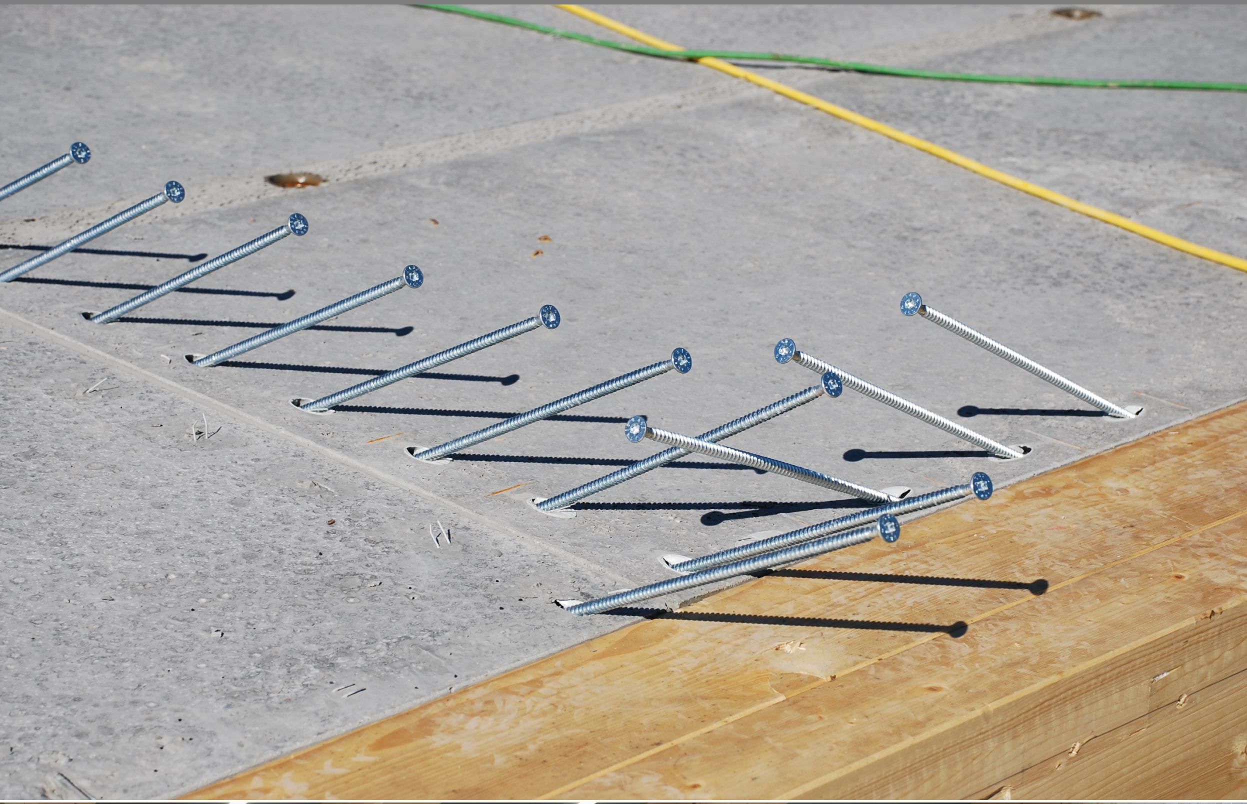
MyTiCon Timber Connectors

WHITE PAPER

TIMBER CONCRETE COMPOSITE -TCC- SYSTEMS



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Timber concrete composite sections *-TCC-* are typically composed of a reinforced concrete topping layer laid on top of a timber section, as shown in Fig. 1. Mechanical jointing between the concrete and timber layer is most typically provided through the use of mechanical fasteners, most commonly self tapping wood screws, that are inserted into the timber element prior to concrete being poured. This effectively creates a mechanical joint between the concrete and timber layers that allows for the two sections to work together, at some degree of efficiency, in order to take advantage of each of the materials strengths. TCC sections are now being used in many countries around the world for new buildings, as well as in renovation and retrofit applications of old structures and in bridges[1].

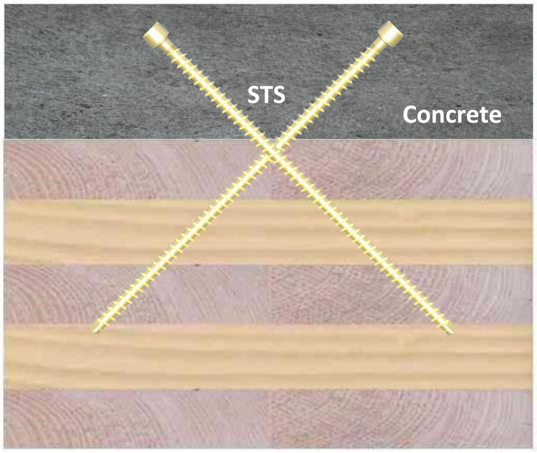

Fig.1– Typical Timber Concrete composite section

All joints are prone to deformations under loading and the mechanical joints created to link these two different elements together are not the exception. For that reason, understanding what parameters and factors most influence the level of composite action or how well these different layers work together, is vital in order to be able to fully exploit the main features that a TCC system may offer in lieu of traditional concrete or timber flooring systems. In simple terms, by identifying which parameters influence the ability of these different material to collaborate and work together, the more benefit we can stand to obtain by utilizing them in our modern day structures.

This White Paper's aim is to introduce TCC systems and present their characteristics, advantages and design considerations. It hopes to show how a TCC system may be an efficient structural solution, as well as to present more in-depth technical information that can guide the designer to more accurately understand the mechanical behaviour that governs timber concrete composite systems. Additionally, this White Paper will discuss the results of a research study that took place at the University of British Columbia. This research study evaluated the performance of TCC Floor Systems through testing of small and large scale TCC Floor system specimens, as shown in Fig. 2.

Fig.2– TCC Floor system under testing at the Univ. of British Columbia[5]

TCC systems are usually composed of a nominally reinforced concrete layer and a timber panel or element that are mechanically connected together to achieve composite action trough the use of a discrete or continuous shear connector. For the purposes of this White Paper, composite action can be efficiently achieved by driving fully threaded self-tapping structural screws into the timber element and then casting the concrete on top to effectively embed a portion of these screws into the concrete slab, as shown in Fig. 3.

Fig.3– SWG ASSY® fully threaded screw embedded in concrete

The main features and advantages of TCC systems are:

- TCC systems are lighter than comparable conventional reinforced concrete systems[1];
- TCC systems can have almost triple the load-carrying capacity and up to six times the flexural rigidity of traditional timber floor systems, if efficient composite action is achieved. This results in larger spans, larger open spaces and potentially less column supports within buildings[1];
- They are much more efficient than conventional reinforced concrete systems in terms of load carried per unit self-weight[1];
- TCC systems have a very large in-plane rigidity that they keep their shape (and consequently the shape of the entire building) during an earthquake[1]. This may allow the use of simplified seismic analysis procedures for rigid diaphragms;
- TCC systems can have viscous damping of up to 2%, under certain conditions. This relatively higher viscous damping value may help reduce the “springiness” that can be uncomfortable for users when they walk and jump on floors[1].
- Composite systems have good sound insulation characteristics[1];
- TCC systems may have a superior fire performance than all other timber systems. The upper concrete slab is an efficient barrier against fire propagation that increases fire resistance compared with all other timber-only systems[1];
- TCC systems may have with the added benefit of creating longer spanning floor systems that result in less interior support requirements[1]. (See Fig. 4)
- Pre-manufacturing is possible. By casting the reinforced concrete topping layer off-site and providing means to insert the mechanical fasteners on-site (see Fig. 5), project costs and schedules can be optimized while also eliminating the number of trades on site.

Fig.4– Interior view of office space with a TCC

Fig.5 - FT® Connector that allow for concrete topping layer to be pre-manufactured

The ultimate goal of a TCC system is to create efficient load bearing structural sections that work together by mostly stressing the *concrete layer in compression*, and the *timber layer in tension*. A composite section is created most typically by utilizing mechanical fasteners to connect the layers together to create a mechanical link between the two different layers. The level of efficiency at which composite action occurs in these types of sections is directly related to the capacity of the shear connector to limit slip between the individual layers.

Roughly speaking, fully rigid connections will completely limit relative slip between layers and allow for "full composite" action to occur (e.g. adhesives). Nevertheless, "softer" mechanical connectors are more commonly utilized (e.g. self-tapping wood screws), "semi-rigid" connections between layers are constructed that permit some degree of relative slip between concrete and timber layers to occur. Therefore, under these conditions, the assumption that cross-sections remain plane under loading will no longer be valid[1]. Fig. 6 illustrates the effect that having a "non-planar" section has on the cross-section's internal stress distribution. This internal stress distribution shows that both the concrete and timber layers may end up being stressed in compression and tension to a certain degree, with relative slip occurring between these layers. The amount of relative slip between layers is highly dependant of the shear connector's rigidity and the overall composite efficiency of the connection between concrete and timber layers. Figure 7 shows how this relative slip between layers occurs at the end of sections after reaching ultimate capacity of the panel. Maximum relative slip will occur at the ends, as seen in Fig. 7, while no relative slip will occur at mid-span if the system is fully symmetric[1].

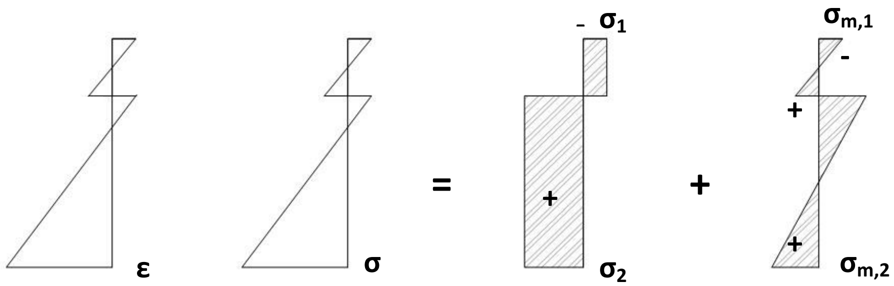

Fig.6 – Strain and stress distributions of "non-planar" composite sections

Fig.7 – Slip at ends of composite sections after

When subjected to bending action, in short term loading scenarios, adjacent layers within a semi-rigid composite system will slide relative to one another in a partially constrained manner. This is what we commonly refer to as relative slip. Furthermore, Figure 8, shows the cross section geometrical relationships between layers (left) as well as the distribution of bending stress (right). Maximum shear stress occur where the normal stresses are zero[3].

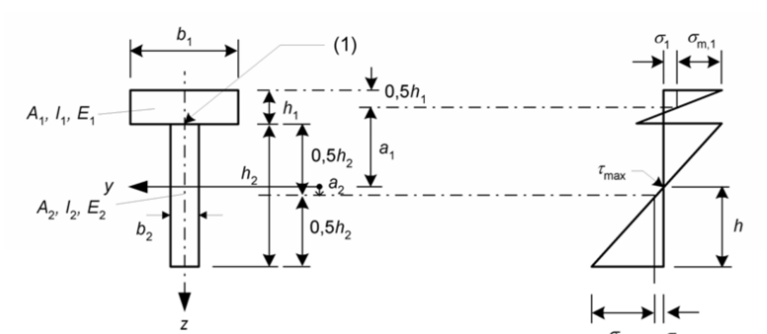

Fig.8 - Behaviour of a composite section with semi-rigid connectors[3]

If a semi-rigid connector is utilized as a shear connector, relative slip will occur between layers consequently violating the Euler-Bernoulli assumption that plane sections will remain plane. Therefore, an alternative procedure to determine the composite section's bending stiffness and stress distribution is required. Presently, a linear elastic solution based on the assumption that all materials (timber, concrete and connectors) will remain elastic allows for determination of stresses in composite elements at different load states. This analytical procedure was first proposed by Mölher in 1956 and then later recommended by Ceccotti in 2002[6].

This approach, commonly known as the "Gamma-Method", is included into an Informative Annex (Annex B) of the Eurocode 5 Part 2[3]. This design procedure assumes the effective bending stiffness $(EI)_{eff}$ of a TCC section to be calculated by the following design equation:

(1) The effective bending stiffness should be taken as:

$$(EI)_{\mathrm{ef}} = \sum_{i=1}^{3}(E_{\mathrm{i}} I_{\mathrm{i}} + \gamma_{\mathrm{i}} E_{\mathrm{i}} A_{\mathrm{i}} a_{\mathrm{i}}^2) \qquad \text{(B.1)}$$

The gamma factor, Υ, present in this design equation is the "connection efficiency factor"; while a is the distance between centroids of both layers to the neutral axis; A is each of the layer's cross sectional area; E is each of the material's Young's Modulus; and finally I is each of the layer's area moment of inertia ($bh^3/12$ for rectangular sections). Based on the design recommendations found in Appendix B of Eurocode 5[3], one can write:

using mean values of E and where:

$$A_{\mathrm{i}} = b_{\mathrm{i}} h_{\mathrm{i}} \qquad \text{(B.2)}$$

$$I_{\mathrm{i}} = \frac{b_{\mathrm{i}} h_{\mathrm{i}}^3}{12} \qquad \text{(B.3)}$$

$$\gamma_2 = 1 \qquad \text{(B.4)}$$

$$\gamma_{\mathrm{i}} = \left[1 + \pi^2 E_{\mathrm{i}} A_{\mathrm{i}} s_{\mathrm{i}} / (K_{\mathrm{i}} l^2)\right]^{-1} \quad \text{for } i = 1 \text{ and } i = 3 \qquad \text{(B.5)}$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2\sum_{i=1}^{3} \gamma_{\mathrm{i}} E_{\mathrm{i}} A_{\mathrm{i}}} \qquad \text{(B.6)}$$

where the symbols are defined in Figure B.1;

$K_{\mathrm{i}} = K_{\mathrm{ser,i}}$ for the serviceability limit state calculations;

$K_{\mathrm{i}} = K_{\mathrm{u,i}}$ for the ultimate limit state calculations.

Because connections between layers may also be continuous, the slip modulus per unit length $k=K/s$ is used rather than K; the variable s in this design equation is the spacing between connections. Additionally, as fasteners in connections are commonly spaced according to the shear force demand in the shear plane, an effective spacing is then assumed. This effective spacing, s_{eff} is then used for the calculation of Υ.[1] The effective spacing, s_{eff}, is calculated by taking into consideration the spacing of fasteners at the ends of the composite systems, s_e; and the spacing of fasteners in the middle of the system, s_m. With, s_m limited to $s_e < s_m < 4s_e$, and $s_{eff} = 0.75s_e + 0.25s_m$.[1] If these fastener spacing conditions are not met, then the "gamma" method should not be utilized.

It is important to note that the 'gamma' method to compute an effective bending stiffness, $EI_{eff,}$ was developed as a closed-form solution that produces exact solutions for simply supported beams with varying sinusoidal bending moment along its length. For other boundary and loading conditions (e.g. multi-span beams) this method produces approximate results that may require several iterations.[6]

Short Term Mechanical Behaviour

Most studies and laboratory testing of TCC has been performed under quasi-static short-term loading conditions. Yet, care must be taken in order to properly account for the connector's time-dependant behaviour, and therefore, the ultimate behaviour of the composite section. Typically, it is understood that the behaviour of connections within a TCC system will vary from a less rigid but ductile system, to a more rigid but brittle system. Ductile behaviour of a TCC system *is not necessarily achieved just because connections exhibit a ductile behaviour*. If connection stiffness is greater thanl expected (due to conservative underestimation) the timber layer may reach its ultimate strength capacity while connections are still responding elastically. This will cause the system to perform much less ductile than anticipated similar to an over-reinforced concrete section.[1]

Long Term Mechanical Behaviour

In the discussion presented in the previous pages, the mechanical properties for a TCC system were considered to be constant. The reality though, is that in the long-term, *the behaviour of all components within a TCC system are influenced differently as time goes by*. In simple terms, over longer periods of time, the concrete layer will shrink and creep; the timber layer meanwhile will respond to ambient moisture content alterations by swelling, shrinking and creeping. Additionally, the connections creating the composite action within layers, will also creep, and typically will creep more than timber.[1]

Material behaviour under loading and varying climatic conditions will typically cause migration of internal forces between the elements (concrete layer, timber layer and connectors) and consequently, cause a variation of the initial internal stresses of the composite section. If, for example, the concrete layer exhibits greater relative creep than the timber layer, stresses will increase in the timber layer and reduce in the slab.[1] This particular phenomenon was also the focus of investigation for the research campaign recently completed at UBC. Preliminary results for these long term mechanical effects are expected towards the summer of 2016.

Vibration

Floor vibration caused by dynamic actions such as people walking on the floor, machinery or similar repetitive actions may cause discomfort to occupants. As a result, timber floors are often designed to ensure that the lowest natural frequency falls within a certain range of natural frequencies. The intent of doing so, is to keep the floor's natural vibrational modes out of the frequency range of the most likely dynamic excitations caused by people walking, occupancy and so on, in order to minimize the potential vibration effects. *Section 7.3.3* of Eurocode 5 provides some basic guidelines for floor systems with a fundamental natural frequency greater than 8Hz, while it recommends special investigation into floor systems with a fundamental natural frequency lower than 8Hz.[3] Past research on this specific topic has pointed out however, that correlation between natural frequency and perceived vibration performance is not close enough for frequency to serve as the only criterion for proper performance. Special cases of unsatisfactory performance of light TCC floors with frequency above 8Hz, as well as satisfactory results for heavier floors with frequency below 8Hz have been noted.[5] Further testing must be performed to better understand the parameters that influence vibration of a TCC panel. For example, mass, damping and disturbing force need to be taken into account to form more reliable predictions.[5] Finite Element models are a great tool to capture a more realistic snapshot of the performance of TCC Systems if proper design assumptions are taken into account.

Dynamic Considerations for Design of TCC Systems

The design of long-span and light weight floor construction is often governed by serviceability rather than strength requirements and dynamic performance is one of those requirements. Therefore, there is a growing need for measurement of dynamic characteristics of floor systems such as natural frequencies, damping ratios and mode shapes that will allow for further investigation of their behaviour.[8] An initial serviceability design of a flooring system requires an assessment of the fundamental frequency (or first natural frequency) in order to check the vibration behaviour of the floor and determine potential occupant discomfort.

For design purposes, the fundamental frequency ranges listed below are normally avoided:

- Frequencies below 3Hz to prevent resonance caused by walking[9]
- Frequencies between 5-8Hz to prevent human discomfort[9]

Typically, it is suggested that for residential/office floors, a natural frequency greater than 10Hz shall be targeted.[9]

A number of analytical prediction models can be found in the design standards and literature to determine the fundamental frequency, particularly of flooring systems. The following documents offer design considerations and design guidelines that can be applied to TCC floor systems:

1. SCI Publication P354. *Design of Floor for Vibration: A New Approach* (revised edition-2009).
   The Steel Construction Institute. United Kingdom.
2. CCIP-016. *A Design Guide for Footfall Induced Vibration of Structures*. (2006).
   Cement and Concrete Industry Publication. United Kingdom.

Introduction / Materials

A comprehensive research campaign was recently completed at the University of British Columbia in which small and large scale specimens of TCC Systems were tested. This extensive test campaign of over 300 small scale specimens tested configurations utilizing glued steel mesh connectors, epoxy based connections, as well as over 200 specimens utilizing STS. Small scale specimens were fabricated and tested in order to evaluate the performance that different arrangement of fully threaded wood screws would produce. The results were utilized to optimize the design of subsequent large scale test specimens that would undergo vibration and bending tests. 27 Large scale specimens were fabricated in total, of which 15 included self-tapping wood screws as connectors.[6]

The test program proved successful in showing that efficient TCC systems can be obtained utilizing a variety of arrangements of SWG ASSY® fully threaded screws and any of the commonly available Engineered Wood Products - EWPs- in the market. This leads to the conclusion that the specific selection for the combination of EWP product and screw arrangement can now be made on the basis of cost, architectural intent, installation conditions and structural condisderations.[6] Table 1 summarizes the EWP products that were utilized for this project.

Table 1. EWP Material Properties[6]

	Material Properties [Mpa]					
Material	**MOE**	F_t	F_c	F_b	F_v	F_{cp}
LSL (1.55E)[]	10,685	20.4	22.6	33.6*	1.95*	6.10*
LVL (2.0E)[]	13,790	18.6	35.2	37.6*	1.79*	9.41*
LVL-V (2.0E)[]	13,790	18.6	35.2	37.0	3.65	6.90
CrossLam CLT[2]	9,500	5.5	11.5	11.8	0.50†	--

- LVL-V = Vertically laminated LVL
- MOE = Modulus of Elasticity; F_t, F_c, F_b, F_v, F_{cp} = specified strengths in tensions, compression, bending, shear and compression perpendicular to grain[6]

* Strength properties for plank orientation[6]

† Rolling Shear (Longitudinal Shear = 1.5MPa)[6]

For the specimens, normal strength concrete was used in order to more closely adhere to local construction and design practice. The specified 28-day compressive strength was 30*Mpa* with a coarse aggregate maximum size of 19mm (3/4”). Additionally, as insulation is sometimes required for TCC floor systems, some specimens were fabricated with an insulation layer placed between the concrete and timber layers. For this project, Foamular® C-200 extruded polystyrene rigid insulation was utilized. This product is locally available and has a prescribed compressive strength of 140 kPa.[6] Fig. 9 shows the various STS arrangements and connectors that were tested for the small scale campaign.

Fig.9 – Small Scale Specimen[5]

Self-Tapping Wood Screws as Shear Connectors

Self-tapping wood screws (STS) do not require pre-drilling and therefore are faster to install. That makes STS a feasible solution for timber structures from a cost-efficiency standpoint if compared to more conventional connectors such as bolts or lag screws. For this project, fully threaded self-tapping screws made out of high strength carbon steel were chosen. Fully threaded SWG ASSY® screws (VG) come equipped with a self-drilling tip in order to reduce the drive in torque during installation, and consequently reduce the risk of splitting. Fig. 10 illustrates the STS selected for this research project, while Table 2 presents the fastener's specifications.

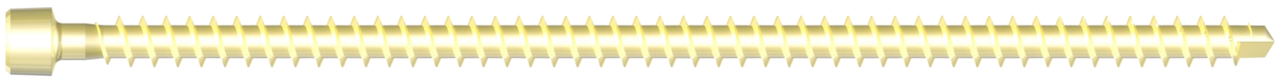

Fig. 10. SWG ASSY® VG CYL STS

Table 2. SWG ASSY® VG CYL Fastener Specifications[4]

SWG ASSY® VG CYL	Fastener Strengths [Mpa]		
Outside Thread Diameter [mm]	**Root Diameter [mm]**	**Bending Yield Strength [MPa]**	**Factored Tensile Resistance [kN]**
10	6.2	942	19.2

Small Scale Specimen Configuration

The use of empirical formulas to determine connector stiffness and using simplified design procedures which do not consider the non-linear (post-yielding) behaviour of shear connectors within a TCC system, can produce significant underestimations of connector stiffness and strength. This can lead to larger resistances but eventual brittle failure of the composite panel or beam rather than achieving connector yielding (ductile failure mode).[1] In order to evaluate the actual connection properties, these small scale specimen push-out tests were performed. Table 3 summarizes the different configuration of small scale specimens that were tested that contained self tapping wood screws as shear connector.[5] A load controlled loading protocol was implemented according to *EN-26891 (1995).* These specimens were loaded up to 40% of the expected maximum load, released, and re-loaded up to failure (F_{ult}). The permanent deformation recorded after releasing the load, referred to as Δ_{res}, was captured in order to correctly determine the stiffness results for these connections[5].

Table 3. Configuration of small scale specimens using STS[7]

SMALL SCALE SPECIMEN TEST RESULTS (6-12 replicates per series)					
Connector Description	**EWP**	**STS Diam. [mm]**	**STS Length [mm]**	**STS embedment [mm]**	
				Concrete Layer	**Timber Layer**
ASSY® fully threaded VG CYL 8x240 @ 30° (Fig. 11)	LSL	8	240	90	150
ASSY® fully threaded VG CYL 10x240 @ 30°	LSL	10			
ASSY® partially threaded SK 10x260 @ 30°	LSL	10	260		170
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LSL	10	240		150
ASSY® fully threaded VG CYL 8x240 @ 30°	LVL	8	240	90	150
ASSY® fully threaded VG CYL 10x240 @ 30°	LVL	10			
ASSY® partially threaded SK 10x260 @ 30°	LVL	10	260		170
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LVL	10	240		150
ASSY® fully threaded VG CYL 8x240 @ 30°	LVL-V	8	240	90	150
ASSY® fully threaded VG CYL 10x240 @ 30°	LVL-V	10			
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LVL-V	10			
ASSY® fully threaded VG CYL 8x240 @ 30°	CLT	8	240	90	150
ASSY® fully threaded VG CYL 10x240 @ 30°	CLT	10			
ASSY® fully threaded VG CYL 10x300 @ 30°	CLT		300		210
ASSY® partially threaded SK 10x260 @ 30°	CLT		260		170
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30° (Fig. 12)	CLT		240		150

Fig.11 - ASSY® fully threaded VG CYL screw head (left), and ASSY® SK partially threaded screw head (right)

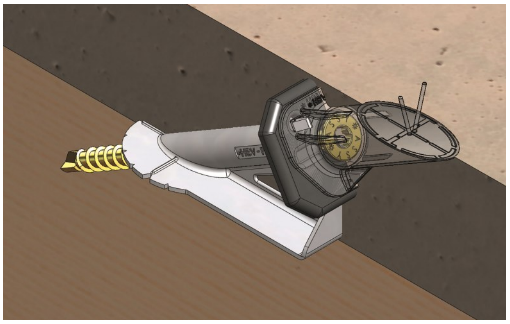

Fig.12 - FT® Connector that allows for concrete topping layer to be pre-manufactured

Large Scale Specimen Configuration

A summary of the panel configurations, for specimens utilizing STS as shear connectors, is presented in Table 4.

Table 4. Large Scale Specimen Configuration[6]

Configuration Parameters (2 replicates per series)									
Connector Description	**EWP**	**b [mm]**	**L [mm]**	**t_c [mm]**	**t_t [mm]**	**t_i [mm]**	**Rows**	**s_1 [mm]**	**s_2 [mm]**
S1 - SWG ASSY® CYL 10x240 @ 30°	**EWP**	610	6,096	70	89	- -	3	150	300
S2 - SWG ASSY® CYL 10x240 @ 30°	LSL	610	6,096	70	89	- -	3	150	300
S3 - SWG ASSY® CYL 10x240 @ 30°	LVL	600	6,000	70	99	- -	3	150	300
S4 - SWG ASSY® CYL 10x240 pairs @ 45° through 25mm of insulation	CLT-3ply	610	6,096	70	89	25	3	300	- -

- b and L= panel width and length, respectively.
- t_c , t_t , t_i = thickness of concrete, timber and insulation layers, respectively.
- Rows = # of connector rows of connectors.
- s_1 and s_2 = connector spacing in high and low shear zones.
- Embedment of STS into concrete was 90mm for all series; Embedment into timber layer was the same for all series (leff = 140mm) except for Series with VG CYL 10x240 pairs @ 45° through insulation (leff = 90mm)

Large Scale Specimen Configuration

The TCC composite system panels were tested for strength and stiffness under four point bending. The dynamic performance of each panel was predicted using the previously discussed *gamma method* to predict the effective bending stiffness, and through the use of dynamic analysis in a Finite Element Modeling software. The panels were initially subjected to a dynamic excitation from a light impact (heel strike), and the accelerations were captured using a digital accelerometer.

The panels were then loaded to *service level* (defined as the actuator load that would cause the same bending moment as would be obtained from a 4.8kPa Uniformly Distributed Load (UDL), held for thirty seconds, unloaded and reloaded to service level a minimum of two times. This was done in order to allow the fundamental frequency to stabilize at service loading in order to correctly capture it during this test. Finally, the panels were loading to failure at a constant rate of 6mm/min.

Results

A summary of the most relevant test results for the small scale specimens is presented in Table 5. As over 300 small scale specimens were tested, only the results for the configurations utilizing STS that are similar to or that were replicated during the large scale testing campaign are presented.

Table 5. Summary of Small Scale Test Results[7]

SMALL SCALE SPECIMEN TEST RESULTS (6-12 replicates per series)							
Connector Description	EWP	F_{ult} [kN]	Δ_{res} [mm]	Average Recorded Stiffness		
				$K_{0.4}$ [kN/mm]	$K_{0.8}$ [kN/mm]	K_{ult} [kN/mm]
ASSY® fully threaded VG CYL 8x240 @ 30°	LSL	26.8	0.09	53.0	31.5	15.6
ASSY® fully threaded VG CYL 10x240 @ 30°	LSL	30.6	0.06	69.3	59.3	44.5
ASSY® partially threaded SK 10x260 @ 30°	LSL	33.4	0.13	31.9	19.1	12.9
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LSL	46.9	0.28	32.1	21.5	13.3
ASSY® fully threaded VG CYL 8x240 @ 30°	LVL	26.8	0.13	57.4	27.3	13.9
ASSY® fully threaded VG CYL 10x240 @ 30°	LVL	34.4	0.06	69.0	35.6	19.6
ASSY® partially threaded SK 10x260 @ 30°	LVL	24.1	0.26	40.0	22.9	13.3
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LVL	36.0	0.27	37.4	22.8	14.2
ASSY® fully threaded VG CYL 8x240 @ 30°	LVL-V	26.8	0.13	57.6	28.1	15.1
ASSY® fully threaded VG CYL 10x240 @ 30°	LVL-V	32.5	0.07	75.4	41.6	27.6
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	LVL-V	33.4	0.45	37.5	24.4	13.6
ASSY® fully threaded VG CYL 8x240 @ 30°	CLT	25.6	0.14	50.0	21.0	10.2
ASSY® fully threaded VG CYL 10x240 @ 30°	CLT	30.5	0.14	83.0	30.5	12.7
ASSY® fully threaded VG CYL 10x300 @ 30°	CLT	33.8	0.13	62.7	30.5	20.4
ASSY® partially threaded SK 10x260 @ 30°	CLT	22.6	0.24	27.6	14.4	5.5
FT® connector embedded in precast concrete + ASSY® fully threaded VG CYL 10x240 @ 30°	CLT	40.4	0.59	32.9	18.9	11.2

- Δ_{res} refers to the residual deformation measured on the small scale specimens as explained on page 9 of this document.

Figures 13-16 aim to present the previously listed results in more visual manner. The plots show the re-created load displacement curves using the values presented in Table 5 to summarize the behaviour of each STS configuration for each of the tested EWP materials.

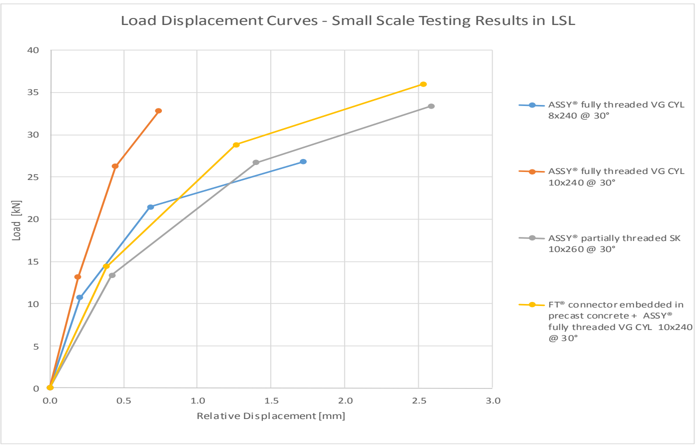

Fig.13 - Load-Displacement Curves for results in LSL

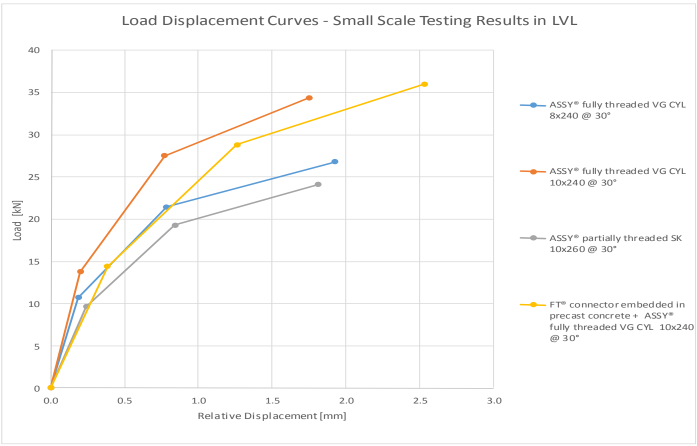

Fig.14 - Load-Displacement Curves for results in LVL

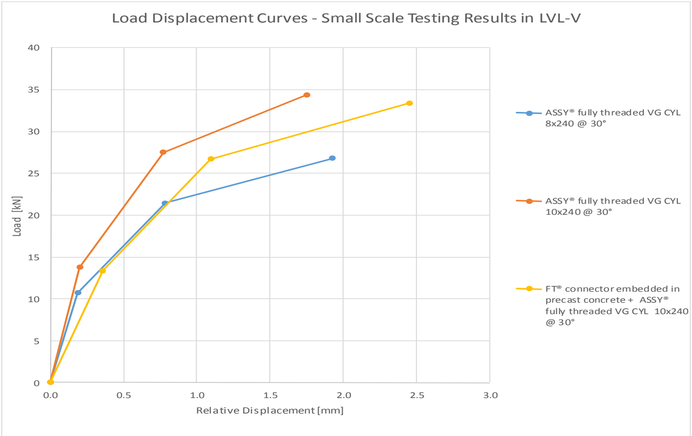

Fig.15 - Load-Displacement Curves for results in LVL-V

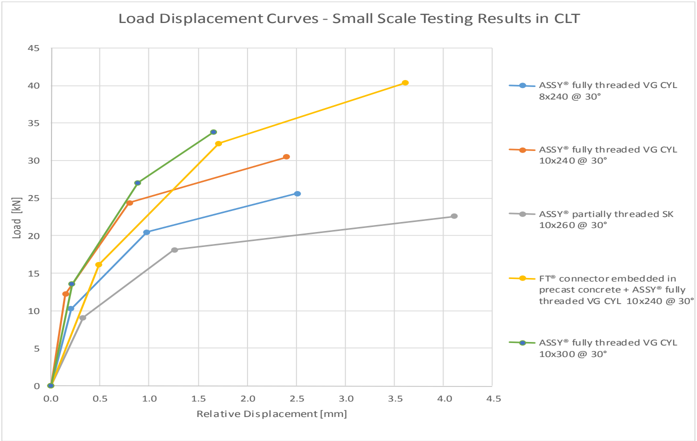

Fig.16 - Load-Displacement Curves for results in CLT

It is very important to note that these plots have been adjusted to not show the initial shear slip that occurred during the pre-loading cycles of the testing campaign. This initial inelastic permanent deformation (Δ_{res}) occurred after the small scale specimens were loaded, to approximately 40% of the expected maximum load.

A summary of the most relevant test results for the large scale specimens is presented in Table 6 and Table 7. Table 6 presents test results and compares them to the predictions that were performed by Gerber and Tannert (2015) following the Gamma method presented throughout this document.

Table 6. Summary of Large Scale Test Results[6]

LARGE SCALE SPECIMEN TEST RESULTS (avg. results out of 2 replicates)							
Series	**EWP**	**$\Delta_{inelastic}$ [mm]**	**Panel Flexural Rigidity - EI_{eff} [x 10^{12} N-mm^2]**		**Fundamental Frequency [Hz]**		**Governing Failure Mode**
			Service Level* (testing)[6]	**ϒ-method (prediction)[6]**	**Service Level* (testing)[6]**	**ϒ-method (prediction)[6]**	
S1	LSL	192.1	3.32	3.24	7.13	7.10	Concrete crushing + timber fracture
S2	LVL	195.1	3.82	3.73	7.76	7.75	Screw withdrawal + timber fracture
S3	CLT-3ply	132.6	3.03	2.84	6.91	7.01	Timber fracture
S4	LSL	182.0	4.50	4.59	8.17	8.44	Screw Withdrawal

* Service Level Loading refers to the actuator load that would cause the same bending moment as would be obtained from a 4.8kPa UDL

\- $\Delta_{inelastic}$ refers to the permanent deformation measured on the large scale panels after an equivalent service Level Loading was attained and subsequently released following the loading protocol, as explained on page 11 of this document.

Table 7 presents the average ultimate force and moment capacities exhibited by the different large scale test specimens. Additionally, an over-strength ratio is presented. This ratio aims to show the inherent margin of safety between the observed ultimate capacity of the tested timber-concrete composite panels and an UDL of 4.8*kPa* that is typically prescribed as live load for office spaces.

Table 7. Summary of Large Scale Test Results[6]

LARGE SCALE SPECIMEN TEST RESULTS (avg. results out of 2 replicates)									
Series	**EWP**	**Panel Width [mm]**	**Panel Length [mm]**	**Panel Area [m^2]**	**M_{ult} [kN-m/m]**	**F_{ult} [kN]**	**Equivalent UDL [kPa]**	**Over-Strength Ratio to Service Load***	**Approx. Shear Connection Cost [CAD$ / kN]**
S1	LSL	610	6,096	3.72	192.1	121.3	32.6	**6.8**	$2.70
S2	LVL	610	6,096	3.72	195.1	125.3	33.7	**7.0**	$2.60
S3	CLT-3ply	600	6,000	3.60	132.6	83.8	23.3	**4.9**	$3.60
S4	LSL	610	6,096	3.72	182.0	115.0	30.9	**6.4**	$3.40

– Service Level Loading refers to a commonly prescrived 4.8kPa UDL for office spaces

Governing Failure Modes

A summary of the governing failure modes of the large scale specimens is presented in Figures 17 to 20.

Fig. 17. Series 1 panel failure due to concrete crushing (left and centre) and timber tensile failure (right) - Brittle[6]

Fig. 18. Series 2 panel failure due to screw withdrawal (left) and timber tensile failure (right) - Brittle[5]

Fig. 19. Series 3 panel failure due timber tensile failure - Brittle[6]

Fig. 20. Series 4 panel failure due screw withdrawal through 25mm of rigid insulation - Ductile[6]

As Timber Concrete Composite systems are not a conventional structural system, here we present a simple guidline of construction recommendations that have been gathered from past research projects, constructed projects and research papers.[1]

1. Do not use wet timber. If this situation is unavoidable, use timber without pith, or make sure that cracks will not affect the location of fasteners[1];
2. Nominal reinforcement for the concrete topping layer is recommended in order to prevent cracking during curing or from environmental conditions leading to swelling or shrinking of the concrete layer.
3. Protect the timber from moisture when casting the concrete. Since composite action is being proportioned by mechanical fasteners, there is no impediment in utilizing plastic layers between the concrete and timber during casting[1];
4. Avoid using timber species with adverse chemical reaction to cement (i.e, Larch is a species with high sugar content extracts that will adversely react to cement)[1];

The following conclusions are meant to serve as general guidelines. Similarly as to the rest of this document, these findings aim to assist designers familiarize themselves with the design procedure most commonly utilized for TCC systems. IT is not intended to serve as a full, comprehensive design document for TCC systems.

1) Self tapping screws provide a cost-efficient solution to create Timber-Concrete composite sections

2) The gamma-method provides a closed-form design approach that is highly accurate when compared to test results. Careful attention should always be placed not to underestimate the slip modulus of the connector as this might produce unrealistic approximations($K_{0.4}$ or K_{ult}). The adequate choice depends on the type of structural check being performed.

3) TCC systems excel at allowing slender systems span longer between supports, if compared against traditional timber structural systems. Vibration therefore, should always be taken into account as slender systems are often more susceptible to dynamic excitations. The following documents offer vibration design considerations and guidelines that can be applied to TCC floor systems:

⇒ SCI Publication P354. *Design of Floor for Vibration: A New Approach* (revised edition-2009).

The Steel Construction Institute. United Kingdom.

⇒ CCIP-016. *A Design Guide for Footfall Induced Vibration of Structures*. (2006).

Cement and Concrete Industry Publication. United Kingdom.

4) It was proved through the completion of the performed testing campaign, that even though a layer of insulation might not be required, it can help produce a desirable ductile failure mode of the screws.

Based upon Mechanically Jointed Beam Theory, the Eurocode 5 standard contains an informative Annex which provides an approximate method for determining an effective bending stiffness of wood and concrete composite section if the slip modulus is know for the shear connectors. Deflections therefore, can be calculated by using that effective bending stiffness *(EI)*$_{eff.}$ This design procedure is based on the theory that a cross-section of a structural member is composed of several parts connected by mechanical fasteners and assuming that the relationship between force and slip is linear in nature. (Eurocode 5 Part 1-1: Annex B)[8]

For this Design Example all stiffness calculations used Service Level parameters (SLS), while strength checks were performed utilizing Ultimate Service Level parameters (ULS).

Timber Concrete Composite Systems
Strength and Stiffness Checks following procedure in CLT Handbook

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1.0 Geometry of Section:

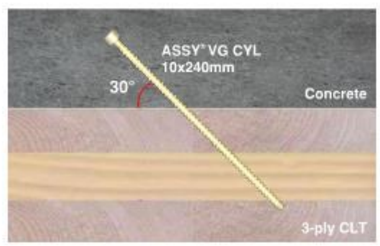
$h_{conc} := 70$ mm, concrete topping thickness

$h_{clt} := 99$ mm, CLT panel thickness

$h_{tot} := h_{conc} + h_{clt} = 169$ mm

$b := 600$ mm, width of strip considered

$L := 6000$ mm, length of span (assuming simple span)



2.0 Material Properties

$$Weight := \left(\left(\frac{h_{conc}}{1000} \cdot 2400\right) + (51)\right) = 219$$ kg/m^2

Concrete Properties:
70mm of concrete topping

$f'_c := 45.45$ MPa, 28-day compression strength of concrete

$f_r := 0.6 \cdot \sqrt{f'_c} = 4.045$ MPa, modulus of rupture for determination of cracking moment according to CSA A23.3

$E_c := 4500 \cdot \sqrt{f'_c} = 3.034 \cdot 10^4$ MPa, Modulus of Elasticity

$A_c := h_{conc} \cdot b = 4.2 \cdot 10^4$ mm^2

CLT Properties: 3-ply CLT CrossLam Panel

$$EI_{clt} := \left(735 \cdot \frac{b}{1000}\right) \cdot 10^9$$ N-mm^2, longitudinal bending stiffness

CLT Properties from Structurlam CrossLam V2M1-SLT3 specifications:

$E_{0clt} := 9500$ MPa

$F_b := 11.8$ MPa

$F_t := 5.5$ MPa

$F_v := 0.5$ MPa

3.0 Calculation of Gamma Factor of Composite Section

3.1 Concrete section:

$E_1 := E_c = 3.034 \cdot 10^4$ MPa $h_1 := h_{conc} = 70$ mm

$I_1 := \dfrac{b \cdot h_1^{\ 3}}{12} = 1.715 \cdot 10^7$ mm^4 $A_1 := A_c = 4.2 \cdot 10^4$ mm^2

3.2 CLT section:

$E_2 := E_{0clt} = 9.5 \cdot 10^3$ MPa $h_2 := h_{clt} = 99$ mm

$A_2 := b \cdot h_2 = 5.94 \cdot 10^4$ mm^2 $I_2 := \dfrac{EI_{clt}}{E_2} = 4.642 \cdot 10^7$ mm^4

3.3 ASSY VG Cyl 10x240mm screw as shear connector:

$d := 10$ mm, outside thread diameter
$l_{tot} := 240$ mm, total screw length
$l_{pen} := 150 - d = 140$ mm, penetration depth of screw into CLT panel
$\alpha := 30$ deg, angle between screw axis and shear plane
$n_r := 3$ number of rows of connectors across witdth of 600mm
$s_e := 150$ mm, spacing between connectors in rows at the ends
$s_m := 300$ mm, spacing between connectors in rows in the middle
$s_{eff} := 0.75 \cdot s_e + 0.25 \cdot s_m = 187.5$ mm, effective spacing between connectors
$k_{def\_c} := 2.5$ deformation factor for long term loading for Concrete
$k_{def\_t} := 0.6$ deformation factor for long term loading for timber
$k_{def\_sts} := 0.6$ deformation factor for long term loading for STS
$\psi_2 := 0.3$ stiffness reduction for ULS for long term loading (EN-1995, Appendix B)

3.3.1 Connector specific design parameters from test results:

$K_{SLS} := 83 \cdot 1000 = 8.3 \cdot 10^4$ N/mm, screw stiffness at SLS for short term loading (Obtained from 3-ply CLT, 150mm embedment, 70mm concrete topping)

$K_{ULS} := 30.5 \cdot 1000 = 3.05 \cdot 10^4$ N/mm, screw stiffness at ULS for short term loading (Obtained from 3-ply CLT, 150mm embedment, 70mm concrete topping)

$F_{rK} := 30.5 \cdot 1000 = 3.05 \cdot 10^4$ N, mean value strength from tests ((Obtained from 3-ply CLT, 150mm embedment, 70mm concrete topping)

3.3.2 Connector Stiffness adjustment for long term loading:

$K_{SLS\_LT} := \dfrac{K_{SLS}}{1 + k_{def\_sts}} = 5.188 \cdot 10^4$ N/mm, screw stiffness at SLS for long term loading

$K_{ULS\_LT} := \dfrac{K_{ULS}}{1 + (\psi_2 \cdot k_{def\_sts})} = 2.585 \cdot 10^4$ N/mm, screw stiffness at SLS for long term loading

3.3.3 Modulus adjustment for long term loading:

$$E_{1\_SLS\_LT} := \frac{E_1}{1+k_{def\_c}} = 8.668 \cdot 10^3 \text{ MPa} \qquad E_{1\_ULS\_LT} := \frac{E_1}{1+(\psi_2 \cdot k_{def\_c})} = 1.734 \cdot 10^4 \text{ MPa}$$

$$E_{2\_SLS\_LT} := \frac{E_2}{1+k_{def\_t}} = 5.938 \cdot 10^3 \text{ MPa} \qquad E_{2\_ULS\_LT} := \frac{E_2}{1+(\psi_2 \cdot k_{def\_t})} = 8.051 \cdot 10^3 \text{ MPa}$$

γ "Gamma" Factor = Shear Connection Reduction Factor

$\gamma_2 := 1$ (B.4)

For Service Level Calculations (SLS):

$$\gamma_{1\_SLS} := \left(\left(1+\frac{\pi^2 \cdot E_{1\_SLS\_LT} \cdot A_1 \cdot s_{eff}}{n_r \cdot K_{SLS\_LT} \cdot L^2}\right)\right)^{-1} = 0.893 \qquad \text{(B.5)}$$

$$a_{2\_SLS} := \frac{\gamma_{1\_SLS} \cdot E_1 \cdot A_1 \cdot (h_1+h_2)}{2 \cdot (\gamma_{1\_SLS} \cdot E_1 \cdot A_1 + E_2 \cdot A_2)} = 56.479 \text{ mm} \qquad \text{(B.6)}$$

$$a_{1\_SLS} := \frac{h_1+h_2}{2} - a_{2\_SLS} = 28.021 \text{ mm}$$

For Ultimate Level Calculations (ULS):

$$\gamma_{1\_ULS} := \left(\left(1+\frac{\pi^2 \cdot E_{1\_ULS\_LT} \cdot A_1 \cdot s_{eff}}{n_r \cdot K_{SLS\_LT} \cdot L^2}\right)\right)^{-1} = 0.806 \qquad \text{(B.5)}$$

$$a_{2\_ULS} := \frac{\gamma_{1\_ULS} \cdot E_1 \cdot A_1 \cdot (h_1+h_2)}{2 \cdot (\gamma_{1\_ULS} \cdot E_1 \cdot A_1 + E_2 \cdot A_2)} = 54.538 \text{ mm} \qquad \text{(B.6)}$$

$$a_{1\_ULS} := \frac{h_1+h_2}{2} - a_{2\_ULS} = 29.962 \text{ mm}$$

3.5 Calculation of Effective Bending Stiffness for TCC System (EN 1995-1-1:2004 Appendix B):

For Service Level Calculations (SLS):

$$EI_{effTCC\_SLS} := \left(E_1 \cdot I_1 + \left(\gamma_{1\_SLS} \cdot E_1 \cdot A_1 \cdot a_{1\_SLS}^{\ 2}\right)\right) + \left(E_2 \cdot I_2 + \left(E_2 \cdot A_2 \cdot a_{2\_SLS}^{\ 2}\right)\right)$$

$$EI_{effTCC\_SLS} = 3.654 \cdot 10^{12} \text{ N-mm^2} \qquad \text{(B.1)}$$

For Ultimate Level Calculations (ULS):

$$EI_{effTCC\_ULS} := \left(E_1 \cdot I_1 + \left(\gamma_{1\_ULS} \cdot E_1 \cdot A_1 \cdot a_{1\_ULS}{}^2\right)\right) + \left(E_2 \cdot I_2 + \left(E_2 \cdot A_2 \cdot a_{2\_ULS}{}^2\right)\right)$$

$EI_{effTCC\_ULS} = 3.562 \cdot 10^{12}$ N-mm^2 (B.1)

4.0 Loading Conditions and Demands on TCC System:

$l := \frac{L}{1000} = 6$ m, simple span length

$B := \frac{b}{1000} = 0.6$ m, width of panel considered

3.1 Loading Conditions, assumed Office/Classroom occupancy:

$DL := 2.2$ kPa $w_d := DL \cdot (B) = 1.32$ kN/m = N/mm

$LL := 2.4$ kPa $w_l := LL \cdot (B) = 1.44$ kN/m = N/mm

$w_f := 1.25 \cdot w_d + 1.5 \cdot w_l = 3.81$ kN/m = N/mm

3.2 Loading Demands due to imposed loading: (EN 1995-1-1:2004 Appendix B)

$V_f := \frac{w_f \cdot l}{2} \cdot 1000 = 1.143 \cdot 10^4$ N, max shear force demand

$M_f := \frac{w_f \cdot l^2}{8} \cdot \left(1000^2\right) = 1.715 \cdot 10^7$ N-mm, max bending moment demand

$\sigma_{Nc} := \frac{\gamma_{1\_ULS} \cdot E_1 \cdot a_{1\_ULS} \cdot M_f}{EI_{effTCC\_ULS}} = 3.527$ Mpa (N/mm^2), Normal Stress in concrete (comp.)

$\sigma_{bc} := \frac{0.5 \cdot E_1 \cdot h_1 \cdot M_f}{EI_{effTCC\_ULS}} = 5.111$ Mpa (N/mm^2), Flexural Stress in concrete

$\sigma_{Nw} := \frac{E_2 \cdot a_{2\_ULS} \cdot M_f}{EI_{effTCC\_ULS}} = 2.494$ Mpa (N/mm^2), Normal Stress in wood (tension)

$\sigma_{bw} := \frac{0.5 \cdot E_2 \cdot h_2 \cdot M_f}{EI_{effTCC\_ULS}} = 2.264$ Mpa (N/mm^2), Flexural Stress in wood

$\tau_{2max} := \frac{0.5 \cdot E_2 \cdot b \cdot h_2{}^2}{b \cdot EI_{effTCC\_ULS}} \cdot V_f = 0.149$ Mpa (N/mm^2), Max Shear Stress in wood

$F_{max} := \frac{\gamma_{1\_ULS} \cdot E_2 \cdot A_2 \cdot a_{2\_ULS} \cdot s_e}{EI_{effTCC\_ULS} \cdot (1000)} \cdot V_f = 11.942$ kN, Max Load on a fastener (connector) where Shear Force is considered

3.3 Cross Sectional Stress Distribution:

For plot; Tensile stress is negative:

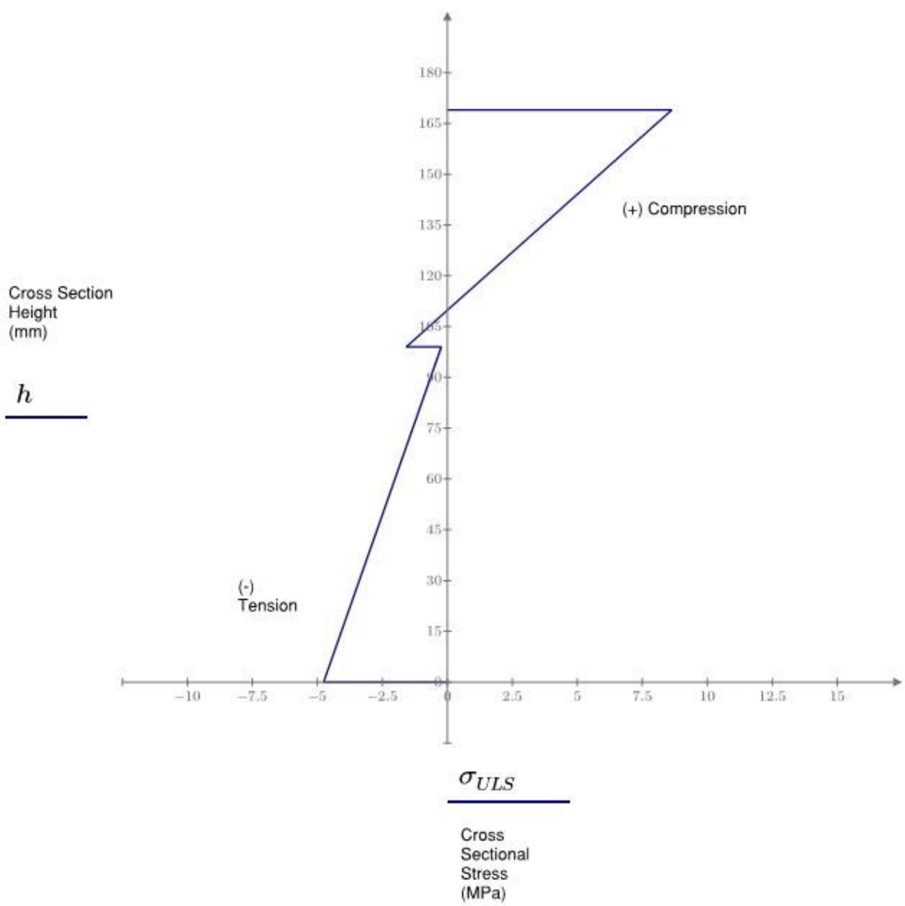
$$h := \begin{bmatrix} 0 \\ 0 \\ h_2 \\ h_2 \\ h_1 + h_2 \\ h_1 + h_2 \end{bmatrix} \qquad \sigma_{ULS} := \begin{bmatrix} 0 \\ -(\sigma_{Nw} + \sigma_{bw}) \\ -\sigma_{Nw} + \sigma_{bw} \\ \sigma_{Nc} - \sigma_{bc} \\ \sigma_{Nc} + \sigma_{bc} \\ 0 \end{bmatrix}$$

Neutral Axis in Timber Section (z):
(measured from bottom of section)

$$A := -(\sigma_{Nw} + \sigma_{bw}) = -4.758$$

$$B := -\sigma_{Nw} + \sigma_{bw} = -0.23$$

$$z := \frac{-A}{\left(\frac{(-A) + B}{(h_2) - 0}\right)} = 104.038 \qquad \text{mm}$$



4.0 Fundamental Frequency Prediction:

4.1 Dynamic Properties of TCC system:

$$m_{tcc} := \frac{DL \cdot \left(\frac{b}{1000}\right)}{9.81} = 0.135$$ kg/m ; mass per unit length

$$M_{tcc} := \frac{m_{tcc} \cdot (l)}{2} = 0.404$$ kg ; Generalized Modal Mass

$$K_{tcc} := \frac{\pi^4 \cdot EI_{effTCC\_SLS}}{2 \cdot L^3} = 824.008$$ kN/mm; Generalized Stiffness

4.2 Prediction:

$$f_n := \frac{1}{2 \cdot \pi} \cdot \left(\sqrt{\frac{K_{tcc}}{M_{tcc}}}\right) = 7.191$$ Hz; Fundamental Frequency

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