DEVELOPMENT OF TIMBER CONCRETE COMPOSITE (TCC) BEAM WITH POST-TENSIONING

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ABSTRACT

Post-tensioning is a method of reinforcing (strengthening) concrete or other materials with high-strength steel strands or bars. The timber type that is used in all researches is the engineering timber products laminated veneer lumber (LVL) and glue-laminated wood (Glu-Lam), while avoiding using natural timber due to its properties' variety. More recent improvement is using post-tensioning technique for the engineering timber products. The main material in this research is natural Kempas timber. The primary objective of this research is to develop of a method of timber beam post-tensioning and proposed empirical Equation to predict the bending moment capacity for timber beam and timber concrete composite beam through the bending strength properties. The experimental program is divided into 3 major phases across the duration of this study. Phase A present post-tensioning method, this phase indicate two methods of post-tensioning "forced bending jacking PT-B" and "pre-stressing jacking PT-J". Phase B present timber bending performance in term of (strength and failure behavior), include 4 point bending tests for timber beams and post-tensioning timber beams, the timber beam size (40 mm x90 mm x1200 mm) the tendon type is threaded rod bar with two colors silver color (9.4 mm) and black color (8.85 mm) . Phase C involved post-tensioned timber concrete composite beam (the concrete layer is 65 mm) Bending Performance. From the test results for the two methods of pre-stressing it is clear that pre-stressing Jacking give higher residual deflection 98.2 % for silver color rod bar against black color with 88.8 % residual deflection. So this type of rod bar (silver color with long nut coupling) is the more efficient in use due to the high contact friction surface area for long coupling nut. The bending tests of post-tensioned beams show the range of bending strength capacity increasing 12 % to 46 %. There were three types of failure is tensile, compression and splitting shear failure depending on the timber quality (degree of natural effect). The PT-TCC give bending strength increment according to degree of composite action 500 mm, 100 mm and 70 mm screw spacing is 68 %, 85 % and 150 % respectively. The empirical Equation derived for bending moment capacity for post-tensioning Kempas timber beam is basically depended on the experimental relation between vertical deflection and rod bar strain. The empirical Equation shows 3.06 kN

constant difference for the theoretical equation and the equation form is $PT_{empirical} = \frac{8EI\Delta}{eL^2} - 3.06$. In the case of PT-TCC the proposed equation depend on the concept of connector slip modulus. A three paths of estimation were discussed depend on push out test slip modulus. The gamma design method show that the slip modulus of pushout test gives a proper estimation in the case of PT-TCC specimens bending strength design.

ABSTRAK

Post-tensioning adalah kaedah pengukuhan (menguatkan) konkrit atau bahan lain dengan helai atau bar keluli kekuatan tinggi. Jenis kayu yang digunakan dalam semua penyelidikan adalah produk kayu kejuruteraan kayu laminasi laminasi (LVL) dan kayu berlapis-laminasi (Glu-Lam), sambil mengelakkan menggunakan kayu semulajadi kerana pelbagai sifatnya. Penambahbaikan yang lebih baru menggunakan teknik pasca-tegangan untuk produk kayu kejuruteraan. Bahan utama dalam kajian ini adalah kayu Kempas semula jadi. Objektif utama penyelidikan ini adalah untuk membangunkan satu kaedah rasuk balak kayu dan persamaan empirikal yang dicadangkan untuk meramalkan kapasiti momen lentur bagi rasuk balak kayu dan rasuk konkrit balak melalui sifat kekuatan lenturan. Program eksperimen dibahagikan kepada 3 fasa utama sepanjang tempoh kajian ini. Fasa A iaito kaedah pasca tegangan menuinjukkan dua kaedah "ketinggian terpaksa membongkok (PT-B) dan pra tekai abicu (PT-J). Tahap B, menunjukkan prestasi lenturan kayu dari segi (kekuatan dan kegagalan tingkah laku), termasuk 4 titik ujian lenturan untuk balak kayu dan bataiy bar pasca tegangan, saiz balak kayu (40 mm x 90 mm x 1200 mm) tendon maralur dengan warma perak (9.4 mm) dan warna hitam (8.85 mm). Fasa C melibatkan balak komposit konkrit pasir bertentangan (lapisan konkrit adalah 65 mm) prestasi lenturan. Dari hasil ujian untuk kedua-dua kaedah pra-menekankan, bahawa pra menekanan bicu memberikan pesongan sisa yang lebih tinggi 98.2 % untuk bar njukkan rod warna perak beterfens warna hitam dengan pesongan sisa 88.8 %. Oleh itu, bar rod jenis ini (warna perak dengan gandinganskru panjang) adalah lebih cekap digunakan kerana kawasan permukaan geseran tinggi untuk ikatan sku yang panjang. Ujian lenturan rasuk pasca-tegangan menunjukkan julat kapasiti kekuatan lenturan yang meningkat 12 % hingga 46 %. Terdapat tiga jenis kegagalan ialah tegangan, pemampatan dan pemisahan kegagalan ricih bergantung kepada kualiti kayu (darjah kesan semula jadi). PT-TCC memberikan kenaikan kekuatan lenturan mengikut tahap tindakan komposit 500 mm, 100 mm dan 70 mm masing-masing adalah 68 %, 85 % dan 150 %. Persamaan empirical yang diperolehi untuk kapasiti momen lentur untuk balak kayu Kempas pasca-tegangan pada dasarnya bergantung pada hubungan eksperimen antara tegangan menegak dan tegangan bar.

Persamaan empirikal menunjukkan perbezaan persamaan 3.06 kN untuk persamaan teoretikal dan bentuk persamaan adalah $PT_{empirical} = \frac{8EI\Delta}{eL^2} - 3.06$. Dalam kes PT-TCC persamaan yang dicadangkan bergantung kepada konsep modulus slip penyambung. Tiga anggaran telah dibincangkan bergantung kepada modulus slip. Kaedah rekabentuk gamma menunjukkan bahawa ujian modulus slip memberikan anggaran yang tepat dalam kes reka bentuk kekuatan lenturan PT-TCC.

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LIST OF SYMBOLS AND ABBREVIATIONS

T	Kempas timber beam
PT	Post-tensioned
TCC	Timber Concrete Composite
PT-TCC	Post-tensioned timber concrete composite T-section beam
ULS	Ultimate Limit State
SLS	Serviceability Limit State
S	Silver color threaded rod bar
B	Black color threaded rod bar
PT-PJ	Post-tensioned timber beam using pre-stressing jacking method
PT-BJ	Post-tensioned timber beam using bending jacking method
LVL	Laminated veneer lumber
MOE	Modulus of elasticity
GLT	Glued Laminated Timber
γ-method	Gamma method (design method)
$V_{0.4,mod}$	Modified initial slip
Fm	Estimated peak load for push out test
DOE	Concrete trial mix concrete design
EI	Full composite action is calculated from gamma method and for non-composite action from Equation(EI = EItimber + EIconcrete)
F_{max}	Max load
LVDT	Linear Variable Differential Transducer
Γ	Is partial factor for material properties, also accounting for modal uncertainties and dimensional variations
B	Breadth of beam
H	Depth of the beam
Z	Section modulus of beam about the y-y axis
$f_{v.k}$	Characteristic shear strength
$f_{c.90.k}$	Characteristic bearing strength

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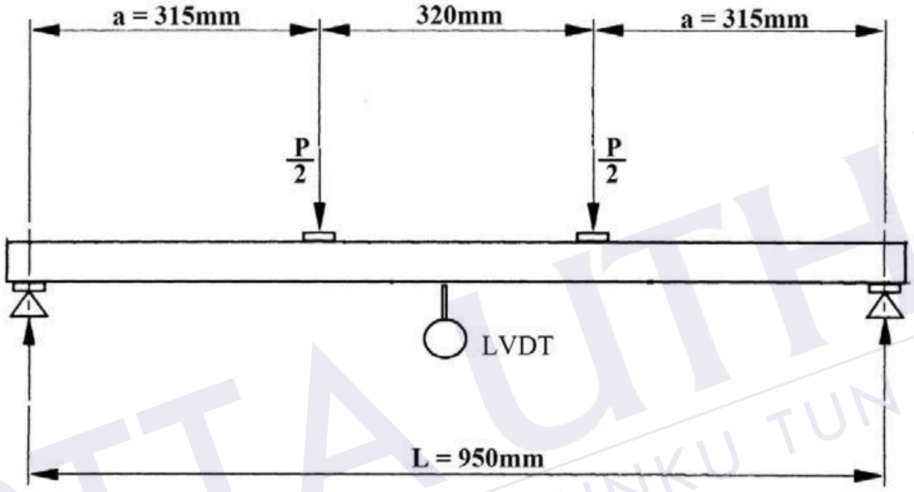
APPENDIX A

Calculation of Young's modulus of timber

The result of specimen Timber (T) had been used as an example to show the calculation of Young's modulus of timber and the test arrangement is depicted on the Figure below.



a=315 mm, L=950 mm, I= bh3/12 =2.43 x 106 mm^4 Emg=

$$E_{m.g} = \frac{3al^2 - 4a^3}{2bh^3\left(2\frac{w2-w1}{F2-F1} - \frac{6a}{5Gbh}\right)}$$

Where M bending moment. P maximum point load. Z section modulus= 54000 mm^3. b width of cross section= 40 mm . h height of cross section = 90 mm . a distance between support and applied point load= 320 mm. $E_{m.g}$ = global modulus of elasticity *F2-F1* = increment of load on the regression line with correlation coefficient of 0.99 (N)

W2-W1 = the increment of deflection corresponding to (*F2-F1*) (mm) G = shear modulus = 650 N/ mm2 according to BS EN 408 2010 clause 10.3 L= length of test sample =950 mm.

Table (D) Calculation Modulus of Elasticity for Single timber

Single Timber	a(mm)	b(mm)	h^3 (mm)	I	L^2 (mm)	F2 (N)	F1 (N)	W2 (mm)	W1(mm)	Em,g N/ mm2
Sample 1	305	40	729000	2430000	902500	9738	6492	5.25	3.3	11686.9
Sample 2	305	40	729000	2430000	902500	9849	6566	5	3.2	12991.1
Sample 3	305	40	729000	2430000	902500	9375	6250	5	3.2	12267.7
Sample 4	305	40	729000	2430000	902500	8796	5864	4.6	3	13062.7
Sample 5	305	40	729000	2430000	902500	12661	6795	8	4	10509.8
Average										12103.6

APPENDIX B

G35 concrete material requirement calculation

Target characteristic strength of concrete: 35MPa Density of concrete : 2380kg/m3

Water/ cement ratio : 0.4

Superplasticizer : 1 %

Calculation for the production of 1m3 concrete

Mass of 1m3 concrete = 2380kg

Mass of cement = 441kg

Mass of water = mass of cement x w/c ratio

= 441 x 0.4

= 176.5 kg

Mass of aggregate = mass of concrete - mass of cement - mass of water

= 2380 - 441 - 176.5

= 1762 kg

Mass of fine aggregate = 1762 x 0.4

= 705kg

Mass of coarse aggregate = 1762 - 705

= 1057 kg

Volume of concrete needed = Volume of [Specimen for Test A + Test B + 6 cubes]

$= (8.32 + 62.4 + 20.25) \times 10^6 = 0.091\ mm^3$

Batching of concrete with volume of 0.091m3

Mass of cement required = 441 x 0.091 = 40.13kg

Mass of Fine aggregate required = 705 x 0.091 = 64.15kg

Mass of Coarse aggregate required = 1057 x 0.091 = 96.18kg

Mass of water required = 176 x 0.091 = 16.016kg

10 % extra mass of the material provided considering loss due to wastage,

Mass of cement = 40.13 x 1.1 = 44.14kg

Mass of Fine aggregate = 64.15 x 1.1 = 70.56kg

Mass of coarse aggregate = 96.18 x 1.1 = 105.79kg

Mass of water = 16.016 x 1.1 = 17.62kg

Table (A) Concrete mix design (DOE) for 1 m^3

Cement	Fine aggregate	Coarse aggregate	Water
441 kg **441/441= 1**	705 kg 705/441=1.59	1057 kg 1057/441=2.39	176 kg 176/441=0.4

APPENDIX C

<u>Determination of strength and stiffness of shear connector Strength of screw connector for single Timber</u>

For 2 screw connectors, Strength = F_{max} = 13.42 kN;

For 1 screw connector, Strength = (1/2) x (13.42) = 6.71 kN.

<u>Stiffness of screw connector for single Timber</u>

SLS - Serviceability Limit State (K_s)

0.4 F_{max} = 0.4 x 13.42 = 5.37 kN; $\Delta_{0.4Fmax}$ = 1.75 mm.

$\Delta_{0.4Fmax}$ mean the displacement at 40 % from F_{max}

$$K_s = \frac{0.4\ \text{Fmax}}{\Delta 0.4\text{Fmax}} = 3.11\ \text{kN/ mm}$$

ULS - Ultimate Limit State (K_u)

0.6 F_{max} = 0.6 x 13.42 = 8.05 kN; $\Delta_{0.6Fmax}$ = 3.66 mm.

$\Delta_{0.6Fmax}$ mean the displacement at 60 % from F_{max}

$$Ku = \frac{0.6\ \text{Fmax}}{\Delta 0.6\text{Fmax}} = 2.737 \text{ kN/ mm}$$

APPENDIX D

Calculation the spacing design for single Kampas timber beam with length 1.2 m

The design procedure of TCC deck was adopted from (CEN,1995).The workflow for design TCC deck as adopted in Figure 5.5. Flow diagram of the design process for prefabricated TCC

Design of timber concrete

Short-term verifications:
Find $EI_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 E_1 A_1 a_1^2 + \gamma_2 E_2 A_2 a_2^2$ From Figure 5.4, K(from test) or $K_{equivalent}=K_{max}\,(1-e^{-n\beta})^{\alpha} + 2$ (new Equation)

Timber strength demand , $\sigma 2(x) = \frac{E_2 A_2\ M(x)}{EI_{ef}}$, $\sigma\ m,2(x) = 0.5\ \frac{E_2 h_2\ M(x)}{EI_{ef}}$

Bending and tension in timber: $\frac{\sigma 2(x)}{ft,0,d} + \frac{\sigma m,2(x)}{fm,d} \leq 1$

Shear in timber: $\tau_{d,max} = 1.5\ \frac{Vd}{A2} \leq f_{vd}$

Concrete strength demand, $\sigma 1(x) = \frac{\gamma 1\ E_1 a_1\ M(x)}{EI_{ef}}$, $\sigma\ m,1(x) = 0.5\ \frac{E_1 h_1\ M(x)}{EI_{ef}}$

$\sigma_{c,tot}=\left|-\sigma_1 - \sigma_{m,1}\right|$ upper stress $\leq$ fcd , $\sigma_{c,tot}=\left|-\sigma_1 + \sigma_{m,1}\right|$ lower stress $\leq$ fct (ok)

Connection strength demand, $F_{(x=0)} = \frac{\gamma 1\, E_1 A_1\, a_1\, Smin}{EI_{ef}} V_{max} \leq F_d$

The process that used to determine the shear connectors spacing is the trial and error this due to the variables in design Equations is a dependent variables. For that it will start with low load and check the connector capacity then increase the load to estimate the highest load before connection failure. Here the calculation present the last load before failure.

The span length used is 1.2 m, the concrete width is 0.12 m , 0.04 is the timber width and 0.09 m is depth of timber and 0.065 m is depth of concrete

E_1 = 34 GPa ,E_2 = 12.103 GPa

E_1= Young's modulus of Elasticity of the concrete

E_2= Young's modulus of Elasticity of the timber

Ku =2.737 kN/ mm, fetched from screw type push out test result

The shear strength in screws= 6.71 KN

<u>The load calculation</u>

W = 1.35 G+ 1.5 Q

G= dead load = 6.21 kN/m

Q= Live load= 0.36 kN/m

W= 8.92 kN/m

M= W*L^2 /8 = 1.60 kN.m

Vd = WL/2 = 5.35 kN

A_1= area of concrete = 65* 120= 7800 mm^2, A_2= area of timber = 3600 mm^2

Max load (Rm) KN for Screw	Max load for single Screw (Rm) KN	Ks(Screw) kN/ mm	Ku (Screw) kN/ mm
13.42	6.71	3.11	**2.737**

Ass um e

the spacing S=100 mm

$$\gamma_1 = \frac{1}{1+\pi^2 E_1 A_1 sef/Kl^2}$$

$$\gamma_1 = \frac{1}{1+3.14^2 * 34000 \quad *7800*100/(2740*1200*1200)} = 0.015$$

γ2= 1

a_1 distance

$$a_1 = \frac{\gamma 2 E_2 A_2 H}{\gamma 1 E_1 A_1 + \gamma 2 E_2 A_2}$$

$$a_1 = \frac{1*12103.6*3600*77.5}{0.015*34000*7800 + 1*12103.6*3600} = 71.26 \text{ mm}$$

$$a_2 = \frac{\gamma 1 E_1 A_1 H}{\gamma 1 E_1 A_1 + \gamma 2 E_2 A_2}$$

$$\frac{0.015*34000*7800*77.5}{0.015*34000*7800 + 1*12103.6*3600} = 6.238 \text{ mm}$$

$$EI_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 E_1 A_1 a_1^2 + \gamma_2 E_2 A_2 a_2^2$$

= 34000*18308333.33+12103.6*48600000+0.015*34000*7800*71.26 $^{\wedge 2}$

+1*12103.6*3600*6.238 $^{\wedge 2}$ = 1.415E+11 N/ mm^2

Fn = 4.129 kN less than shear strength of screw = 6.71 kN, it is OK

Fn= Shear strength from Equation of design cod

The spacing S = 100 which was assumed it OK

However, took this value S=100 for TCC with 1200 mm length

APPENDIX E

TCC design according to EC5 for timber beam 1.2 m

The length of span is 1.2 m, 0.12 m width of concrete with 0.065 m depth, the length of timber 1.2 m , the cross section of unite timber 0.04 width of timber and 0.09 m depth of timber.

L=1.2 m, h_1(concrete)=0.065 m, h_2(timber)=0.09 m, width (w) for concrete = 0.12m, w for timber=0.04 m

Imposed load = 3 kN/m^2

Permanent load (self-weight) = 0.21 kN/m

Permanent uniform load = 6 kN/m

Total permanent load, G = 0.21 + 6 = 6.21 kN/m

Total imposed load, Q = 3× 0.12 = 0.36 kN/m

ULS short-term load combinations,

for uniformly distributed load, w = 1.35G + 1.5Q = 8.93 kN/m

design bending moment, Md = wL2/8 = 1.61 kNm

design shear force, Vd = wL/2 = 5.36 kN

Design data for connector:

The connection slip moduli and strength were determined by experimental push-out test
for screw of 68.7 (L) × 5.47(d) where L and d as the length and diameter in mm, respectively.

For definition of connection spacing):

Connection slip modulus for ULS, K_u = 2.74 kN/ mm

Connection slip modulus for SLS, Ks = 3.11 kN/ mm

Characteristic strength of connection, Fk = 6.71 kN

Maximum spacing of connection, smax = 100 mm

Minimum spacing of connection, s_{min} = 100 mm

Effective spacing of connection, seff = 0.75smin + 0.25smax = 100 mm

Timber Strength Capacity for Kempas timber

Young's modulus of Kempas, E2= 12103.6 MPa; mean value of timber bending stress fm =86.5 N/ mm2 , γm=1.3 is partial factor for material properties Kempas ;and kmod =0.8 is modification factor for medium term load duration. Pm timber density 850 kg/m3 , (CEN,1995)

Timber bending charachteristic calculation:

fk = 60 N/ mm^2 (Kempas timber D60 according MS 544: part 3, 2001)

All the factors used to determine the tensile, compressive, shear stresses are adopted according BS EN 384:2016+A1:2018

Timber design bending strength, fm,d= kmod ×fm,k/γm=0.8×(60) /1.3= 36.92 N/ mm^2

Timber characteristic tensile strength ft,0,k = 0.6x fmk = 0.6x (60) = 36

Timber design tensile strength, ft,0,d= kmod ×ft,0,k/γm=0.8×36/1.3= 22.15 N/ mm^2

Timber shear characteristic strength, $f_{v,k}$ = 4.8 N/ mm^2 (If $f_{m.k} \leq 60$, the $f_{v.k}$= 3+ 0.03* $f_{m.k}$ = 4.8 N/ mm^2

Timber shear design strength, fv,d=kmod ×fv,k/γm=0.8× 4.8/1.3 =2.95 N/ mm2

Timber compression parallel to grain characteristic strength $f_{c,0,k} = 4.3x\ f_{mk}^{0.5}$ $=4.3*60^{0.5}$= 33.31 N/ mm^2

Timber compression parallel to grain design strength,$f_{c,0,d}=k_{mod}\times f_{c,0,k}/\gamma_m$ =0.8×33.31 /1.3 =20.5 N/ mm^2

Concrete Strength Capacity for grade 35

Young’s modulus of concrete, E1=31494 MPa

Concrete characteristic compressive strength, fck = 35 N/ mm2

Concrete design compressive strength, fcd =fck /γc=35/1.5 =23.33 N/ mm2

Concrete characteristic tensile strength, fctk= 2.2 N/ mm2 (EN 1992-1-1:2004, Table 3.1 section 3.1.3)

Concrete design tensile strength, fctd=fctk/γc=2.2/1.5=1.47 N/ mm2

where γc= 1.5 is partial factor for concrete at ultimate limit state(Eurocode, 2 Part 1–1).

Connection Strength Capacity for Screw

Characteristic strength of connection, Fk=6.71 kN

Design strength of connection, Fd=kmod ×Fk/γm=0.8×6.71 /1.3=4.13 kN

where γm=1.3 is partial factor for material properties TCC and kmod =0.8 is modification factor for permenant load duration and moisture content in Service Class 1 (Eurocode5, Part 1–1)

Verifications for Ultimate Limit State in the Short Term

This verification is carried out for the load condition with uniformly distributed imposed and permanent loads; w=1.35G+1.5Q= 8.93 kN/m.

Bending Stiffness Properties for Ultimate Limit State Short-Term Verifications are as follows. Concrete ga mma coefficient,

Area of concrete (A1)= 7800 mm^2

Area of Timber (A2)= 3600 mm^2

E_1 = 31.494 GPa ,E_2 = 12.103 GPa

E_1= Young's modulus of Elasticity of the concrete

E_2= Young's modulus of Elasticity of the timber

$$\gamma 1 = \frac{1}{1+\pi^2 E_1 A_1 sef/Kl^2}$$

$$\gamma 1 = \frac{1}{1+3.14^2 * 31494 \quad *7800*100/(2740*1200*1200)} = 0.015$$

$\gamma 2 = 1$

a_1 distance

$$a_1 = \frac{\gamma 2 E_2 A_2 H}{\gamma 1 E_1 A_1 + \gamma 2 E_2 A_2}$$

$$a_1 = \frac{1*12103.6*7800*77.5}{0.015*31494*7800 + 1*12103.6*3600} = 71.26 \text{ mm}$$

$$a_2 = \frac{\gamma 1 E_1 A_1 H}{\gamma 1 E_1 A_1 + \gamma 2 E_2 A_2}$$

$$\frac{0.015*31494*7800*77.5}{0.015*31494*7800 + 1*12103.6*3600} = 6.238 \text{ mm}$$

$$EI_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 E_1 A_1 a_1^2 + \gamma_2 E_2 A_2 a_2^2$$

= 31494*18308333.33+12103.6*48600000+0.015*31494*7800*71.26 ^^{2}

+1*12103.6*3600*6.238 ^^{2} = 1.415E+11 N/ mm^2

where $I_1 = b_1 h^3{}_1/12$ and $I_2 = b_2 h^3{}_2/12$

Timber Strength Demand

Timber axial stress due to axial force

$$\sigma_2(x) = \frac{E_2 a_2 \, M(x)}{EI_{ef}}$$

$$\frac{12103.6*6.24*1.61*1000000}{1.42E+12} = 0.89 \text{ N/ mm}^2$$

Timber axial stress due to bending moment,

$$\sigma\, m,_2(x) = 0.5 \, \frac{E_2 h_2 \, M(x)}{EI_{ef}}$$

$$0.5 \, \frac{12103.6*90*1.61*1000000}{1.42E+11} = 6.38 \text{ N/ mm}^2$$

Combined bending and tension ratio

$$\frac{\sigma 2(x)}{ft,0,d} + \frac{\sigma m,2(x)}{fm,d} \text{ less than one}$$

$$\frac{0.89}{22.36} + \frac{6.38}{37.26} = 0.21 \text{ less than one (1) is ok (satisfactory)}$$

Timber shear stress, with the simplified and conservative assumption that only the timber part resists shear:

$\tau d_{max} = 1.5\frac{\mathrm{Vd}}{\mathrm{A2}} = \; 1.5 * \frac{5.36*1000}{3600}$ =2.23 N/ mm^2 less than fvd =2.46 N/ mm^2 (Ok)

Concrete Strength Demand

Concrete axial stress due to axial force,

$\sigma_1(x) = \frac{\gamma 1\ E_1 a_1\ M(x)}{EI_{ef}} = \frac{0.015*31494*71.26\ *1.61*1000000}{1.41E+11} =$ 0.41 N/ mm^2

Concrete axial stress due to bending moment

$\sigma\ m,_1(x) = 0.5\ \frac{E_1 h_1\ M(x)}{EI_{ef}} = 0.5\ \frac{31494*\ 65*1.61*1000000}{1.41E+11}$ = 12.01 N/ mm^2

Concrete total upper fibre stress

$\sigma_{c,tot} = |-\sigma_1 - \sigma_{m,1}| = |-0.41\ - 12.01|$ =12.42 N/ mm^2 upper stress ≤ fcd (23.33) ok

Concrete total lower fibre stress

$\sigma_{c,tot} = |-\sigma_1 + \sigma_{m,1}| = |-0.41 + 12.01| =$ 11.61 N/ mm^2 lower stress tension tensile stress ≥ fct =1.46 not ok require tension reinforcement at bottom of concrete.

Connection Strength Demand

Shear force in connection at maximum shear,

$F_{(x=0)} = \frac{\gamma 1\ E_1 A_1\ a_1\ smin}{EI_{ef}} V_{max} = \frac{0.015*31494*7800\ *71.1*100}{1.40E+11} * 5.36$ *1000

= 4.12 less than 4.13 it is ok

The ultimate limit state force is (ULS) = 8.93 KN/m = 10.716 kN

APPENDIX F

Guidelines for moment capacity for post-tensioned timber beam

1- The timber beam bending capacity is calculated experimentally with COV % not exceeding 10 % (part one).

2- Identify the experimental correlation between the mid-span vertical deflection and the rod bar deformation in the term strain = coefficient x vertical deflection, ϵ = coefficient x Δ.

3- Estimate the post-tensioning force through (PT= Ex ϵx A) and replace strain ϵ by relation in the above point (2).

4- Determine the bending moment resulting from the post-tensioning process PT= Ex (coefficient x Δ) x A x e (part two).

5- The total estimated bending moment is the sum of the two parts (1 and 4).

6- The total estimated bending moment can also be expressed by the Equation PT = ((8EI xΔ)/ (e×L^2)) + coefficient. This coefficient is calculated by the next point.

7- The coefficient is equal to the average value of difference between the theoretical PT value (8EI xΔ)/ (e×L^2) and the experimental PT value, under the condition that (this average x 100/bending average value) is less than 10 %

APPENDIX G

<u>Proposed empirical formula to predict moment strength in PT-TCC beams</u>

A- Method 2

The control value present TCC10 from 4 point bending test data find 0.6Fmax and corresponding relative horizontal slip between timber and concrete o.6H.

Then calculate the factor f_{TCC}

$$f_{TCC} = \frac{0.6\,Fmax}{0.6\,H} = \frac{26.43}{1.4} = 18.88 \quad \text{For TCC10 specimen (control value)}$$

In same manner calculate the factor for example the PT-TCC2.15 specimen

$$f_{PT-TCC\,2.15} = \frac{0.6\,Fmax}{0.6\,H} = \frac{33.44}{1.19} = 28.13 \qquad \text{For PT-TCC2.15 specimen}$$

The second step is to calculate the ratio for between $f_{PT-TCC\,2.15}$ and f_{TCC} and called F factor

$$F = \frac{f_{PT-TCC}}{f_{TCC}} = \frac{28.13}{18.88} = 1.49$$

Then Ku equivalent is equal

$ku_{equivalent} = Rxku_{pushout-test}$ = 1.49 x 2.74 = 4.09 kN/ mm

This value is used instead of normal Ku resulting from pushout test in the design of TCC to estimate bending stress for PT-TCC.

B- Method 3

The centroid of T-section Y is calculated by

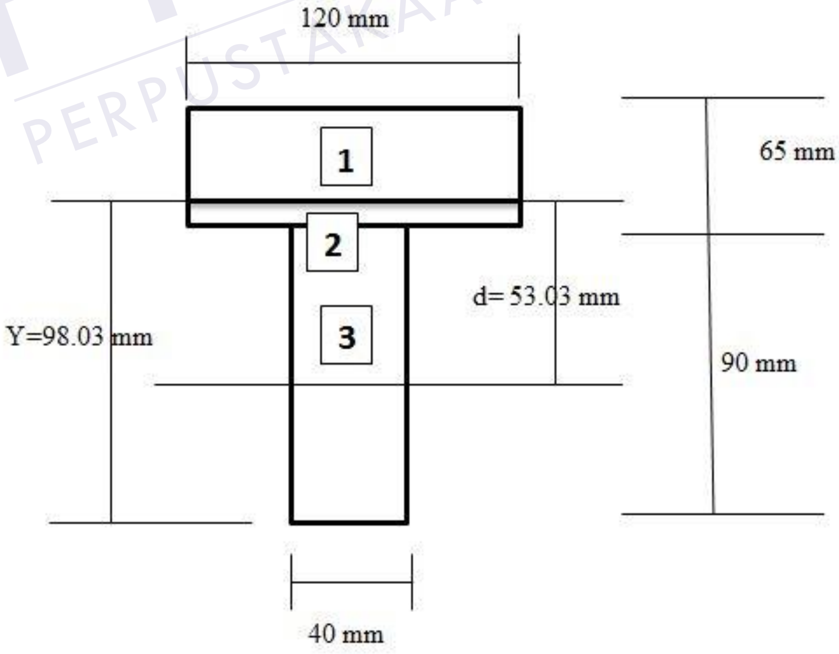
$$Y = \frac{\sum AiYi}{\sum Ai} = \frac{120x65x\left(90+\frac{65}{2}\right)+90x40x(\frac{90}{2})}{120x65+90x40} = 98.03 \text{ mm from bottom web}$$

The moment area for T-section I_x is calculated by

$I_{total} = \sum Ii$

$I_{total} = I_1 + I_2 + I_3\ (Iz + A_3 d) = \ = \frac{120X(155-98.03)^3}{3} + \frac{120X(98.03-90)^3}{3} + \frac{40X90^3}{12} + 40X90X(98.03 - \frac{90}{2}) = 19970592\ \text{mm}^4$

$Z = \frac{I}{Y} = \frac{19970592}{Y98.03} = 203726.8\ \text{mm}^3$



List of publications

- Wissam, M., Yeoh, D., Jalal, M., Abd Ghafar, D., and Heng Boon, K. (2019). Kempas Timber Un-Bonded Post-Tensioning Solution New Approach. *International Journal of Civil Engineering and Technology*, *10*(3).
- Wissm,M., Yeoh, D.,Jalal.,M, Abd Ghafar, D., and Heng Boon, K., Behaviour of Post-Tensioned Kempas Timber Beam with Two Tendon Types ,*The 2nd Global Congress on Construction, Material and Structural Engineering GCoMSE 2019*
- Jalal, M., Wissam,M., Abd Ghafar, D., Yeoh, D., and Heng Boon, K. (2019). Experimental Tests of Nail and Screw Connectors for Timber Concrete Composite Deck.
- Jalal,M, Abd Ghafar, D., Yeoh, D., Wissam.,M, and Heng Boon, K., Vibration Behaviour of Natural Timber and Timber Concrete Composite Deck System ,*The 2nd Global Congress on Construction, Material and Structural Engineering GCoMSE 2019.*