# MyTiCon Timber Connectors

WHITE PAPER

## TIMBER CONCRETE COMPOSITE SYSTEMS

**WOOD** you like to **CONNECT?**


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Timber Concrete Composite (TCC) systems are usually a concrete slab connected to a wooden panel or beam. Composite action can be achieved through a mechanical connector, geometric interlocking or adhesive bonding.

The use of TCC systems has been broadly investigated in a variety of different approaches., e.g. T-Beam configurations where an Engineered Wood Product (EWP) is used as the web member and the flange is composed out of concrete (Yeoh et al. 2010)[1] and plate configurations (Gerber 2016)[3]. The use of TCC systems in existing buildings where the structure is not in compliance with modern design requirements has also been investigated. Often, the existing structure is composed out of timber sections and is able to carry additional load but an upgrade is required for serviceability, i.e. deflection and vibration (Cecotti 2002)[2].

TCC systems positively contribute to an enhanced building performance under modern design considerations in three key-areas:

1. Structural Performance - Utilizing a TCC system combines the advantages of wood and concrete. Wood performs well in tension and concrete in compression. The behaviour of both material is well understood. With the appropriate shear connector, stringent ultimate limit state and serviceability limit state design requirements can be met with slender EWP elements and allow for efficient structural design. The composite action allows for narrow beam width and reduced beam depth which may reduce the building height, the overall building envelope and therefore construction and maintenance cost.
2. Human comfort - New or existing structures shall always consider how humans experience the structure they life or work in. Excessive floor vibrations may result in discomfort for occupants. TCC systems can help control floor vibrations and ultimately improve the building experience. Besides floor vibrations, the building climate also contributes to the well-being of building occupants. While a light wood frame structure generally has a low thermal mass and heats up or cools down quickly, a building using TCC floor systems has a higher thermal mass that advances human comfort.
3. Fire performance – In heavy timber buildings, fire safety is commonly addressed through concealing connection systems with a charring layer consisting out of wood. It is also possible to conceal a connector with a layer of concrete to avoid direct heat and/or fire exposure. In a TCC system where the shear connector is covered with wood and concrete, no extra measures for fire protection are required.

Designing an efficient TCC system requires a shear connector for which the stiffness value has been determined through testing. In an ideal world, an easy to install, cost efficient, off the shelf, standardized and approved shear connector is available. Approved and tested full-thread, self-tapping wood screws (STS) offer this opportunity. With the appropriate shear connector slender EWP elements such as Cross Laminated Timber (CLT) can satisfy stringent design requirements and allow for efficient structural design.

This paper presents findings from testing on TCC systems with EWP and STS as shear connector and provides a design example considering a TCC- T-section.

TCC systems are generally designed using the gamma method also called mechanically jointed beam theory. The design approach relies heavily on the shear connector stiffness to be known. If STS are used as shear connector it seems reasonable to use the existing test data and stiffness values to attempt the design of a TCC T - section beam. The concrete flange must act in compression and the timber web member must act in tension. The approach may be particularly valuable in upgrades of existing buildings where occupational demands impose larger loads or more comforting structural performance through deflection reduction and vibration control. A TCC beam system will:

- Help conserve the original timbers and may allow for larger loads on existing beam members while reducing original ceiling height minimally only.
- Optimizes mass - additional introduced load is much smaller when compared to a traditional concrete slab which reduces lateral loads and gravity load demands on existing foundations
- Improved thermal mass performance of the building

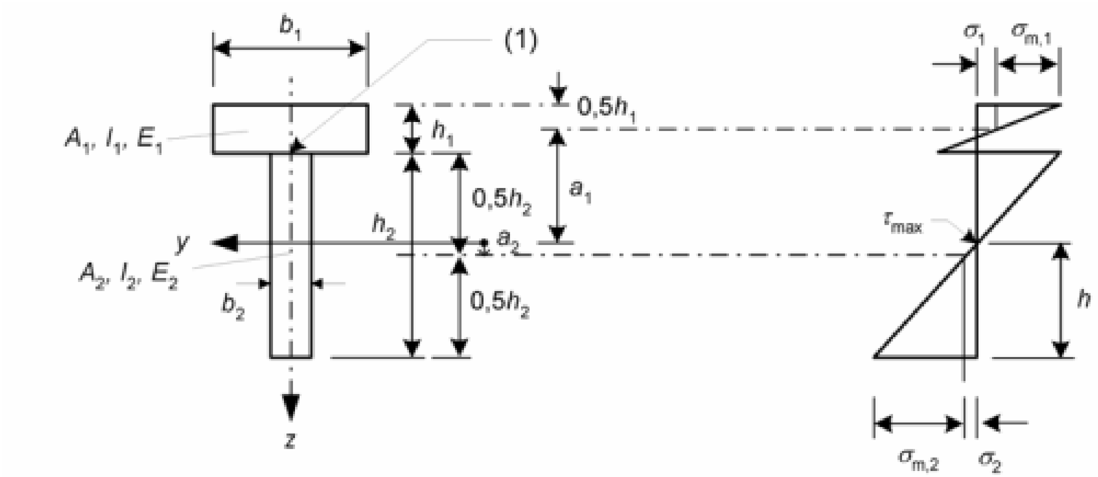


*Figure 1: Cross sectional stress distribution as per [7]*

The design of a TCC T– beam must also consider the active width of the concrete flange $b_{1,eff}$. In [6] a theoretical approach is described referring to the relationship of the flange width with respect to the clear span of the beam.

For uniform loading:

$$b_{1,eff}=S\,(1-1.4\left(\frac{S}{L}\right)^2)$$

For concentrated loading:

$$b_{1,eff}=S\,(1-1.4\left(\frac{S}{L}\right)^2-0.8(\frac{S}{L}))$$

*S = Spacing of timber girder and L = Clear span of the girder*

When designing TCC systems one of the most important input parameters to consider are the properties of the shear connector. In order to utilize standard, off the shelf code approved STS in TCC systems small and full scale tests have been conducted. The outcome of the testing provides information about the shear connector properties in Table 2. A variety of results were presented in one of our previous papers (Figure 2) however design parameters of TCC systems with a soft and lightweight interlayer (Figure 3) were not considered . The soft and light weight interlayer is constructed of rigid insulation material with 1" (25mm) thickness. The advantage is that beneficial cross sectional depth is added without adding much weight (besides other benefits). Since the STS screws are installed at an angle to the shear plane for optimal stiffness a normal clamping force is generated (Figure 3) which can compress/crush the soft and light weight interlayer. To avoid crushing a screw cross consisting out of two equivalent screws in opposite direction is installed.

Presented are the research results to address TCC systems with a 25mm thick interlayer.

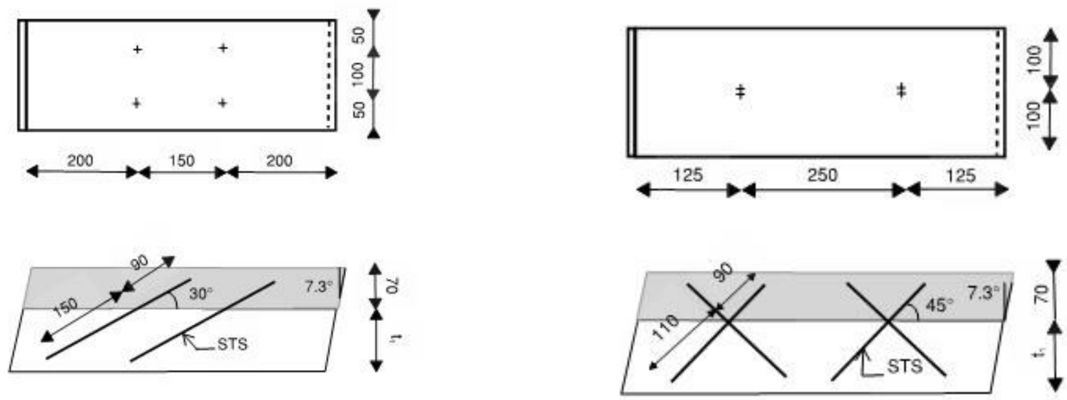


*Figure 2: STS @30° and 45° angle to the shear plane without interlayer as per [3] - dimensions in mm*

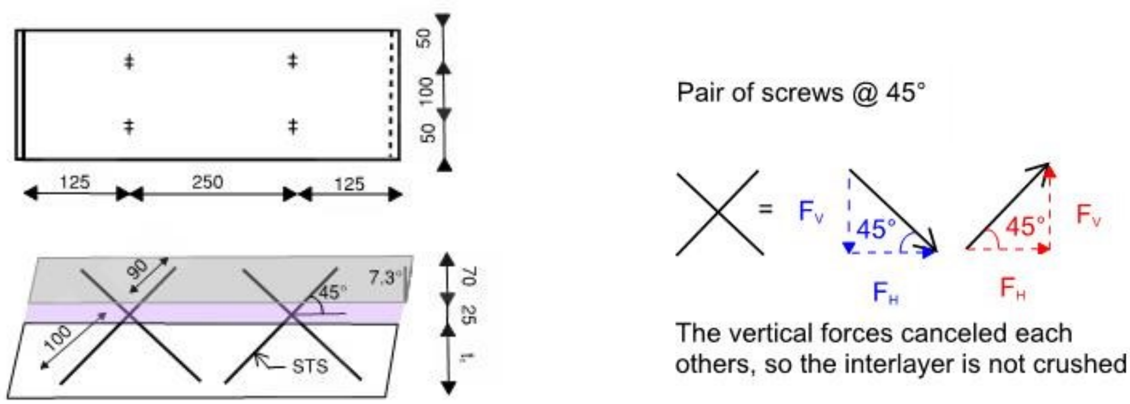


*Figure 3: STS @45° to the shear plane with interlayer as per [3] and resulting forces– dimensions in mm*

Fasteners such as STS exercise a certain inelastic slip under loading. According to EN 26891 (CEN 1991)[4] loading to 40% of the estimated failure load before loading to failure will allow to compute tangent stiffness that is not significantly impacted by the initial slip which would only be expected in a structure when loading is applied for the first time[3]. Details on respective derivations are provided in Table 1.

*Table 1 : Design parameter derivations[3]*

| Description | Equation |
|---|---|
| Tangent stiffness at 40% of peak load (used for Serviceability Limit State) | $K_{0.4} = \frac{F_{0.4}-F_{0.1}}{\Delta_{0.4}-\Delta_{0.1}}\ [kN/mm]$ |
| Tangent stiffness at 60% of peak load | $K_{0.6} = \frac{F_{0.6}}{\Delta_{0.6}-\Delta_{res}}\ [kN/mm]$ |
| Tangent stiffness at 80% of peak load (used for Ultimate Limit State) | $K_{0.8} = \frac{F_{0.8}}{\Delta_{0.8}-\Delta_{res}}\ [kN/mm]$ |
| Tangent stiffness at peak load | $K_{ult} = \frac{F_{ult}}{\Delta_{ult}-\Delta_{res}}\ [kN/mm]$ |
| Peak load recorded | $F_{ult}\ [kN]$ |
| Inelastic slip after loading to service level | $\Delta_{res} = \Delta_{0.4} - \frac{F_{0.4}}{K_{0.4}}\ [kN/mm]$ |

K = slip modulus in kN/mm

$F_{0.1}$ = 10% of peak load, $F_{0.4}$ = 40% of peak load, $F_{0.6}$ = 60% of peak load, $F_{0.8}$ = 80% of peak load

$F_{ult}$ = Peak load recording

$\Delta_{0.1}$ = Displacement at 10% of peak load, $\Delta_{0.4}$ = Displacement at 40% of peak load

$\Delta_{0.6}$ = Displacement at 60% of peak load, $\Delta_{0.8}$ = Displacement at 80% of peak load

$\Delta_{res}$ = Recorded inelastic slip after service load application

$\Delta_{ult}$ = Recorded ultimate displacement

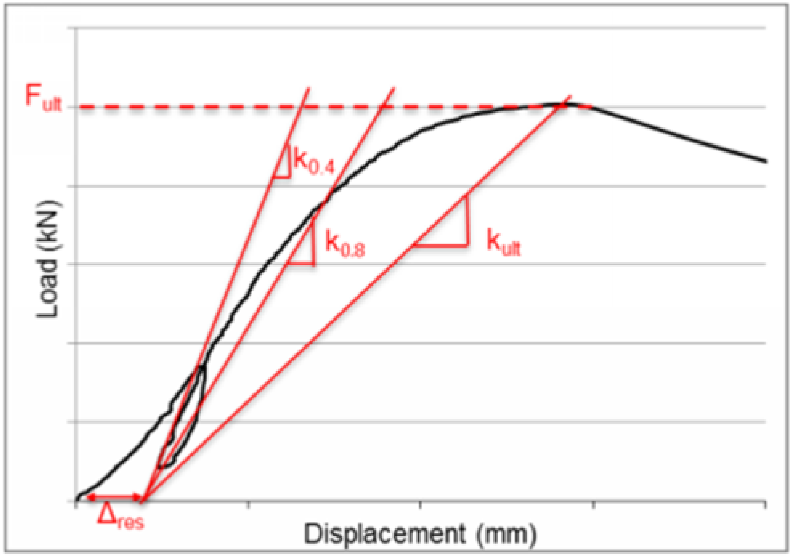


*Figure 4: Graphical display of design parameter derivation as per [3]*

Small scale testing was required to determine the shear connector performance values for TCC design (Figure 5). From this test setup the design parameters presented in Table 2 were derived.


*Figure 5: Test Setup of the Shear Test: schematic (left) and picture (right) as per [3]*

*Table 2: Summary of Small Scale Test Results for pair of STS @45° [3]*

| SMALL SCALE SPECIMEN TEST RESULTS | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | | | Average Recorded Stiffness | | | |
| **Connector description (results per pair of screws)** | **EWP** | **$F_{ult}$ (kN)** | **$\Delta_{res}$ (mm)** | **$K_{0.4}$ (kN/mm)** | **$K_{0.8}$ (kN/mm)** | **$K_{ult}$ (kN/mm)** | **# of tests** |
| ASSY fully threaded VG Pair 10 x 200 @ 45° | LSL | 49 | 0.06 | 74.4 | 41 | 19.2 | 6 |
| ASSY fully threaded VG Pair 10 x 220 @45° + 25 mm Insulation | | 30 | 0.01 | 61.7 | 38.8 | 26.9 | 6 |
| ASSY fully threaded VG Pair 10 x 200 @ 45° | LVL | 34.9 | 0.14 | 55.6 | 30.2 | 10.3 | 6 |
| ASSY fully threaded VG Pair 10 x 220 @45° + 25 mm Insulation | | 26.3 | 0.03 | 47.3 | 35.1 | 38.2 | 6 |
| ASSY fully threaded VG Pair 10 x 200 @ 45° | CLT | 30.6 | 0.13 | 67.5 | 22.4 | 8.8 | 6 |
| ASSY fully threaded VG Pair 10 x 220 @45° + 25 mm Insulation | | 23.6 | 0.07 | 42.8 | 24.9 | 18.1 | 5 |

LSL = Laminated Strand Lumber, LVL = Laminated Veneer Lumber

$F_{ult}$ = Recorded peak load

$\Delta_{res}$ = Recorded inelastic slip after loading of specimen to service level load

$K_{0.4}$, $K_{0.8}$, $K_{ult}$ as outlined in Table 1

A design example of a TCC -T beam is presented on the following pages. A summary of the results and possibilities in reducing deflection and timber cross section are presented in Table 3 and Table 4. Based on the boundary conditions used in the design example it seems possible to reduce deflections while reducing the required timber cross sections.

*Table 3: Estimate –Comparing T-Beam without composite action and TTC T-Beam system*

| | **Floor system without composite action** | **TTC T-Beam system** |
|---|---|---|
| Description :<br>GL D-Fir-L 20f-E Beam, Clear Span 11.50 m,<br>Spacing 2.40m & Live Load 4.8 kpa | Concrete flange 150 mm + T&G 50.8 mm | |
| Dead Load of the floor system kN/m$^2$ | 5.94 | |
| GL cross-section $b_2$ x h $_{wood}$: (mm) | **365 x 760** | |
| Deflection Live Load : $\Delta_{360}$ (mm) | 25.35 | 8.15 |
| Deflection Dead Load +Live Load: $\Delta_{180}$ (mm) | 56.70 | 18.23 |
| Estimated Natural Frequency : fn (Hz) | 4 | 7 |

*Table 4: Comparison—T-Beam without composite action and TTC T-Beam system*

| | **T-Beam without composite action** | **TTC T-Beam system** |
|---|---|---|
| Description :<br>GL D-Fir-L 20f-E Beam, Clear Span 11.50 m,<br>Spacing 2.40m & Live Load 4.8 kpa | Concrete 150 mm + T&G 50.8 mm | Concrete 125 mm + T&G 50.8mm |
| Dead Load of the floor system kN/m$^2$ | 5.94 | 5.07 |
| GL cross-section $b_2$ x h $_{wood}$: (mm) | **365 x 760** | **265 x 608** |
| Deflection Live Load : $\Delta_{360}$ (mm) | 25.35 | 17.85 |
| Deflection Dead Load +Live Load: $\Delta_{180}$ (mm) | 56.70 | 36.71 |
| Estimated Natural Frequency : fn (Hz) | 4 | 5.3 |


*Figure 6: Project example TCC system utilizing STS @ 30° (photo credit Western Archrib—Structural Wood Systems)*

**Note to the designer:** The calculations provided in this example are sample calculations only! Under no conditions shall they be viewed as a generally valid design guideline. It remains the responsibility of a qualified design professional to design a TCC system to the specifics of a project and in compliance with local design standards.

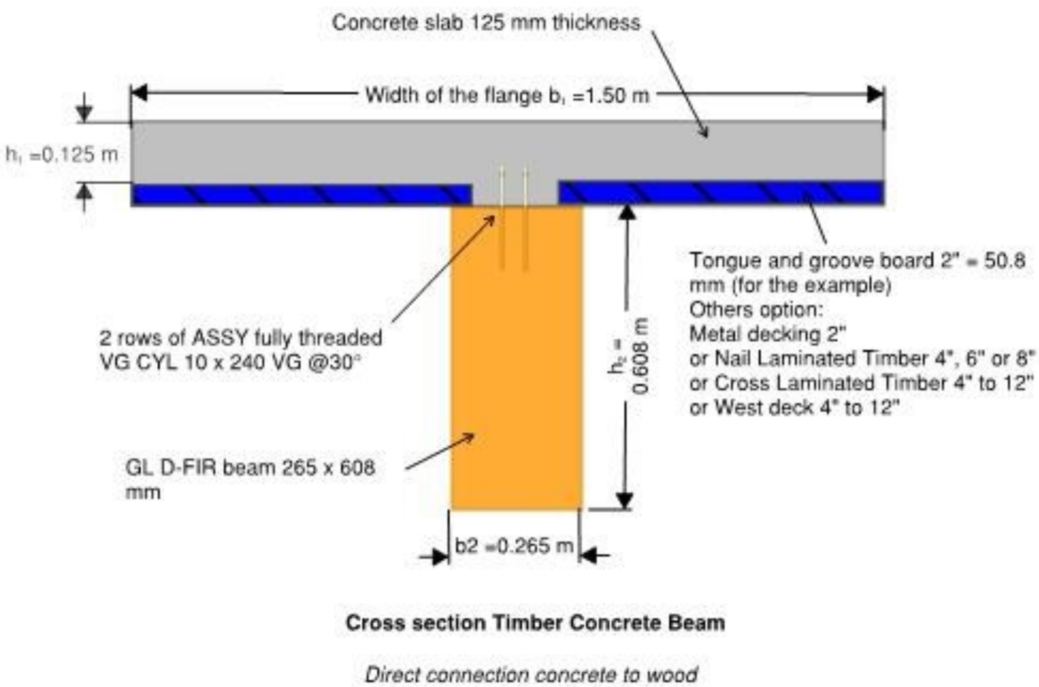


*Figure 7: Timber Concrete Composite Section—T-Beam*

## 1.0 Geometry of Timber Concrete Composite section

*Thickness of layer*

| | | | |
|---|---|---|---|
| $h_{conc}$ = | 125 | [mm] | Concrete Floor thickness |
| $h_b$ = | 50.8 | [mm] | Thickness of the Tongue & groove board |
| $h_{wood}$ = | 608 | [mm] | Depth of the glulam beam |
| $h_{tot.}$= | 783.8 | [mm] | Total TCC section Depth $\quad h_{tot.}=h_{conc.}+h_{b.}+h_{wood}$ |
| $b_1$ = | 1,500 | [mm] | *note: $b_1 < b_{1,eff}$ conservative estimate* |
| | | | $b_{1,eff}=S\,(1-1.4\left(\frac{S}{L}\right)^2)$ |
| | | | $b_{1,eff}=2400\,(1-1.4\left(\frac{2,400}{11,500}\right)^2)$ |
| $b_{1,eff}$ = | 2,254 | [mm] | According to Dragoslav Stojie, Radovan Cvekovie, the active width of slab for uniform loading [6] |
| S= | 2,400 | [mm] | Spacing of glulam girders |
| L = | 11,500 | [mm] | Clear span of single beam , (assuming simple span) |
| $b_2$ = | 265 | [mm] | Width of the glulam beam |

## 2.0 Geometrie of Timber composite Section

| | | | |
|---|---|---|---|
| Weight: | 337 | [kg/m²] | Effective weight of floor system |

$$\text{Weight} = \frac{h_{conc}}{1000}.24\frac{kN}{m^3} + \frac{h_{wood}}{1000}.\frac{b_2}{1000}.\frac{5.5\frac{kN}{m3}}{S}.(100)$$

*Concrete Material Properties:*

| | | | |
|---|---|---|---|
| $f'_c$ = | 30 | [MPa] | Specified compressive strenght of concrete at 28 days |
| $f_r$ = | 3.286 | [MPa] | Factored Concrete Strenght CSA A23.3 ( art. 8.4.2) [13] |

$$f_r = 0.6\sqrt{f'_c} = 0.6\sqrt{30}Mpa$$

| | | | |
|---|---|---|---|
| $E_c$ = | 24,648 | [MPa] | Modulus of Elasticity CSA A23.3 (art. 8.6.2.3) |

$$E_c = 4500\sqrt{f'_c} = E_c = 4500\sqrt{30}Mpa$$

| | | | | |
|---|---|---|---|---|
| $A_c$ = | 187,500 | [mm²] | Cross sectional Area of Concrete topping | $A_c = b_1.h_c = 1{,}500.(125)$ |
| $I_c$ = | 2.44E+08 | [mm⁴] | Moment of Inertia of Concrete Topping | $I_c = \frac{b_1.hc^3}{12} = \frac{1{,}500.(125)^3}{12}$ |
| $E_{wood}$ = | 12,400 | [MPa] | Modulus of Elasticity of Glued laminated timber Douglas Fir-L 20f-E | |
| $EI_{eff}$ = | 6.15E+13 | [N-mm²] | Effective Bending Stiffness of the glued laminated timber beam | |
| $A_{wood}$ = | 161,120 | [mm²] | Cross sectional Area of glulam beam | $A_{wood} = b_2.h_{wood} = 608.(265)$ |
| $I_{wood}$ = | 4.96E+09 | [mm⁴] | Moment of Inertia of glued-laminated timber | $I_{wood} = \frac{b_2.hwood^3}{12} = \frac{265.(608)^3}{12}$ |
| $F_b$ = | 19.2 | [MPa] | Factored Bending Strength, for glued laminated timber | |
| $F_{tn}$ = | 20.4 | [MPa] | Factored Tension Strength, for glued laminated timber | |
| $F_v$ = | 2 | [MPa] | Factored Longitudinal shear for glued laminated timber | |

*note : assume standard conditon for glulam resistance factors ( =1)*

### 3.0 Mechanical Properties for ASSY VG Cyl 10x240mm screw as Shear Connector:

| | | | |
|---|---|---|---|
| $d$ = | 10 | [mm] | Outside thread diameter |
| $l_{tot}$ = | 240 | [mm] | Total screw length |
| $l_{ef}$ = | 170 | [mm] | Effective penetration length |
| $\alpha$ = | 30 | | |
| | 0.52 | [rad] | Angle of axis of screw to shear plane |
| $n_r$ = | 2 | | Number of rows (per beam) |
| $s_e$ = | 150 | [mm] | spacing of screws at ends |
| $s_m$ = | 300 | [mm] | Spacing of screws at midspan |

$$S_{eff} = 0.75\,se + 0.25\,sm = 0.75\,(150mm) + 0.25\,(300\,mm)$$

*according to Ceccotti (2002)*

| | | | |
|---|---|---|---|
| $s_{eff}$ = | 187.5 | [mm] | Effective spacing of screws |
| $K_{def\_c}$ = | 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 (EN-1995, Appendix B) |
| $\psi_2$ = | 0.3 | | Stiffness reductionfor ULS for long term loading (EN-1995, Appendix B) |

### 3.1 Connector Specific Design Parameters (from test results) :

| | | | |
|---|---|---|---|
| $K_{SLS}$ = | 69,000 | [N/mm] | Screw stiffness at SLS for short term loading |
| $K_{ULS}$ = | 19,600 | [N/mm] | Screw stiffness at ULS for short term loading |
| $F_{rK}$ = | 34,400 | [N] | Mean value for load carrying capacity of screw |

### 3.2 Connector Stiffness Adjustment for Long Term Loading (EN1995):

| | | | |
|---|---|---|---|
| $K_{ser\_LT}$ = | 43,125 | [N/mm] | Adjusted screw stiffness at Serviciability Limit State (SLS) for long term loading |

$$K_{SER\_LT} = \frac{K_{SLS}}{1+kdef_{\_sts}} = \frac{69{,}000}{1+0.6}$$

| | | | |
|---|---|---|---|
| $K_{ULS\_LT}$ = | 16,610 | [N/mm] | Adjusted screw stiffness at Ultimate Limit State (ULS) for long term loading |

$$K_{ULS\_LT} = \frac{K_{SLS}}{1+(\psi_2 . kdef_{\_sts})} = \frac{19{,}600}{1+(0.3.(0.6))}$$

*Note:*

*ULS : Ultimate Limit State*

*SLS: Serviceability Limit State*

### 3.3 Modulus adjustment for long term loading

*for concrete*

$E_{1\_ULS\_LT}$: 14,084 [MPa] $E_{1\_ULS\_LT} = \frac{E_1}{1+(\psi_2 . Kdef_{\_c})} = \frac{24,648}{1+(0.3\,(2.5))}$

$E_{1\_SLS\_LT}$: 7,042 [MPa] $E_{1\_SLS\_LT} = \frac{E_1}{1+Kdef_{\_c}} = \frac{24,648}{1+2.5}$

*for glued laminated timber*

$E_{2\_ULS\_LT}$: 10,508 [MPa] $E_{2\_ULS\_LT} = \frac{E2}{1+(\psi_2 . Kdef_{\_t})} = \frac{12,400}{(1+0.3(0.6))}$

$E_{2\_SLS\_LT}$: 7,750 [MPa] $E_{2\_SLS\_LT} = \frac{E_2}{1+Kdef_{\_t}} = \frac{12,400}{1+0.6}$

### 4.0 Calculation of Gamma Factor for Composite Section (EN-1995, Appendix B)

*ULS at Long Term Loading*

$\gamma_{1\_ULS}$ = **0.697** "Gamma" Factor for Composite Section ( Shear *Connector Reduction Factor* )

$$\gamma_i = [1 + \pi^2 . EiAi . Si/(n_r . Ki . L^2)]^{-1} \quad \textit{for i=1 and i=3}$$

$$\gamma_1 = [1 + \pi^2 . EA_{r\_ULS} . S_{eff}/(n_r . K_{ULS\_LT} . L^2)]^{-1}$$

$$\gamma_1 = [1 + \pi^2 . (1.03E+09) . (187.5)/(2 . (16,610) . 11,500^2)]^{-1}$$

*Note: Full Composite action in TCC is achieved when the concrete slab and wooden beam are rigidly connected*
***( γ = 1 full composite action) composite element 0 ≤ γ ≤1 ( γ = 0 no composite action )***

$\gamma_2$ = 1.0 Assumption from EN-1995, Appendix B

$Ea_{r\_ULS}$ = 1.03E+09 (Ceccotti; Gerber et. al)

$$(EA)_r = \frac{(E_1 . A_1) . (E_2 . A_2)}{(E_1 . A_1) + (E_2 . A_2)} = \frac{(E_{1\ ULS} . A_1) . (E_{2\ ULS} . A_2)}{(E_{1\ ULS} . A_1) + (E_{2\ ULS} . A_2)}$$

$$(EA)_r = \frac{(14,084) . (187,500) . (10,508) . (161,120)}{(14,084) . (187,500) + (10,508) . (161,120)}$$

Note: $(EI)_{eff}$ depends on the load distribution and span so it is not a fundamental cross-sectional property (Ceccoti 2002)

$a_{2\_ULS}$ = 204.1 [mm] Distance to centroid of Timber section from Neutral Axis

$$a_{2\_ULS} = \frac{(\gamma_{1\ ULS} . E_{1\ ULS} . A_1 . (h_{conc} + hb + h_{wood}))}{(2 . (\gamma_{1\ ULS} . E_{1\ ULS} . A_1 + E_{2\ ULS} . A_2))}$$

*Note: $h_{total}$ instead of $h_1 + h_2$*

$$a_{2\_SLS} = \frac{(0.697\ (14{,}084) . (187{,}500) . (125 + 50.8 + 608))}{(2 . (0.697\ (14{,}084) . (187{,}500) + (10{,}508) . (161{,}120))}$$

$a_{1\_ULS}$ = 213.2 [mm] Distance to centroid of Concrete section from Neutral Axis

$$a_{1\_ULS} = \frac{hc}{2} + hb + \frac{hwood}{2} - a_{2\_ULS} = \left(\frac{125}{2} + 50.8 + \frac{608}{2}\right) - 204.1 = 213.2$$

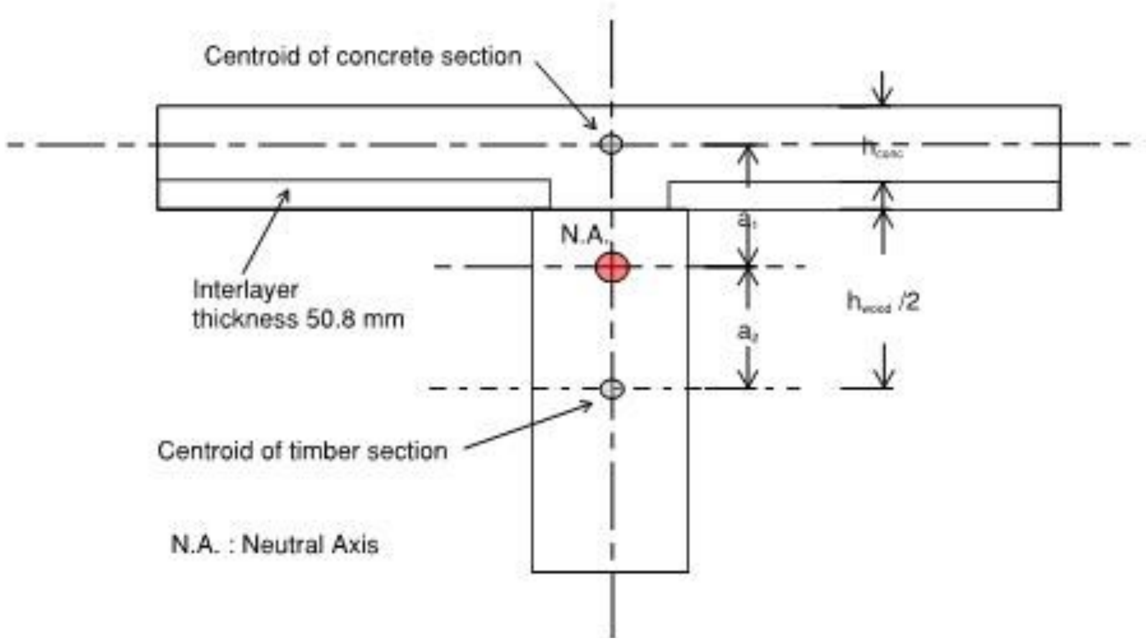


*Figure 8: Location estimate of the Neutral Axis in the TTC T-Beam system*

**$Ei_{eff\_ULS}$ = 2.10E+14** [N-mm²] Effective Bending Stiffness of TCC System

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i\, Ii + \gamma_i\, Ei\, Ai\, ai^2)$$

$Ei_{eff\_ULS} = (E_{1\_ULS}.I_1 + (\gamma_{1\_ULS}.E_{1\_ULS}.A_1.a_{1\_ULS}^2) + (E_{2\_ULS}.I_2 + (E_{2\_ULS}.A_2.a_{2\_ULS}^2)$

$Ei_{eff\_ULS} = (14{,}084).(2.44E+08) + 0.697\ (14{,}084).(187{,}500).(213.2)^2 + (10{,}508).(4.96\ E+09) + (10{,}508).(161{,}120).(204.1^2)$

*SLS at Long Term Loading*

$\gamma_{1\_SLS}$ = **0.906** Shear *Connector Reduction Factor at SLS*

$$\gamma_{1\_SLS} = [1 + \pi^2.EA_{r\_SLS}.S_{eff}/(n_r.K_{SLS\_LT}.L^2)]^{-1}$$

$$\gamma_{1\_SLS} = [1 + \pi^2.(6.42E + 0.8).(187.5)/(2.(43{,}125).11{,}500^2)]^{-1}$$

$Ea_{r\_SLS}$ = 6.42E+08 (Ceccotti; Gerber et. al)

$$(EA)_{r\_SLS} = \frac{(E_{1\ SLS}.A_1).(E_{2\ SLS}.A_2)}{(E_{1\ SLS}.A_1) + (E_{2\ SLS}.A_2)}$$

$$(EA)_r = \frac{(7{,}042).(187{,}500).(7{,}750).(161{,}120)}{(7{,}042).(187{,}500) + (7{,}750).(161{,}120)}$$

$a_{2\_SLS}$ = 191.7 [mm] Distance to centroid of Timber section from Neutral Axis

$$a_{2\_SLS} = \frac{(\gamma_{1\ SLS}.E_{1\ SLS}.A_1.(h_1 + h_2)}{(2.(\gamma_{1\ SLS}.E_{1\ SLS}.A_1 + E_{2\ SLS}.A_2))}$$

*Note: $h_{total}$ instead of $h_1 + h_2$*

$$a_{2\_SLS} = \frac{(0.906\ (7{,}042).(187{,}500).(125 + 608 + 50.8))}{(2.(0.906\ (7{,}042).(187{,}500) + (7{,}750).(161{,}120))}$$

$a_{1\_SLS}$ = 225.6 [mm] Distance to centroid of Concrete section from Neutral Axis

$$a_{1\_ULS} = \frac{hc}{2} + hb + \frac{hwood}{2} - a_{2\_ULS} = (\frac{125}{2} + 50.8 + \frac{608}{2}) - 191.7 = 255.6$$

**$Ei_{eff\_SLS}$ = 1.47E+14** [N-mm²] Effective Bending Stiffness of TCC System

$Ei_{eff\_SLS} = (E_{1\_SLS}.I_1 + (\gamma_{1\_SLS}.E_{1\_SLS}.A_1.a_{1\_SLS}^2) + (E_{2\_SLS}.I_2 + (E_{2\_SLS}.A_2.a_{2\_SLS}^2)$

$Ei_{eff\_SLS} = (7{,}042).(2.44E+08) + 0.906\ (7{,}042).(187{,}500).(225.6)^2 + (7{,}750).(4.96\ E+09) + (7{,}750).(161{,}120).(191.7^2)$

### 5.0 Loading Conditions for TCC System

| | | | |
|---|---|---|---|
| S = | 2.40 | [m] | Spacing of glulam girders |
| b = | 1.50 | [m] | Effective width of slab |
| L= | 11.50 | [m] | Clear span of single beam, (assuming simple span) |
| | 0.37 | [kPa] | Weight of the glulam beam |
| | 1.70 | [kPa] | Partition wall + Tongue & groove board (50.8 mm) + electrical & mechanical |
| | 3.00 | [kPa] | concrete slab 125 mm |
| DL = | 5.07 | [kPa] | DL of System + partition wall |
| LL = | 4.80 | [kPa] | Live Load |
| $w_{DL}$ = | 12.17 | [kN/m] | Unfactored uniformly distributed Dead Load |
| | | | $W_{DL}$=DL.(S) = 5.07 (2.40) = 12.17 |
| $w_{LL}$ = | 11.52 | [kN/m] | Unfactored uniformly distributed Live Load |
| | | | $W_{LL}$=LL.(S) = 4.80 (2.40) = 11.52 |
| $w_{ULS}$ = | 32.49 | [kN/m] | Total Factored uniformly distributed load (ULS) |
| | | | $w_{ULS}$ = (1.25 DL +1.5 LL) = 1.25 (12.17) + 1.5 (11.52) |
| $V_f$ = | 186,804 | [N] | Factored Shear Force |
| | | | $V_f = \frac{W_{ULS} L}{2} . 1000 = \frac{32.49\ (11.50)}{2} . 1000$ |
| $M_f$ = | 5.37E+08 | [N-mm] | Factored Bending Moment (assuming simple span) |
| | | | $M_f = \frac{W_{ULS} L^2}{8} . (1000)^2 = \frac{32.49\ (11.50)^2}{8} . (1000)^2$ |
| $\sigma_{N,c}$ = | 5.36 | [MPa] | max normal compressive stress in concrete section |
| | | | $\sigma_{N,C} = \frac{\gamma_1 . E_1 . a_1 . Mf}{(EI)_{ef}} = \frac{0.697\ (7{,}04214{,}084)(204.1).(5.37\ E+08)}{2.10E+14}$ |
| $\sigma_{b,c}$ = | 2.25 | [MPa] | max bending stress in concrete section |

$$\sigma_{b,C} = \frac{0.5 . E_1 . h_1 . Mf}{(EI)_{ef}} = \frac{0.5 . (14{,}084) . (125) . (5.37E + 08)}{2.10E + 14}$$

$\sigma_{N,w}$ = 5.49 [MPa] max normal tension stress in wood section

$$\sigma_{N,W} = \frac{\gamma_2.E_2.a_2.Mf}{(EI)ef} = \frac{1.(10{,}508).(204.1).(5.37E+08)}{2.10E+14}$$

$\sigma_{b,w}$ = 8.18 [MPa] max bending stress in wood section

$$\sigma_{b,W} = \frac{0.5.E_2.h_2.Mf}{(EI)_{ef}} = \frac{0.5.(10{,}508).(608).(5.37\,E+08)}{2.10\,E+14}$$

$\tau_{w,max}$ = 1.73 [MPa] max shear stress in wood section

$$\tau_{2,max} = \frac{0.5.E_2.b_2.h_2.}{b_2(EI)_{ef}}.V = \frac{0.5.(10{,}508).(265).(608).}{(265).(2.10E+014)}.(186{,}804)$$

$F_{max}$ = 20.11 [kN] max load on fastener at section where shear force was computed

$$F_{2,max} = \frac{\gamma_1.E_2.A_2.a_2.seff}{n_r(EI)_{ef}}.\frac{V}{1000} = \frac{0.697.(10{,}508)\,(161{,}120).(204.1).(187.5)}{2.(2.10\,E+014)}.\frac{(186{,}804)}{1000}$$

### 6.0 Strength and Stiffness Checks

*Deflection check*

$\Delta max$ = 31.9 [mm] max deflection allowed (L/360) under LL (11,500/360)

$\Delta LL$ = 17.85 [mm] max deflection at mid-span due to unfactored Live Loading

$$\Delta_{LL} = \frac{5.WLL.L^4}{384.E.I._{SLS}} = \frac{5.(11.52).(11{,}500)^4}{384.(1.47\,E+14)}$$

L/h = 14.7 span-to-depth ratio of section

**0.559** OK Utilization Ratio

$\Delta max$ = 63.89 [mm] max deflection due to unfactored Dead Load + Live Load (L/180)

$\Delta DL$ = 36.71 [mm] $\Delta_{DL} = \frac{5.WLL.L^4}{384.E.I._{SLS}} = \frac{5.(11.52+12.17).(11{,}500)^4}{384.(1.47\,E+14)}$

**0.575** OK Utilization Ratio

*Strength Checks:*

***Glulam :***

| | | | |
|---|---|---|---|
| $\phi F_b =$ | 19.2 | [MPa] | Factored Bending Strength of Wood section |
| | **0.426** | OK | Utilization Ratio |

$$\frac{\sigma_{b\,w}}{\varphi.\dot{F}b} = \frac{19.2}{8.18} \leq 1$$

| | | | |
|---|---|---|---|
| $\phi F_t =$ | 20.4 | [MPa] | Factored Tension Strength of Wood section |
| | **0.269** | OK | Utilization Ratio |

$$\frac{\sigma_{N\,w}}{\varphi.\dot{F}t} = \frac{20.4}{5.49} \leq 1$$

*Combined Stress Check*

| | | |
|---|---|---|
| **0.695** | OK | Utilization Ratio |

$$0.426 + 0.269 < 1$$

Shear Resistance Check

| | | | |
|---|---|---|---|
| $V_r =$ | 2 | [MPa] | Factored Shear Resistance of Wood section |
| | **0.865** | OK | Utilization Ratio |

$$\frac{\tau_{2\ max}}{Vr} = \frac{1.73}{2} < 1$$

***Shear Connector:***

| | | | |
|---|---|---|---|
| $F_{rK} =$ | 34.40 | [kN] | mean value for load carrying capacity of screw (from testing) |
| | **0.585** | OK | Utilization Ratio |

$$\frac{F_{max}}{F_{rkr}} = \frac{20.11}{34.4} < 1$$

***Concrete :***

| | | | |
|---|---|---|---|
| $\phi f'_c =$ | 19.50 | [MPa] | Compressive strength of Concrete as per CSA-A23.3 |
| | **0.390** | OK | Utilization Ratio |

$$\frac{\sigma_{N\,c} + \sigma_{b\,c}}{\varphi_{f'c}} = \frac{5.36 + 2.25}{19.50} < 1$$

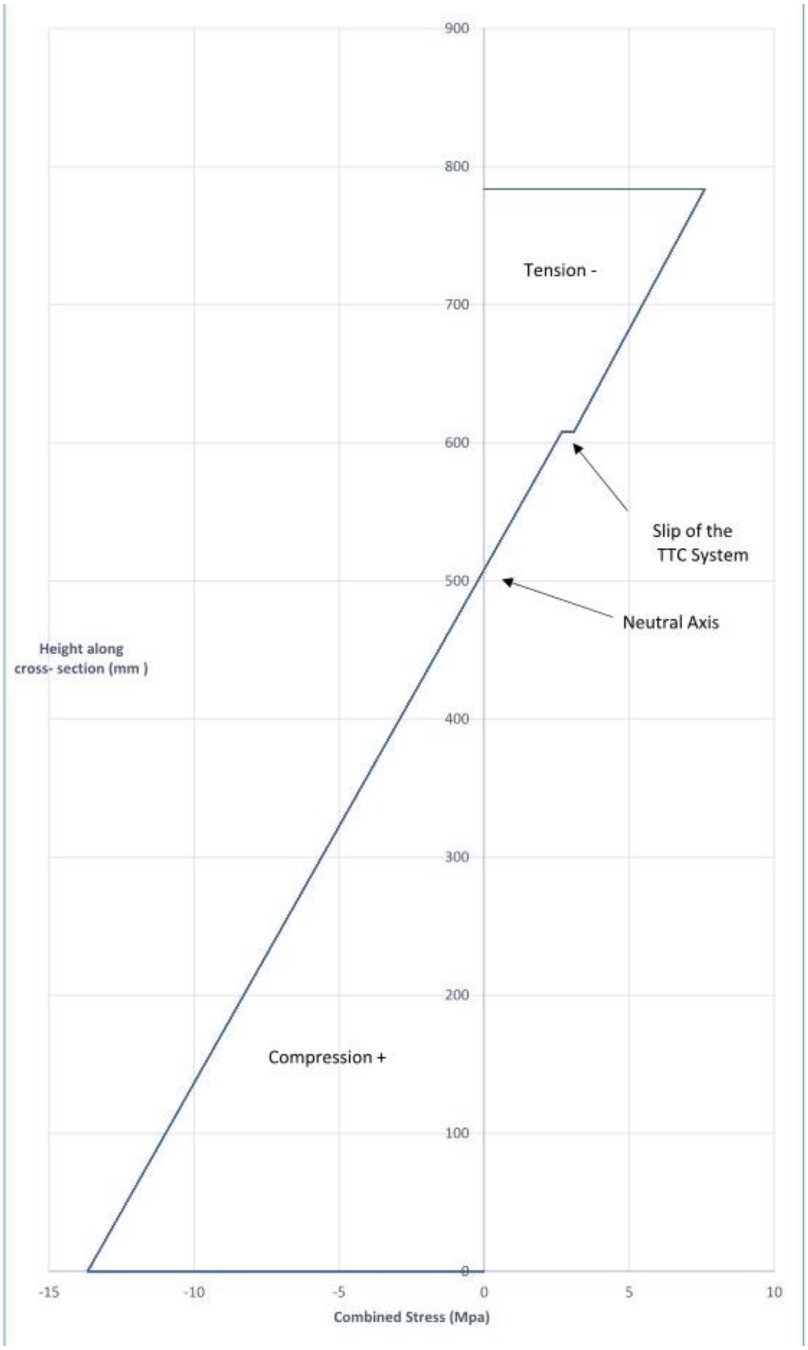


*Figure 9: TCC Stress Distribution estimate*

**Vibration Check**

| | | | |
|---|---|---|---|
| W = | 336.9 | [kg/m^2] | Effective Weight per area of Floor System |
| $m_{tcc}$ = | 808.6 | [kg/m] | Mass per unit length (W.L) = (336.9). (2.40) |
| $M_{modal}$ = | 4,650 | [kg] | generalized modal mass |

$$M_{modal} = \frac{m_{tcc}.(l)}{2} = \frac{808.6\ (11.50)}{2}$$

| | | | |
|---|---|---|---|
| K = | 5,176 | [N/mm] | generalized stiffness |

$$K_{tcc} = \frac{\pi^4.EI_{effTCC\ SLS}}{2.L^3}.\ x1.10 = \frac{\pi^4.(1.47E+14)}{2.(11{,}500)^3}.\ (1.10)$$

| | | | |
|---|---|---|---|
| $f_n$ = | 5.3 | [Hz] | natural frequency |

$$f_n = \frac{1}{2.\pi}.\left(\sqrt{\frac{K_{tcc}}{M_{tcc}}}\right) = \frac{1}{2.\pi}.\left(\sqrt{\frac{5{,}176}{4{,}650}}\right)$$

Outlook:

TCC systems in large wood structures may be used in a variety of different configurations. It may be possible to create a mass timber section consisting out of a GL beam, a mass timber panel and a concrete floor surface. The T– Timber section can be pre– manufactured in a shop with pre-installed connector mounted to the beams. After dropping the T section in place on site concrete can be poured.

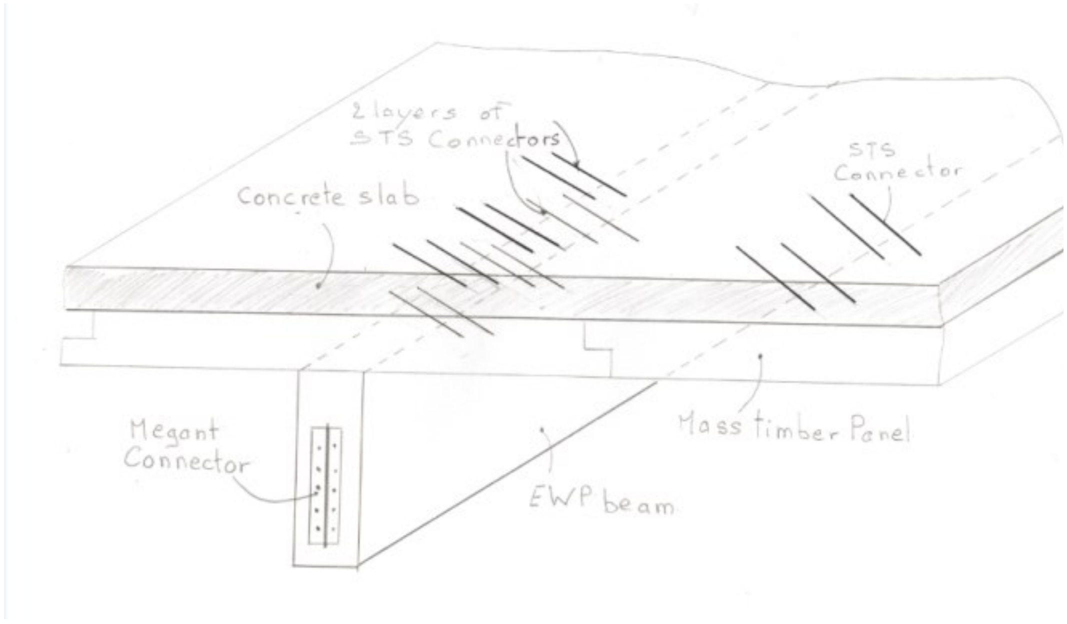


*Figure 10: TCC System with Mass Timber panel flange and concrete topping*

**REFERENCES**:

[1] Yeoh David Eng Chuan (2010) "Behaviour and Design of Timber-Concrete Composite Floor Systems", PhD thesis, University of Canterbury Chistchurch New Zealand

[2] A. Cecotti (2002) " Composite Concrete-Timber Structures, Prog. Struc. Engineering Mater. 2002; 4:264.

[3] Gerber, A. Thesis, Timber-Concrete Composite Connectors in Flat-Plate Engineered Wood Products

[4] EN 26891 (CEN 1991) Timber Structures—Joints made with mechanical fasteners—general principles for the de termination of strength and deformation characteristics

[5] Peggi , C and Schreyer A.(2008). Design and Use of wood-Concrete Composites

[6] Dragoslav Stojie, Radovan Cvekovie (2001) Analysis of a composite timber-concrete Structures according to the limit states

[7] Eurocode 5 Part 1 - EN95-1-1:2004. Design of Timber Structures. Annex B (Informative). Simplified Analysis for Mechanically Jointed Beams.

[8] Evaluation Report - SWG ASSY® VG Plus and ASSY® 3.0 Self-Tapping Wood Screws. (2013) CCMC 13677-R. Canadian Construction Materials Centre. Ottawa, Canada.

[9] Franklin, K. and Hough, R. (2014). Modelling and Measurement of the Dynamic Performance of a Timber Concrete Composite Floor. Proceedings from World Conference on Timber Engineering. Quebec City, Canada. August, 2014.

[10] Gerber, A. and Tannert, T. (2015). Full Scale Testing of Timber-Concrete Composite Floor Systems. The University of British Columbia. Version 2. October 30, 2015

[11] Hanes RM. (1970). Human sensitivity to whole-body vibration in urban transportation systems: a literature review. Silver Springs (MD): Applied Physics Laboratory. The John Hopkins University.

[12] Rijal, R. et al (2015). Experimental and analytical study on dynamic performance of timber-concrete composite beams. Construction and Building Materials 75, p.46-53.

[13] CSA A23.3—Concrete Design Handbook

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