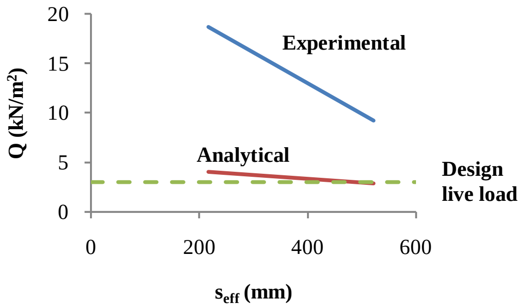

Fig. 23. Experimental-analytical comparisons of the safe imposed load, Q, at SLS

Important observations to be pointed out from Fig. 22 and 23 are:

(1) The smaller the effective spacing of the connections, the higher the number of connectors on the CTC floor system.

(2) Increment in the effective spacing reduces the capacity of the CTC floor in sustaining imposed load.

(3) All experimental safe imposed loads obliged to the standard design value of 3 kN/m^2 and exceeded with relatively large difference.

(4) Analytically, the performance of the CTC in terms of safe imposed load does not show a significant difference from the design imposed load. In other words, the analytical imposed load exhibited close to that of the design imposed load (3 kN/m^2).

(5) Significant differential of safe imposed load by varying the connector's effective spacing, for the experimental values. As for the analytical, a slight difference can be observed by varying the effective spacing.

(6) At both ULS and SLS, all experimental safe imposed load capacities were at least 3 times larger than the analytical capacities, for the case of under-designed beams. This scenario can be concluded that the Gamma method underestimated the short-term ULS and SLS capacities for the 4- and 10-connector system.

(7) The analytical imposed load capacity for the 4-connector floor system, computed at 2.89 kN/m^2, did not pass the requirement of the standard design load at 3 kN/m^2.

The slip modulus values at ULS and SLS which were obtained from the push-out tests on the connection system exhibited approximate equal values. The values of $K_{0.4}$ and $K_{0.6}$ are 2.29 kN/mm and 2.10 kN/mm, which are the corresponding values at SLS and ULS, respectively. The Gamma method applies the reduction factor, γ_1, into computing the analytical bending stiffness. The reduction factor is dependent on the slip modulus values at SLS and ULS. In this case, the final outcome of the estimated bending stiffness and safe imposed load capacity by using Gamma method did not significantly reflect to the experimental trendline. Therefore, the plots for the analytical values resulted in a constant trendline. Tables IX and X summarize the reliability of the Gamma method in the form of ration.

TABLE IX
EXPERIMENTAL-ANALYTICAL RATIO OF EFFECTIVE BENDING STIFFNESS

No. of connectors	s_{eff} (mm)	Effective bending stiffness, $(EI)_{eff}$ ($\times 10^{11}$ Nmm^2)					
		SLS			ULS		
		Exp	Theo	Exp/Theo	Exp	Theo	Exp/Theo
4	522	20.4	8.7	2.34	22.2	8.7	2.55
10	217	30.8	8.8	3.50	32.8	8.8	3.73

TABLE X
EXPERIMENTAL-ANALYTICAL RATIO OF SAFE IMPOSED LOAD CAPACITY

No. of connectors	s_{eff} (mm)	Safe imposed load capacity, Q (kN/m^2)					
		SLS			ULS		
		Exp	Theo	Exp/Theo	Exp	Theo	Exp/Theo
4	522	9.2	2.9	3.17	14.6	4.6	3.17
10	217	18.7	4.1	4.56	28.8	6.3	4.57

XIV. Mode of Failure

From the post-test observations, the timber beams were heavily ruptured for every case from the no-connector to the 10-connector slabs. Tensile cracks were observed on the bottom-side of the beams, and only minor damages, such as fine cracks, were spotted on the concrete topping. This can be explained that the composite system's performance is dominantly contributed by the timber constituent.

XIV.1. No-Connector Floor System

Basically, the no-connector CTC slab does not have the composite action as compared to the 4- and 10-connector. The slab was simply resting on top of the 3 aligned timber beams, without the presence of the shear connector which was responsible in providing the composite action. Based on the load test conducted on the mentioned floor specimen, it has been expected that the maximum loading capacity that could be achieved would be lower as compared to those specimens with shear connectors along their span. The result obtained for the maximum loading recorded was at 23.38 kN, with a maximum mid-span deflection of 36.43 mm.

From the observations of the slab after the loading test was subjected on it, the timber joists received the most severe damage, visually. Tensile crack was spotted at the mid-span of the slab (Fig. 24). The concrete topping had cracks vertically along the side span of the slab due to the bending moment effect (Fig. 25). However, no cracks were spotted on the surface of the concrete topping.

Fig. 24. Tensile crack along the timber beam at mid-span

Fig. 25. Crack spotted on the concrete topping close to the support

XIV.2. 4- and 10-Connector Floor Systems

For the case of the 4 connectors and 10 connectors' composite slabs, the failures at the connections were observed to happen in the inward direction towards the centre of the composite slab. At the point of failure of the connections, the Sikadur-330, epoxy resin grout was still intact. However, there were detachments of the grouting from the timber notches (Fig. 26(a)). The occurrence of this situation was due to the nails being pulled out from the timber, which the nails were initially embedded half-length into the timbers. The nails were in tension before the failure due to the pull-out. In Fig. 26(b) and 27, tensile crack was observed at the further end of the pin support, for the 10-connector specimen.

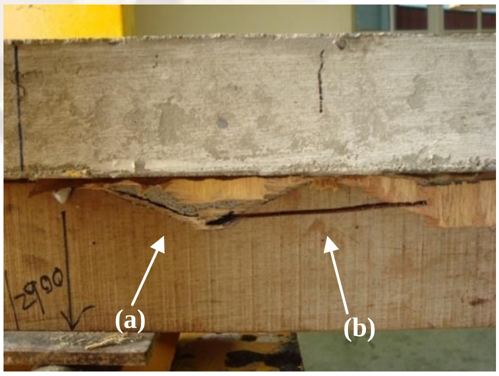

Fig. 26. (a) Detachment of grout from notch; and (b) tensile crack at the connection far end of the slab

Fig. 27. Tensile crack approximately 250 mm in length along the span of 10-connector slab at the pin support: (a) Rear view; (b) Side view

In Fig. 28, for the 4-connector slab specimen, minor hairline cracks were found on the concrete under the point load region.

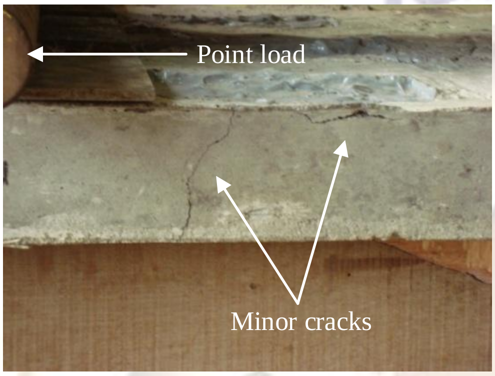

Fig. 28. Minor hairline cracks spotted on 4-connector slab

Cracks were also spotted on the further right towards the roller support, also occurred at the grouted region. From the observation, it seems that the epoxy resin grout was holding firm of the concrete's integrity, protecting the concrete topping from further cracking (Fig. 29).

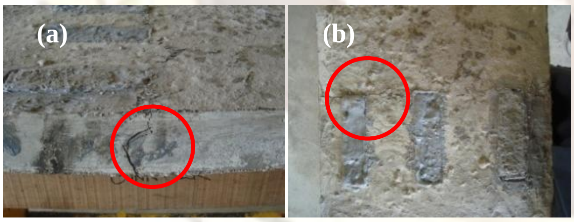

Fig. 29. Hairline cracks (as per marked) at the grouted region close to the roller support: (a) Side view; (b) Plan view

After the loading tests were conducted on the composite specimens, the concrete topping of the slabs were still in a good condition, only left with very minor cracks. Most of the concrete cracks were detected at the region of the connectors, and the cracks occurred in the direction inwards, in sequence, to the centre of the slabs. Neither on the plan nor side views, that defect was detected on the epoxy resin grout, after the testing procedure. Thus, no failure was occurred within the grouted region itself.

XV. Cost and Duration of Project

Table XI below summarizes the approximation of the total project cost for the no-, 4- and 10-connector precast-CTC floor system. Every cost elements are broken down and listed into the table. Table XII summarizes the duration of the construction, for every related job activity, and also the work-force in order to compute the rate in cost per hour per labor. Note that the figures in Tables XI and XII are close approximations.

TABLE XI
SUMMARY OF THE OVERALL PROJECT COST

No.	Items / Descriptions	Qty.	Unit	Cost (RM)
1.	Plywood (9-mm thick)	4	nos.	175.00
2.	Nails (100-mm)	1	kg	10.00
3.	Concrete (Grade 30) + 30%	0.35	m^3	115.00
4.	wastage	9	m^2	80.00
5.	BRC-type A6	9	nos.	225.00
	Timber beams (0.05(B)x0.10(D)x3.50(L)-m)			
6.	Sikadur-330 (Part A+B)	30	kg	2,000.00
			∑ (Total cost) =	2,605.00

Note: The symbols (B), (D) and (L) denote the dimensions in breadth, depth and length, respectively.

TABLE XII
SUMMARY OF THE OVERALL DURATION PER LABOR OF THE PRECAST-CTC FLOOR SYSTEM CONSTRUCTION JOB ACTIVITY

No.	Job activity/description	Labor	Duration (hr)	Duration/ Labor (hr/labor)
1.	Notch cutting – Involved drawing and cutting 42 notches on timber joists	2	16	8
2.	Nail embedment – Involved pre-drilling of holes and driving 84 nails into the 42 predetermined holes	2	8	4
3.	Construction of formworks – Involved cutting of plywood to desired dimensions, forming moulds for the connection pockets and constructing the formworks	2	24	12
4.	Concrete casting – Included slump testing and concrete casting	5	3	0.6
5.	Grouting – Involved attaching cured concrete slabs to respective sets of timber joists and grouting the connection pockets	3	24	8
		∑ (Total) =	75	32.6

Based on the statistic collected from an approximation throughout the whole project, the construction rate of cost was averaged at RM79.91/hour/labor, for a total duration of 75 hours.

Therefore, the construction rate per floor would be averaged at RM26.64/hour/labor, for an average duration of 25 hours.

The construction cost for one floor was relatively high recorded at an approximate of RM675.00 per labor. From Table XI, cost of the epoxy resin, Sikadur-330, gave a huge impact on the overall cost of the project.

XVI. Recommendations

The main objectives of this study were to quantify the structural behavior of the concrete-timber composite floors under 4-point bending, to derive and compare the experimental and analytical effective bending stiffness quantity, and to investigate the structural performance of the floor systems. In addition, there are some discussions on the cost, ease of construction and the economic feasibility of the floor system.

XVI.1. Structural Behavior of Precast-Concrete-Timber Composite Floor System Under Short-Term Loading

The test results show that the degree of composition determines the maximum capacity that a slab could have. The main criteria of the CTC slab system is the presence of shear connectors which give the required composite action. For the case study of this research, the no-connector slab is said to has no composite action, whereas for the 4- and 10-connector CTC slab system are assumed to have partial and full composite action, respectively. As conform to the degree of composite action, the 10-connector slab system has the highest magnitude of load sustaining value, followed by the 4-connector and, lastly, the no-connector slab system. The specimens showed redistribution of stress in the connections, thus resulted in the strength recovery in the event the outer connections fail. Therefore, proper design and workmanship are highly required.

The control of the mid-span deflection during loading is dependent to the degree of composite action available on the slabs. However, the trend, based on the percentages computed, proves that there will be a point at where the optimum number of connectors becomes the governing factor of giving the highest degree of control over the mid-span deflection.

From the relative slip results obtained, especially for the 4- and 10-connector slab system, the related load-relative slip plots displayed the same gradient for the respective curves. This can be argued that the individual connection system, regardless of the number of connectors, provides the same degree of composite action, provided that the slabs are constructed in the same manner as of one another. However, the number of connectors plays an important role, too, in further controlling of the interlayer slip, as far as the capacity of the slab could sustain, in terms of loading.

The observation of the post-test clearly showed that the concrete topping is within the compression region. The topping did not fail due to bending or tensile stress, as the timber beams took the tensile stress and sustaining to failure. Hence, the neutral axis of the composite is located at the interlayer of the two constituents. Therefore, in order to fully utilize the advantages of both materials, the concrete and timber, the position of the neutral axis is best located within the interlayer's depth.

XVI.2. Reliability of Gamma Method at Ultimate Limit State and Serviceability Limit State

The effective bending stiffness method or Gamma method according to Annex B of Eurocode 5 was used to evaluate the analytical or theoretical bending stiffness, $(EI)_{theo}$, of the slab specimens. The purpose of accounting the analytical value was to compare the results with the experimental bending stiffness, $(EI)_{expt}$. The comparison was then used to check on the reliability of the Gamma method in the design of CTC slabs. However, in the analysis of the quantified bending stiffness data, experimentally and analytically, the comparisons showed a huge difference of value. The experimental-to-analytical ratio of the bending stiffness was computed at a minimum of 2. In other words, the Gamma method underestimated the effective bending stiffness of the floor system.

The slip modulus values at ULS and SLS conducted from a series of push-out tests were relatively low and approximately the same. Further review needs to be emphasized on the push-out tests to understand the mechanism that produce such an outcome.

However, in contrast with the reliability of the Gamma method in evaluating the effective bending stiffness, it was proven that the reduction in the effective spacing of the connectors produces a higher quantity of effective bending stiffness, ultimately increases the overall performance of the floor system.

XVI.3. Structural Performance of Triple-T Precast-Concrete-Timber Composite Floor System at Ultimate Limit State and Serviceability Limit State Under Short-Term Loading

The efficiency of the composite action needs to be taken into account as a much higher efficiency allows more reduction of depth and more increment of span of the slab. Furthermore, the safe imposed load capacity that could be sustained by the floor system can be increased drastically.

Based on the results obtained and computed for the experimental-analytical safe imposed load capacity, the Gamma method also underestimated the imposed load capacity, where the experimental was larger than the analytical capacity by at least 3. Similar to the explanation regarding on the reliability of the analytical bending stiffness, the slip modulus values are important in determining the effectiveness of the Gamma method in analyzing the performance of the floor system.

From the data analysis, reduction in the effective spacing resulted in a higher imposed load capacity. However, in order to investigate the optimum effective spacing that produces the best result, further research needs to be conducted.

XVI.4. Cost Effectiveness, Ease of Construction and Sustainability of Technology

In order for the concrete-timber composite flooring system to be fitted in the construction industry, certain criteria has to be taken into account, not only in terms of the structural reliability or performance, but also to the extent of the ease of manufacturing, cost effectiveness and also the technology's sustainability. Only then can the concrete-timber composite flooring system compete, or be at par with the other existing flooring systems available in the market.

Logically, the triangular notch used as the connection system in this study by right is one type of connection that requires the least time to be formed on the timber joists, as compared to the rectangular notch.

The average cost of construction per slab was relatively high. This economic scenario needs to be revised and further research is required to find an alternative solution to reduce the cost.

The use of epoxy resin is harmful to the environment. This creates an issue over the sustainability of the precast-CTC floor system grouted with epoxy resin.

XVII. Conclusion

From this study, the test result, analytically and experimentally exhibited the Gamma method underestimated the effective bending stiffness of the precast-CTC floor system. The experimental bending stiffness was at least 2 times larger than the theoretical bending stiffness computed by the applying the Gamma method. Correspondingly, the values of the effective bending stiffness affected the safe imposed load capacity as well. Furthermore, reduction of the effective spacing between the connectors produced a higher composite action for the floor system. The bending stiffness values of the studied specimens were inversely proportionate to the effective spacing of the connectors.

The usage of epoxy resin, Sikadur-330, resulted in the enormous hike of construction cost of the triple-T precast-CTC floor system. The cost per hour per labor was computed at RM26.64 for duration of 25 hours per slab construction.

Further research is needed to study the optimum number of connectors, or effective spacing to produce an economic CTC floor system. Alternatives to epoxy resin have to be studied because the usage of epoxy resin is costly and harmful to health and environment.

Acknowledgements

Universiti Tun Hussein Onn Malaysia partially supported the fundings for this project. Lam Kah Leong and supervisors for provided technical support and inspiration to make this study a success. Without their commitment, knowledge and financial support, it would be hard to make ends meet.

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