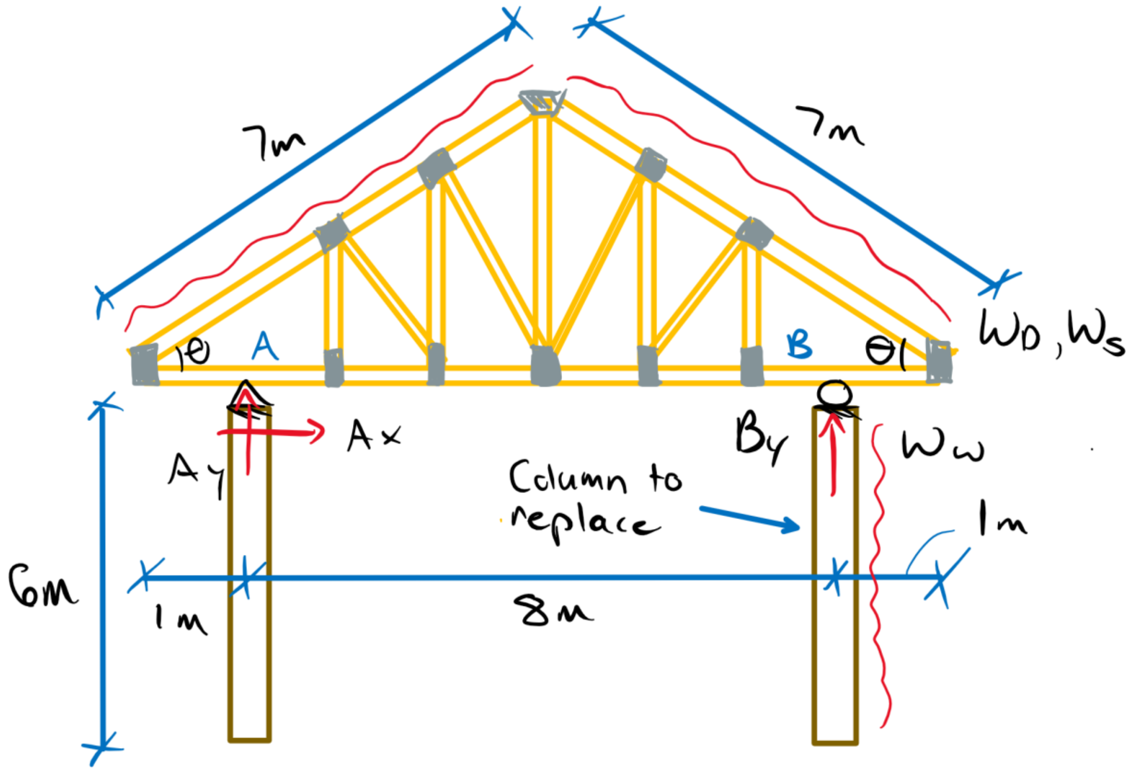
CWC Wood Engineering
Final Exam Solutions

QUESTION 1

The column to be replaced supported the truss in bearing. We can take the global equilibrium of the truss to determine the axial load on the column from the distributed dead and snow loads. We are assuming there is no eccentricity in the axial column load.



w_D = 68.2 kN/m w_s = 28.4 kN/m w_w = 25 kN/m

$\cos\theta = 5/7$
$\theta = 44.4°$

$\Sigma M_A = 0 = -w(7\text{ m})(3.5 - 1 \times 7/5\text{ m}) - w(7\text{ m})(4 + 5/2\text{ m})5/7 + w(7\text{ m})(7/2 \times \sin 44.4\text{ m})\sin 44.4 + 8B_y$
$B_y = 4.4w$
$B_y = 4.4w_s + 4.4w_D$

$P_D = 4.4(68.2\text{ kN/m}) = 300\text{ kN}$
$P_s = 4.4(28.4\text{ kN/m}) = 125\text{ kN}$

The column is in combined compression + bending, therefore we must identify the possible governing load cases.

Load Case 1: 1.4D $K_D = 0.65$

$P_f = 1.4P_D =$ 420 kN
$V_f = 0$
$M_f = 0$

Load Case 3: 1.25D+1.5S+0.4W $K_D = 1.15$

$P_f = 1.25P_D + 1.5P_s =$ 562.5 kN
$w_{w,f} = 0.4w_w = 10$ kN/m
$V_f = wL/2 = 30$ kN $V_f/K_D = 26$ kN
$M_f = wL^2/8 = 112.5$ kNm $M_f/K_D = 98$ kNm

Load Case 4: 1.25D+1.4W+0.5S $K_D = 1.15$

$P_f = 1.25P_D + 0.5P_s =$ 437.5 kN
$w_{w,f} = 1.4w_w = 35$ kN/m
$V_f = wL/2 = 105$ kN $V_f/K_D = 91$ kN **(Governs factored shear)**
$M_f = wL^2/8 = 157.5$ kNm $M_f/K_D = 137$ kNm **(Governs factored moment)**

All cases must be checked for axial compression resistance as well as axial + moment interaction. Additionally, we are specified a deflection limit requirement of L/180. We can determine the minimum bending stiffness to meet this requirement:

Serviceability Case 4: 1.0D + 1.0W + 0.5S (only W contributes to bending)

$\Delta \leq L/180 = 5/384\ wL^4/EI$
$EI \leq wL^4/2560$
$EI \leq (25\ N/mm)(6000\ mm)^4/2560$
$EI \leq 12660 \times 10^9\ Nmm^2$ **(Minimum EI required for deflection limit)**

Trial Section: 365x380 SPF 20f-EX
From Tables:

$M_r = 291$ kNm
$V_r = 175$ kN
$P_r = 1840$ kN
$EI = 17200 \times 10^9\ Nmm^2 > EI_{req}$ **(Deflection OK – 74% utility)**

Shear Capacity (**7.5.7.2**):

$Z = 0.365 \times 0.380 \times 6\ m^3 = 0.83\ m^3 < 2.0\ m^3$
$V_r = \phi F_v 2/3 A_g$
$F_v = f_v K_D K_H K_{sv} K_T = (1.75\ MPa)(1.15)(1)(1)(1) = 2.01$ MPa

$V_r = (0.90)(2.01\ MPa)2/3(365x380\ mm^2)$
$V_r = 167\ kN \geq V_f = 105$ kN **(OK for shear requirement – 63% utility)**

Moment Capacity (**7.5.6.5**):

$K_{zbg} = (130x610x9100/\ 182.5x380x6000)^{0.1} = 1.06 \leq 1.3$

$C_B = (L_e d/b^2)^{1/2} = 4.1 \leq 10$ $K_L = 1.0$ (**7.5.6.4**)

Since $K_{zbg} > K_L$, M_{r2} will be more critical.

Case 1: $K_D = 0.65$

$M_{r2} = \phi F_B S K_X K_L = (0.90)(25.6 \text{ MPa} \times 0.65 \times 1 \times 1 \times 1)(365\times380^2/6 \text{ mm}^3)(1)(1)$
$M_{r2} = 132 \text{ kNm} \geq M_f = 0$ **(Moment OK for Case 1)**

Case 3/4: $K_D = 1.15$

$M_{r2} = (0.90)(25.6 \text{ MPa} \times 1.15 \times 1 \times 1 \times 1)(365\times380^2/6 \text{ mm}^3)(1)(1)$
$M_{r2} = 233 \text{ kNm} \geq M_f = 157.5$ **(Moment OK for Cases 3/4 – 68% utility)**

Axial Compression (**7.5.8**):
For no intermediate bracing, the weak axis will govern the compression resistance:

$C_{cy} = L_e/b = 6000/365 = 16.4 \leq 50$ (slenderness OK)
$K_{zcg} = 0.68(0.83 \text{ m}^3)^{-0.13} = 0.70 \leq 1.0$

$F_c = f_c K_D K_H K_{sc} K_T = (25.2 \text{ MPa}) K_D (1)(1)(1) = 25.2 K_D$

Case 1: $K_D = 0.65$

$F_c = 16.4$ MPa
$K_c = (1.0 + (16.4 \text{ MPa})(0.70)(16.4)^3/35(10300\times0.87 \text{ MPa})(1)(1)\ )^{-1}$
$K_c = 0.86$

$P_r = \phi F_C A K_{zcg} K_C = (0.80)(16.4 \text{ MPa})(365\times380 \text{ mm}^2)(0.70)(0.86)$
$P_r = 1095 \text{ kN} \geq P_f = 420 \text{ kN}$ **(Axial OK for Case 1)**

Case 3/4: $K_D = 1.15$

$F_c = 29.0$ MPa
$K_c = (1.0 + (29.0 \text{ MPa})(0.70)(16.4)^3/35(10300\times0.87 \text{ MPa})(1)(1)\ )^{-1}$
$K_c = 0.78$

$P_r = (0.80)(29.0 \text{ MPa})(365\times380 \text{ mm}^2)(0.70)(0.78)$
$P_r = 1757 \text{ kN} \geq P_f = 565.2 \text{ kN}$ **(Axial OK for Cases 3/4)**

Combined Axial + Bending (**7.5.12**):

$P_E = \pi^2 E_{05} K_{SE} K_T I/L_e^2 = \pi^2(0.87\times10300 \text{ MPa})(1)(1)(365\times456^3/12 \text{ mm}^4)/(6000 \text{ mm})^2$
$P_E = 7085$ kN

Case 1: $K_D = 0.65$

No interaction

Case 3: $K_D = 1.15$

$(P_f/P_r)^2 + (M_f/M_r)(1/1\text{-}P_f/P_E) \leq 1.0$
$(562.5/1757)^2 + (112.5/233)(1/1\text{-}(562.5/7085) \leq 1.0$

$0.63 \leq 1.0$ **(Section OK for Case 3 Interaction)**

Case 4: $K_D = 1.15$

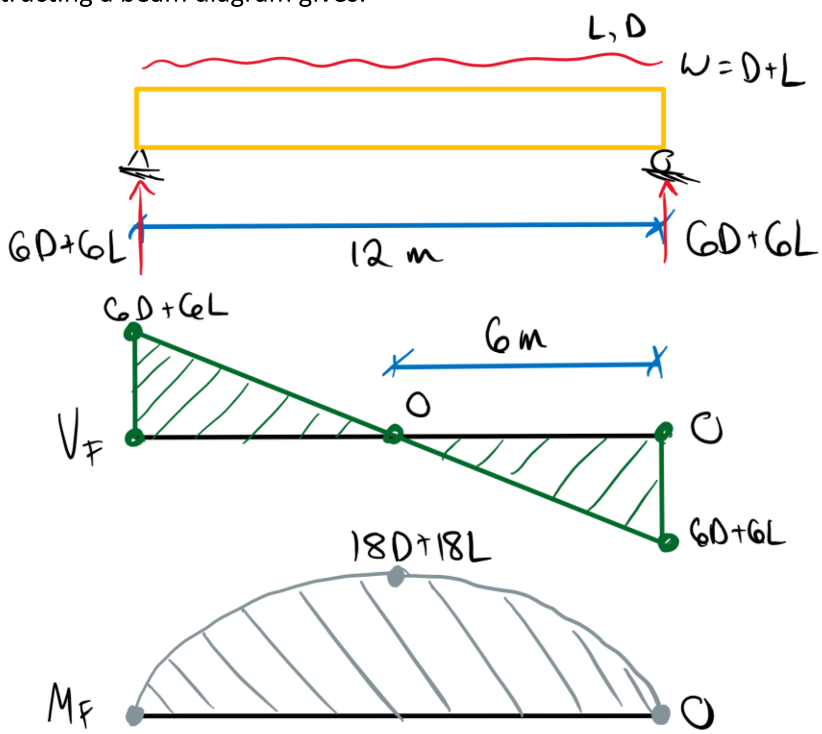
$(437.5/1757)^2 + (157.5/233)(1/1-(437.5/7085) \leq 1.0$

$0.78 \leq 1.0$ **(Section OK for Case 3 Interaction – utility > 75%)**

Therefore, a 365x380 mm SPF 20f-EX member is suitable and achieved a utility greater than 75%.

QUESTION 2

a) This question requires the determination of the critical distributed load for the 365x1254 mm D. Fir-L 24f-E member, given that the live load is 80% the dead load and that the beam is required to have 30 minute FRR. Constructing a beam diagram gives:



V_f = 6D + 6L
W_f = 12D + 12L
M_f = 18D + 18L

L = 0.8D

Load Case 1: 1.4D K_D = 0.65

W_f = 1.4(12D) = 16.8D W_f/K_D = 25.8D
M_f = 1.4(18D) = 25.2D M_f/K_D = 38.8D

Load Case 2: 1.25D +1.5L K_D = 1 – 0.5log(D/0.8D) = 0.95

W_f = 29.4D W_f/K_D = 30.9D **(Governs Factored Shear)**
M_f = 44.1D M_f/K_D = 46.4D **(Governs Factored Moment)**

Fire Load Case: 1.0D + 1.0L K_D = 1.15 (per **B.3.3**)
Assuming **B.2.2** is met.

W_f = 21.6D **(Must check independently of cases above)**
M_f = 32.4D **(Must check independently of cases above)**

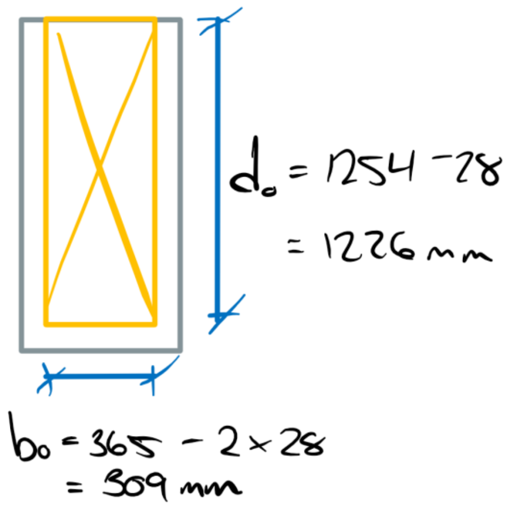
$K_{fi} = 1.35$ (**B.3.9** for glulam)

For charring and zero-strength layer determination:

$B_n = 0.7$ mm/min (**B.4.2** for multiple exposed sides)
$x_t = 7$ mm (**B.5.1** for FRR > 20 min)

$x_n = \beta_n t + x_t = (0.7\text{ mm/min})(30\text{ min}) + 7\text{ mm} = 28\text{ mm}$ (**B.4.4**)

The reduced cross-section will have the following dimensions (assuming the top of beam is protected from fire).



Shear Resistance (**7.5.7**):

$Z = 0.365 \times 1.254 \times 12 = 5.5\text{ m}^3 \geq 2.0\text{ m}^3$
$Z_o = 0.309 \times 1.226 \times 12 = 4.5\text{ m}^3 \geq 2.0\text{ m}^3$

Must apply **7.5.7.2a** for all cases.

$F_v = f_v K_D K_H K_{sv} K_T = 2.0\text{ MPa} \times K_D(1)(1)(1) = 2K_D$

Calculating C_v (**7.5.7.5**):

$r^* = 0$ $C_v = 3.69$ (**Table 7.5.7.5**)

Case 2:

$W_r = \phi F_v 0.48 A_g C_v Z^{-0.18} = (0.90)(2\text{ MPa} \times 0.95)0.48(365\text{x}1254\text{ mm}^2)(3.69)(5.5\text{ m}^3)^{-0.18}$
$W_r = 1020\text{ kN} \geq W_f = 29.4D$
$D \leq 34.7$ kN/m **(GOVERNS Critical deadload for beam shear)**

Fire:

$W_r = \phi F_v 0.48 A_g C_v Z^{-0.18} K_{fi} = (1.0)(2\text{ MPa} \times 1.15)0.48(309\text{x}1226\text{ mm}^2)(3.69)(4.5\text{ m}^3)^{-0.18}(1.35)$
$W_r = 1589\text{ kN} \geq W_f = 21.6D$
$D \leq 73.6$ kN/m **(Critical deadload for beam shear in fire)**

Moment Resistance (**7.5.6.5**):

The size factor is calculated with the original beam dimensions for all cases.

K_{zbg} = (130x610x9100/182.5x1254x12000)$^{0.1}$ = 0.87 ≤ 1.3

Case 2:

C_B = ((12000 mm)(1254 mm)/(365 mm)^2)$^{1/2}$ = 10.6 > 10 (**7.5.6.4**)
F_b = (30.6 MPa)$K_DK_HK_{sb}K_T$ = 30.6 MPa x (0.95)(1)(1)(1) = 29.1 MPa

C_k = (0.97E$K_{SE}K_T/F_b$)$^{1/2}$ = ((0.97)(12800)(1)(1)/(29.1))$^{1/2}$ = 20.6

K_L = 1 – 1/3(C_B/C_k)^4 = 1 – 1/3(10.6/20.6)^4 = 0.98

Since $K_{zbg} < K_L$, M_{r1} will be more critical.

M_{r1} = ϕ$F_bSK_xK_{zbg}$ = (0.90)(29.1 MPa)(365x1254^2/6 mm^3)(1)(0.87)
M_{r1} = 2180 kNm ≥ M_f = 44.1D
D ≤ 49.4 kN/m **(Critical deadload for beam moment resistance)**

Fire:

C_B = ((12000 mm)(1226 mm)/(309 mm)^2)$^{1/2}$ = 12.4 > 10 (**7.5.6.4**)
F_b = (30.6 MPa)$K_DK_HK_{sb}K_T$ = 30.6 MPa x (1.15)(1)(1)(1) = 35.2 MPa

C_k = (0.97E$K_{SE}K_T/F_b$)$^{1/2}$ = ((0.97)(12800)(1)(1)/(35.2))$^{1/2}$ = 18.8

K_L = 1 – 1/3(C_B/C_k)^4 = 1 – 1/3(12.4/18.8)^4 = 0.94

Since $K_{zbg} < K_L$, M_{r1} will be more critical.

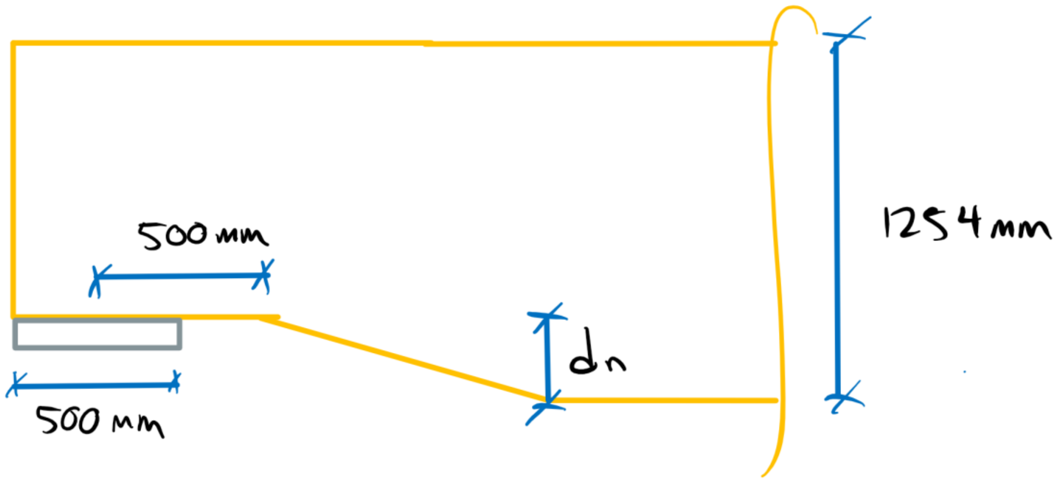
M_{r1} = ϕ$F_bSK_xK_{zbg}K_{fi}$ = (1)(35.2 MPa)(309x1226^2/6 mm^3)(1)(0.87)(1.35)
M_{r1} = 3200 kNm ≥ M_f = 32.4D
D ≤ 98.8 kN/m **(Critical deadload for beam moment resistance during fire)**

Therefore, based on the moment and shear capacity of the beam and the 30 minute FRR, the maximum specified dead load that can be applied is 34.7 kN/m.

b) The goal of Annex B is to predict the results of the standard fire resistance test CAN/ULC-S101 for a timber member. Therefore, the use of a specified loading and mean strength values (instead of specified strengths) is done to predict the actual performance of a member in the standard fire test which results in apparent strengths greater than those calculated through the methods in main body of O86-14.

c) This part of the question requires the determination of the maximum notch depth required to support a factored load of 55 kN/m assuming a 500 mm long support and a notch length of 500 mm for the same beam referred to in part A (now ignoring fire resistance considerations). Assume K_D = 1.0.

V_f = $w_fL/2$ = (55 kN/m)(12 m)/2
V_f = 330 kN



<u>Tension Side Notch Shear Resistance (**7.5.7.4.2**):</u>

$F_r = \phi F_f A_g K_N$

$f_f = 2.5(365/2\ mm)^{-0.2} = 0.88 \leq 0.90$
$f_f = 0.90$ MPa

$F_f = f_f K_D K_H K_s K_T = 0.90$ MPa x 1 x 1 x 1 x 1 = 0.90 MPa

We must solve for the notch depth, d_n, by determining the critical K_N for e = 500 mm.

$F_r = (0.90)(0.90\ MPa)(1254 \times 365\ mm^2)K_N \geq V_f = 330$ kN
$K_N \geq 0.89$

The equation given in **7.5.7.4.2** is complicated, so let's interpolate with **Table 6.5.5.3.2:**

Interpolating with:

$K_N d^{1/2} = 31.5$ (this is the minimum acceptable value of $K_N d^{1/2}$)
$\eta = e/d = 0.40$

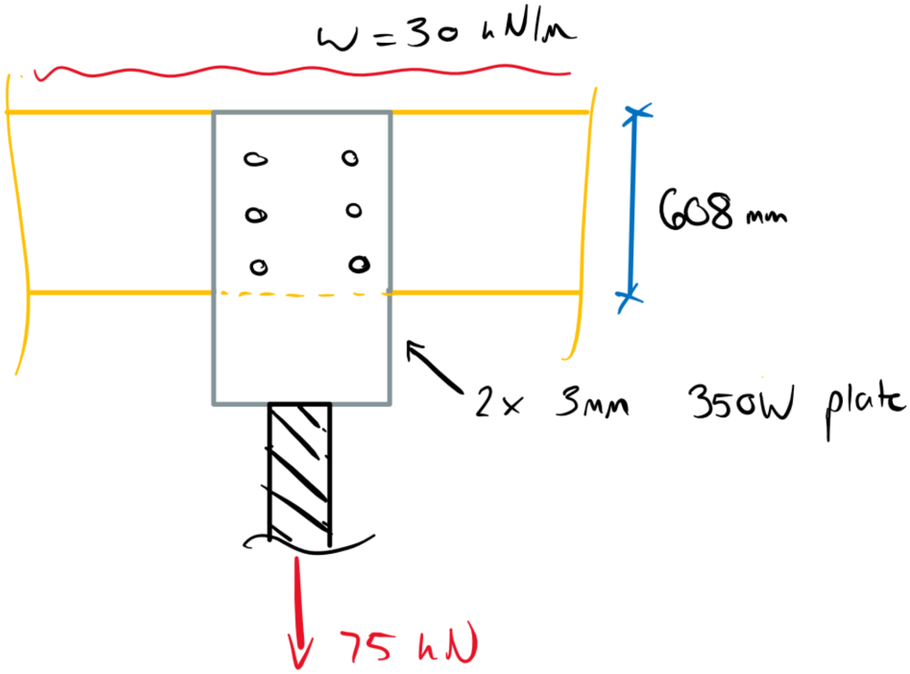
$\alpha - 0.95 / 0.90 - 0.95 = 31.5 - 38.8 / 26.5 - 38.8$
$\alpha = 0.92$ (Since α and K_N are directly proportional, this is the minimum acceptable α)

$\alpha \geq 0.92$
$1 - d_n/d \geq 0.92$
$d_n \leq 0.08(1254\ mm)$
$d_n \leq 100$ mm **(Maximum allowable notch depth)**

Therefore, the maximum allowable notch depth for a notch length of 500 mm is 100 mm based on the applied load.

QUESTION 3

This question requires the design of a bolt connection to a 5 m long, simply supported 265x608 SPF 20f-E beam under wet service conditions. The beam and connection were fabricated when the timber was dry. The question specifies the use of 1/2" ASTM A307 bolts and two 3 mm 350W steel plates to create a 3 member connection. The beam also supports a 30 kN/m distributed load. The load duration factor is assumed to be 1.0. The factored shear force can be calculated as follows:



$V_f = wL/2 + P_f/2 = (30\ kN/m)(5\ m)/2 + (75\ kN)/2 = 112.5\ kN$

From the beam selection tables, the beam shear resistance is:

$V_r = 169\ kN$

We can determine the critical loaded edge distance based on the reduction of shear capacity:

$V_{re} = V_r \times K_{sv} \times d_e/d \geq 112.5\ kN$
$d_e \leq (112.5\ kN)(608\ mm)/(169\ kN)(0.85)$
$d_e \leq 476\ mm$
$d_e \sim 480\ mm$ (Required effective shear depth)

$d_{bolt} = 0.5 \times 25.4\ mm = 12.7\ mm$ (bolt diameter)
$d_{hole} = d_{bolt} + 2\ mm = 14.7\ mm$ (bolt hole diameter)

Therefore, the loaded edge distance can be:

$e_p = 608 – 480 + 14.7/2 = 135\ mm$

Perpendicular to Grain Fastener Spacing Requirements (**12.4.3.2**):

a) $S_r \geq 3d_{bolt} = 38.1$ mm
b) $S_c \geq 3d_{bolt} = 38.1$ mm
c) $e_Q \geq 4d_{bolt} = 50.8$ mm
d) $e_p \geq 1.5d_{bolt} = 19.05$ mm

Bolt Yielding (**12.4.4.3**):
We can determine the minimum number of bolts required to resist fastener yielding as a starting point.

$N_r = \phi_y n_u n_s n_f$
$K_{sf} = 0.67$ (**Table 12.2.1.6**)

$f_1 = K_{sp}\phi_{steel}/\phi_y f_u = (3.0)(0.80/0.80)(450\text{ MPa}) = 1350\text{ MPa}$ (350W steel plates)
$t_1 = 3$ mm
$f_2 = f_{iQ} = 22G(1\text{-}0.01d_f)K_{sf} = 22(0.42)(1\text{-}0.01(12.7))(0.67) = 5.4\text{ MPa}$ (G = 0.42 for SPF)
$t_2 = 265$ mm
$f_y = 310$ MPa (ASTM A307 bolts)

a) n_u =51.4 kN
c) n_u =9.1 kN
d) n_u =13.0 kN
g) n_u =5.4 kN **(GOVERNS)**

$N_r = (0.80)(5.4\text{ kN})(2\text{ shear planes})n_f \geq N_f = 75\text{ kN}$
$n_f \leq 8.7$ bolts (Min. number of bolts to resist yielding)

Let's try 9 bolts with the following spacing:

$e_p = 135$ mm $e_Q = 75$ mm $S_c = 75$ mm $S_R = (608 - 135 - 75)/2 = 199$ mm
$d_e = 480$ mm

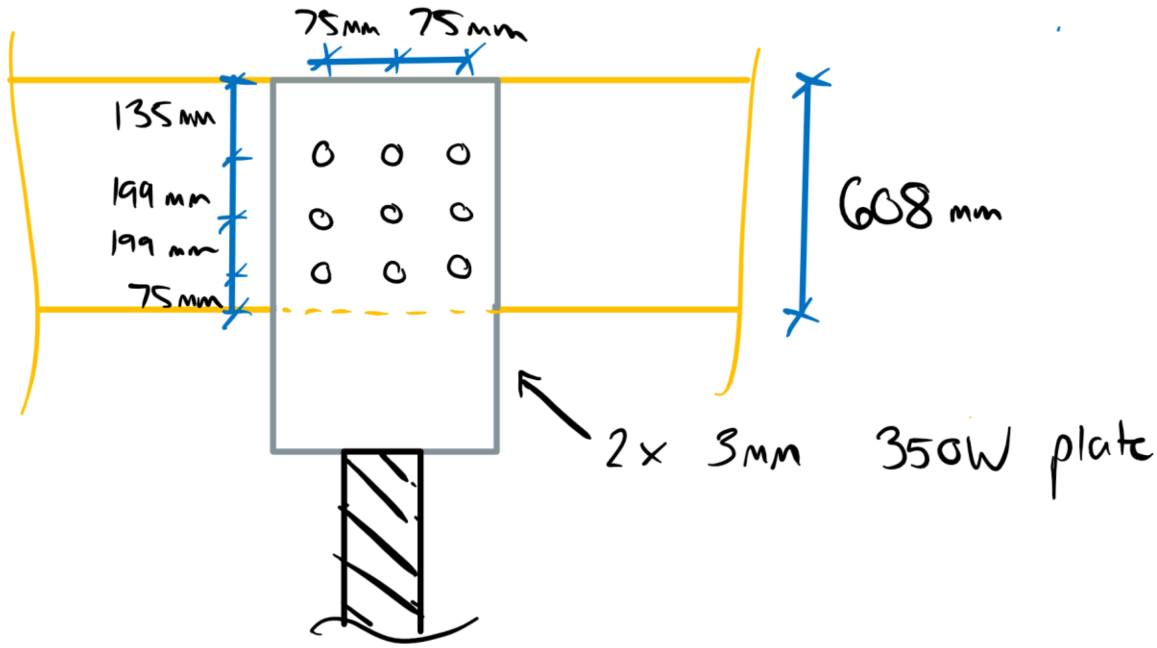
Splitting Perpendicular to Grain (**12.4.4.7**)

$QS_r = \phi_w Q_{si} K_D K_{sf} K_T$

$QS_i = 1.4t(d_e/1\text{-}d_e/d))^{1/2} = 1.4(265)(480/(1\text{-}480/608))^{1/2} = 177\text{ kN}$

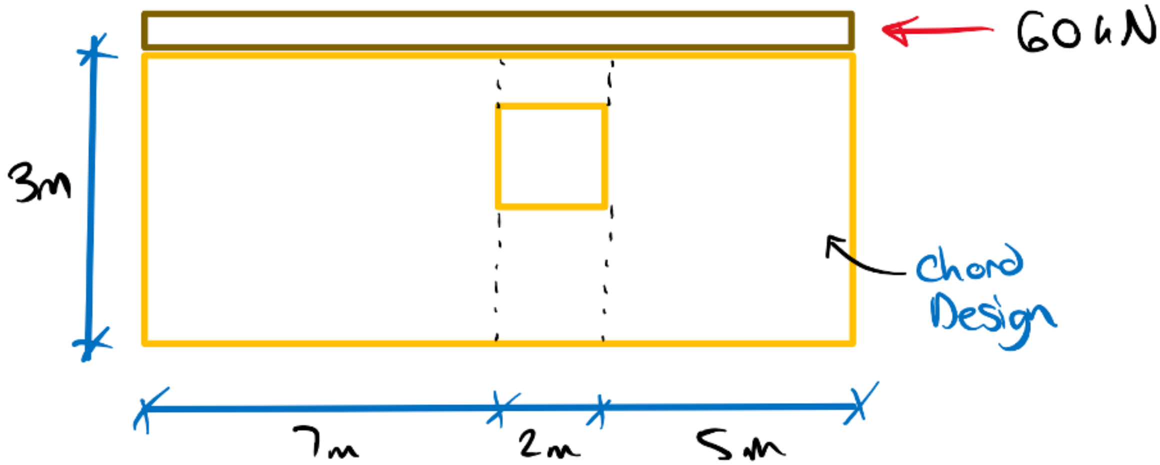
$QS_r = (0.70)(177\text{ kN})(1)(0.67)(1)$
$QS_r = 83\text{ kN} \geq QS_f = 75\text{ kN}$ **(Connection splitting resistance OK)**

Therefore, the following bolt layout for the 3-member connection satisfies fastener yielding, perpendicular to grain splitting, and beam shear.


75mm
75mm
135mm
199 mm
199 mm
75mm
608 mm
2x 3mm 350W plate

QUESTION 4

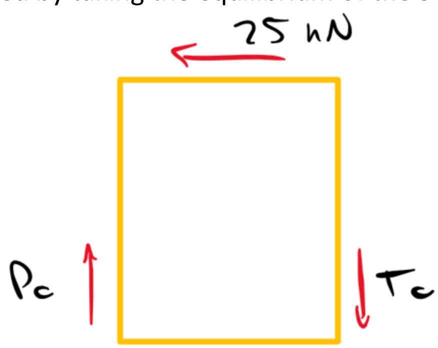
In order to determine the tension chord force, we must first distribute the overall building shear to the two shear wall segments.



Assuming the wall stiffness is proportional to length we get the following factored shear force for the 5 m long wall.

V_f = 5 m/(5 m + 7m) x 60 kN = 25 kN

The chord forces can be determined by taking the equilibrium of the 5 m wall.



$\Sigma M_p = 0 = -T_c(5\text{ m}) + (25\text{ kN})(3\text{ m})$
$T_c = 15\text{ kN}$

Therefore, we must design the tension chord for 15 kN. The chord is specified to use SPF No.3/Stud. We can use the selection tables to find a suitable member. The load duration factor is taken as 1.15. Note there is no reduction in net section area for nailed members.

Try 38x89 mm SPF No. 3:

$T_r = 14.6\text{ kN} \times 1.15 = 16.8\text{ kN} > T_f = 15\text{ kN}$

Therefore, use 38x89 SPF No.3 members for the tension chord.

Design of Splice:
Assuming a max chord member length of 1.5 m, we now need to design a nail splice connection to transmit the chord force. We will use another 38x89 SPF No.3 member for the splice. The nail penetration requirements from **12.9.2.2** must be met:

Try 2" common wire nails: d_f =2.84 mm **(Table A.12.9.5.2)**
$t_1 \geq 3d_f = 8.56$ mm < 38 mm (Member 1 Penetration OK)
$t_2 \geq 5d_f = 14.2$ mm
$L_f = 2" \times 25.4 = 50.8$ mm $\leq 38 + 14.2 = 52.2$ mm (Nail is too short to meet member 2 penetration)

Try 2.5" common wire nails: d_f =3.25 mm **(Table A.12.9.5.2)**
$t_1 \geq 3d_f = 9.75$ mm < 38 mm (Member 1 Penetration OK)
$t_2 \geq 5d_f = 16.25$ mm
$L_f = 2.5" \times 25.4 = 63.5$ mm $\geq 38 + 16.25 = 54.25$ mm (Nail is long enough)
$t_2 = 63.5 – 38 = 25.5$ mm

Check Nail Spacing Requirements (**12.9.2.1**):

$a = 16d_f = 52$ mm (Spacing para. to grain)
$b = 12d_f = 39$ mm (End dist. para. to grain)
$c = 8d_f = 26$ mm (Spacing perp. to grain)
$d = 4d_f = 13$ mm (Edge dist. perp. to grain)

Nail Yielding (**12.9.4**):

$N_r = \phi N_u n_f n_s J_F$

$J_f = J_E J_A J_B J_D = (1)(1)(1)(1.3) = 1.3$ (for shearwall construction)

$t_1 = 38$ mm
$f_1 = 50(0.42)(1\text{-}0.01(3.25)) = 20.3$ MPa
$t_2 = 25.5$ mm
$f_2 = 20.3$ MPa
$f_3 = 110(0.42)^{1.8}(1\text{-}0.01(3.25)) = 22.3$ MPa
$f_y = 50(16\text{-}3.25) = 637.5$ MPa

a) $n_u = 2.51$ kN
b) $n_u = 1.68$ kN
d) $n_u = 0.86$ kN
e) $n_u = 0.69$ kN **(GOVERNS)**
f) $n_u = 0.84$ kN
g) $n_u = 0.71$ kN

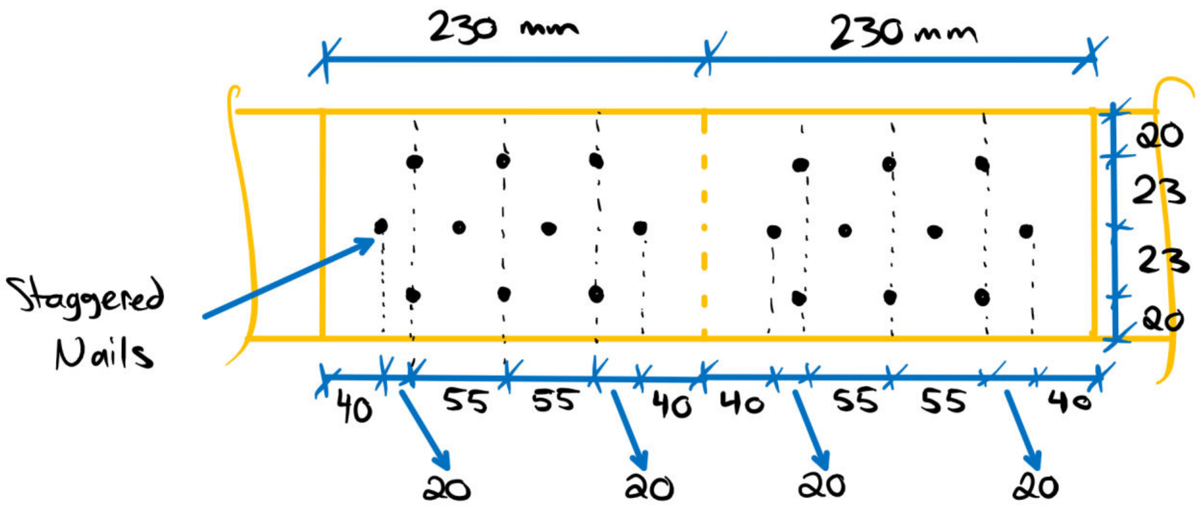
$N_u = n_u K_D K_{sf} K_T = 0.69$ kN$(1.15)(1)(1) = 0.79$ kN

$N_r = (0.80)(0.79$ kN$)n_f(1)(1.3) \geq N_f = 15$ kN
$n_f \geq 18.3$ nails (Min. nails for fastener yielding)

Therefore, let's try 12 nails in two rows of 6, with 8 nails staggered on the diagonals per **12.9.2.1**. The spacings are taken as follows. Remember that all member edges in this splice are loaded edges.

a = 55 mm (fastener spacing)
b = 40 mm (end distance)
c = 46 mm (row spacing)
d = 20 mm (edge distance)

Based on these spacing values, the splice member must be at least 560 mm long and is also a 38x89 SPF No. 3 member.



QUESTION 5

This question requires the determination of the critical moisture content such that the glulam swelling will induce a critical bearing failure in the 175x418 SPF 20f-E joist. A perpendicular to grain compression stiffness is given as 114 kN/mm and we are told to assume the critical bearing near the support (**7.5.9.3**) will govern. First, let's assess the critical bearing capacity of the configuration for wet conditions under short duration loading.

Critical Bearing Resistance (Bearing Near Support – **7.5.9.3**):

$Q_r' = 2/3\ \phi F_{cp} A_b' K_B K_{zcp}$

$F_{cp} = f_{cp} K_D K_{scp} K_T$ = (5.8 MPa)(1.15)(0.67)(1) = 4.5 MPa

$b_1 = b_2$ = 175 mm
L_1 = 265 mm
$L_2 >> L_1$ (therefore, the upper limit on A_b' will be reached per **7.5.9.3.2**)

$A_b' = 1.5bL_1$ = 1.5(175 mm)(265 mm) = 69562.5 mm^2

K_B = 1.0 (near end of member per **6.5.7.5**)
K_{zcp} = 1.0 (b/d < 1.0 per **6.5.7.5**)

Q_r' = 2/3 (0.80)(4.5 MPa)(69562.5 mm^2)(1)(1)
Q_r' = 167 kN

Therefore, for a critical load of 167 kN, the required amount of restrained swelling can be determined.

$\Delta_{req} = \Delta_{gap} + Q_r'/k$ = 20 mm + 167 kN/(114 kN/m) = 20 mm + 1.5 mm = 21.5 mm

For an original moisture content of 5%, the required final moisture content to yield the critical swelling can be calculated.

$\Delta_{req} = S = d(M_2 - M_1)c$ (**A.5.4.6**)
$M_2 = \Delta_{req}/cd + M_1$ (c = 0.002 for perpendicular to grain swelling – **A.5.4.6**)
M_2 = (21.5 mm)/(0.002)(418 mm) + 5%
M_2 = 30.7% **(Critical moisture content for critical bearing in the joist)**

Therefore, the moisture content of the wood would need to increase to over 30% to induced critical bearing in the member. However, moisture contents above 28% do not further contribute to swelling since the fiber saturation point is reached. Therefore, the potential swelling poses no risk to the critical bearing of the joist.