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18 March 2022

Enquiries: Francisco Toledo Silva
Project No: TBC

Chalet 1/11 Crackenback Drive, Thredbo
NSW, 2625

Attention: Mark Brown

Dear Mark

RE: Structural Engineering Assessment Chalet 1/11 Crackenback Drive, Thredbo – Loadbearing Capacity Existing Building.

Stantec have been engaged to review the proposed alterations to the internal works for a new mezzanine floor for the loadbearing capacity of the building as denoted on plan.

A structural assessment has been done and the proposed works reviewed in accordance with the existing structural arrangements as per site photos and provided drawing. The load bearing capacity of the existing foundations for the proposed construction is suitable and the proposed works present no geotechnical impact on the site or related land. It is noted that proposed works will be completed by an appropriately qualified and licenced contractor.

Stantec have reviewed, assessed and deem the proposed alterations as feasible.

Yours sincerely

Francisco Toledo Silva

Structural Project Engineer, Associate

BEng(Hons) GradCertEng(Structural)

for **Stantec**

Structural Project Engineer (Review);

Signature: Date: 18/03/22

Michael Ruescher

Structural Project Engineer, Associate

BEng BSc MIEAust NER (2378509)

Appendix A General Arrangement

GENERAL NOTES:

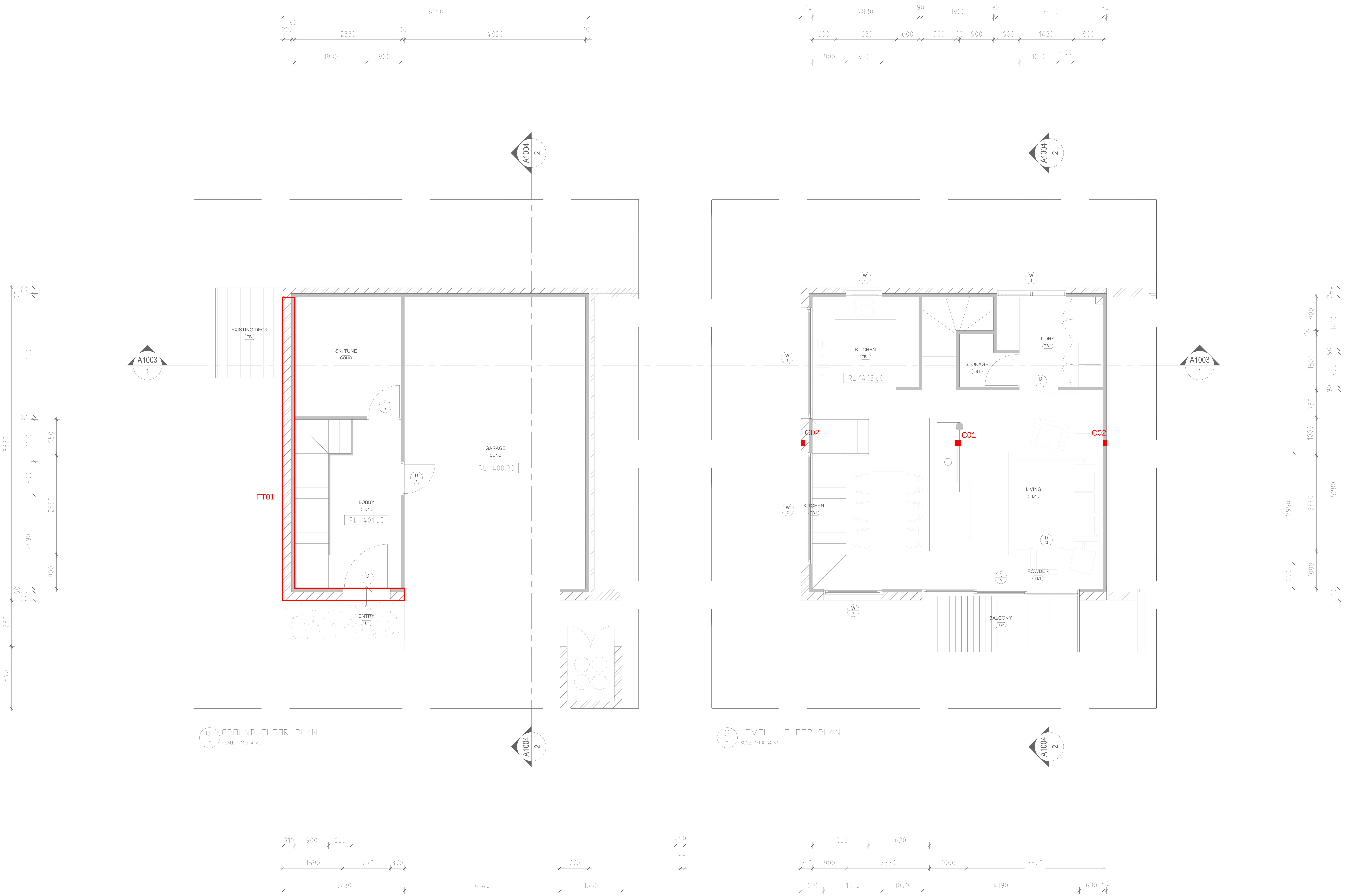
- ALL WORK TO COMPLY WITH BUILDING CODE OF AUSTRALIA, REQUIREMENTS OF RELEVANT STATUTORY AUTHORITIES/ LOCAL GOVERNMENT & RELEVANT AUSTRALIAN BUILDING STANDARDS
- CONTRACTOR TO VERIFY ALL DIMENSIONS ON SITE BEFORE COMMENCING WORK- SHOULD A DISCREPANCY BE IDENTIFIED PLEASE CONFIRM WITH ARCHITECT PRIOR TO PROCEEDING (DO NOT SCALE FROM DRAWINGS)
- ALL DRAWINGS TO BE READ IN CONJUNCTION WITH ALL RELEVANT DISCIPLINES SUCH AS SCHEDULES, BASIX CERTIFICATE & NATHERS SPEC, BCA & ACCESS REPORTS, STRUCTURAL, CIVIL, MECHANICAL, ELECTRICAL, HYDRAULIC, LANDSCAPE DRAWINGS, ETC.
- COPYRIGHT OF DESIGN SHOWN HEREON IS RETAINED BY BS ARCHITECTS AND AUTHORITY IS REQUIRED FOR ANY REPRODUCTION
- WHEN PROPRIETARY PRODUCTS ARE REFERRED TO, INSTALL IN ACCORDANCE WITH THE MANUFACTURERS WRITTEN INSTRUCTIONS
- ARCHITECTURAL PLANS TO BE READ IN CONJUNCTION WITH CONSULTANT'S DRAWINGS, SPECIFICATIONS & REPORTS

LEGEND:

COS	CONFIRM ON SITE
EQ	EQUAL DISTANCE
D	DOOR
W	WINDOW
TL	TILE
CPT	CARPET
TB	TIMBER BOARD

WALL HATCH LEGEND:

- EXISTING STUD WALL
- EXISTING BRICK WORK
- PROPOSED STUD WALL



PROJECT:

PROPOSED INTERIOR ALTERATION

1 / 11 CRACKENBACK RIDGE, THREDBO VILLAGE

CLIENT: Kim and Graham Selig

AMENDMENTS

ISSUE	DATE	DESCRIPTION	ISSUE	DATE	DESCRIPTION
P1	24/08/21	PRELIMINARY ISSUE 1			
P2	19/10/21	PRELIMINARY ISSUE 2			
DA	21/11/21	DEVELOPMENT APPLICATION ISSUE			

N

SCALE 1:100 @ A3
SCALE 1:50 @ A1

0 1 2 5m

DA-A1001

GROUND/LEVEL 1 FLOOR PLAN (EXISTING)

ISSUE - DA

NOVEMBER 2021

BS | ARCHITECTS

BS ARCHITECTURE
733 Bourke Street, Redfern NSW 2016
P - (+61) 402 117 955 E - B.D.Selig@gmail.com W - Benjaminselig.com

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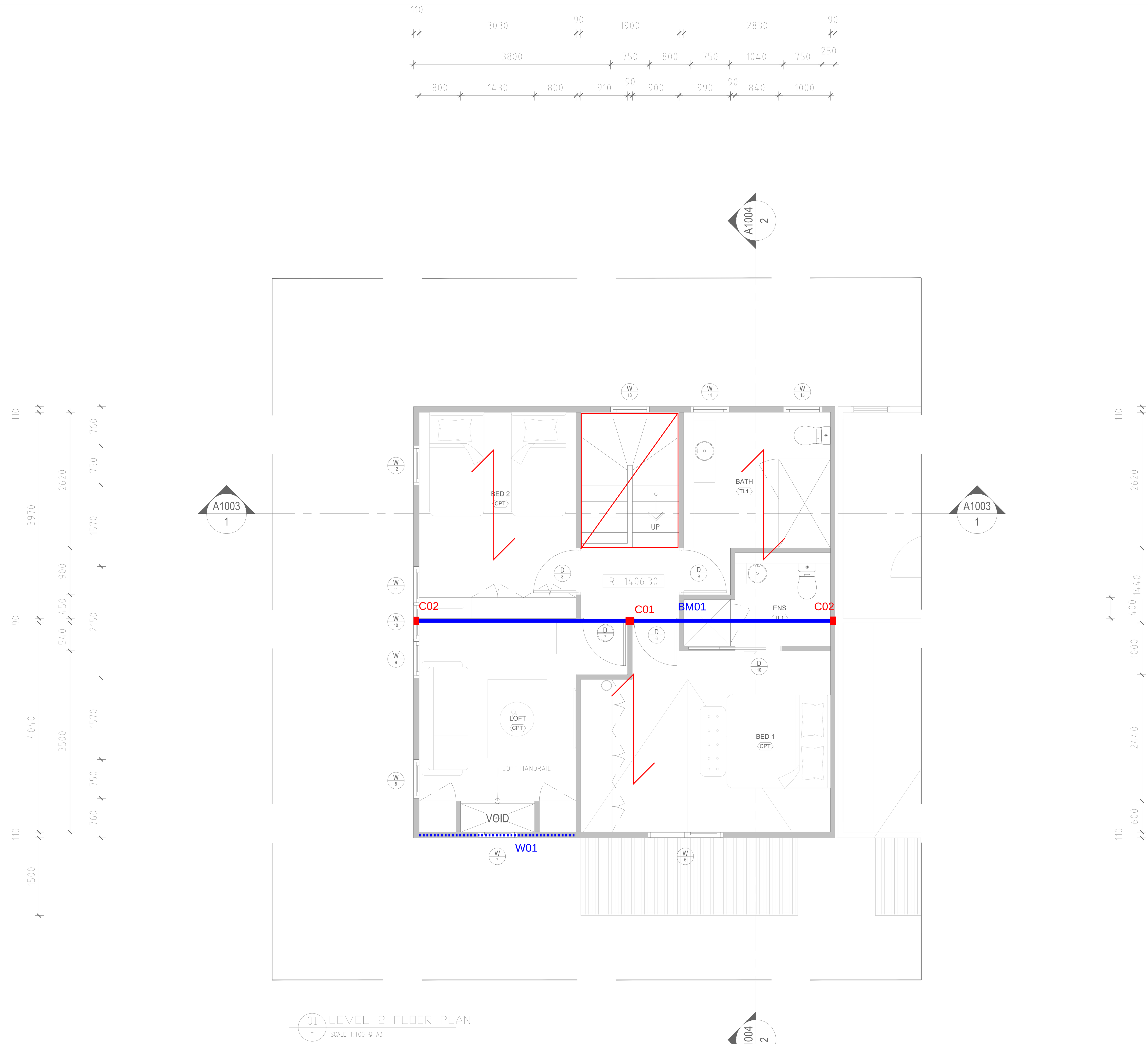

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PROJECT:

PROPOSED INTERIOR ALTERATION

1 / 11 CRACKENBACK RIDGE, THREDBO VILLAGE

CLIENT: Kim and Graham Selig

N

SCALE 1:100 @ A3
SCALE 1:50 @ A1

0 1 2 5m

DA-A2002

LEVEL 2 FLOOR PLAN (PROPOSED)

ISSUE - DA

NOVEMBER 2021

ARCHITECTS

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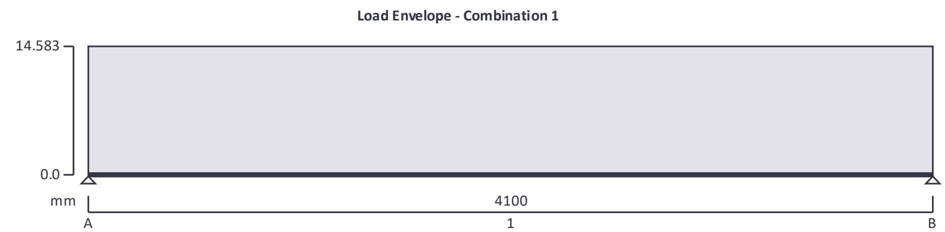
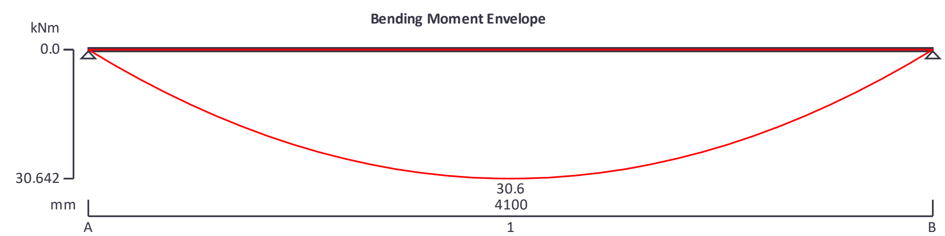
Appendix B Calculation Pack

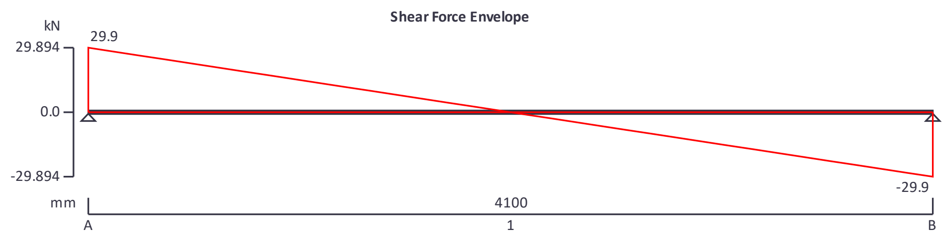
STEEL BEAM ANALYSIS & DESIGN (AS4100)

In accordance with AS4100-1998 incorporating Amendment No.1 2012

TEDDS calculation version 3.0.10







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 4.15 kN/m
	Imposed full UDL 6.225 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.20
		Imposed × 1.50
		Permanent × 1.20

Imposed × 1.50

Support B — Permanent × 1.20

Imposed × 1.50

Analysis results

Maximum moment — M_{max} = **30.6** kNm — M_{min} = **0** kNm

Maximum shear — V_{max} = **29.9** kN — V_{min} = **-29.9** kN

Deflection — δ_{max} = **9.3** mm — δ_{min} = **0** mm

Maximum reaction at support A — R_{A_max} = **29.9** kN — R_{A_min} = **29.9** kN

Unfactored permanent load reaction at support A — $R_{A\_Permanent}$ = **9** kN

Unfactored imposed load reaction at support A — $R_{A\_Imposed}$ = **12.8** kN

Maximum reaction at support B — R_{B_max} = **29.9** kN — R_{B_min} = **29.9** kN

Unfactored permanent load reaction at support B — $R_{B\_Permanent}$ = **9** kN

Unfactored imposed load reaction at support B — $R_{B\_Imposed}$ = **12.8** kN

Section details

Section type — **200x22.3 UB (AISC 1994)**

Steel grade — **300**

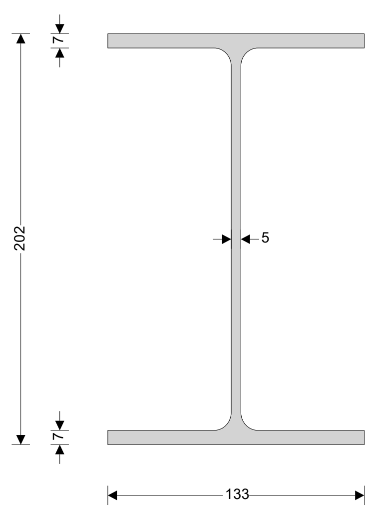
From table 2.1: Strengths of steels

Thickness of material — $t = \max(t_f, t_w)$ = **7.0** mm

Yield stress — f_y = **320** N/mm^2

Tensile strength — f_u = **440** N/mm^2

Modulus of elasticity — E = **200000** N/mm^2



Capacity factors (ϕ) for strength limit states - Table 3.4

Capacity factor — ϕ = **0.90**

Lateral restraint

Span 1 has lateral restraint at supports only

Section slenderness - Section 5.2.2

Flange slenderness — $\lambda_{ef} = (b_f - t_w) / (2 \times t_f) \times \sqrt{[f_y / 250 \text{ N/mm}^2]}$ = **10.3**

Flange yield slenderness limit - Table 5.2 — λ_{eyf} = **16** — $\lambda_{ef} / \lambda_{eyf}$ = **0.646**

Web slenderness $\lambda_{ew} = d_1 / t_w \times \sqrt{[f_y / 250 \text{ N/mm}^2]} = \mathbf{42.5}$

Web yield slenderness limit - Table 5.2 $\lambda_{eyw} = \mathbf{115}$ $\lambda_{ew} / \lambda_{eyw} = \mathbf{0.370}$

Section slenderness $\lambda_s = \mathbf{10.3}$

Section plasticity limit - Table 5.2 $\lambda_{sp} = \mathbf{9}$

Yield slenderness limit - Table 5.2 $\lambda_{sy} = \mathbf{16}$

$\lambda_{sp} < \lambda_s < \lambda_{sy}$ - Section is non-compact

Shear capacity of webs - Section 5.11

Design shear force $V^* = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{29.9}$ kN

$d_1 / t_w < 82 / \sqrt{[f_y / 250 \text{ N/mm}^2]}$

Nominal shear capacity of the web shall be taken as the nominal shear yield capacity

Shear yield capacity - Clause 5.11.4

Gross sectional area of web $A_w = A_y = \mathbf{1010}$ mm^2

Nominal shear yield capacity $V_w = 0.6 \times f_y \times A_w = \mathbf{193.9}$ kN

Shear capacity - Clause 5.11.1

Nominal shear capacity - cl.5.11.2 $V_v = V_w = \mathbf{193.9}$ kN

Design shear capacity $V_{vc} = \phi \times V_v = \mathbf{174.5}$ kN

PASS - Design shear capacity exceeds design shear force

Design for bending moment - Section 5.1

Design bending moment $M^* = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{30.6}$ kNm

Section moment capacity for bending about a principal axis - Section 5.2

Effective compact section modulus - cl.5.2.3 $Z_c = \min(S_x, 1.5 \times Z_x) = \mathbf{232000}$ mm^3

Effective section modulus - cl.5.2.4 $Z_e = Z_x + [(\lambda_{sy} - \lambda_s) / (\lambda_{sy} - \lambda_{sp}) \times (Z_c - Z_x)] = \mathbf{227469}$ mm^3

Nominal section moment capacity - cl.5.2.1 $M_s = f_y \times Z_e = \mathbf{72.8}$ kNm

Design section moment capacity $M_{sc} = \phi \times M_s = \mathbf{65.5}$ kNm

Segments with full lateral restraint - Section 5.3.2

Smaller segment end moment $M_{s1\_end\_min} = \mathbf{0}$ kNm

Larger segment end moment $M_{s1\_end\_max} = \mathbf{0}$ kNm

End moment ratio $\beta_m = -0 = \mathbf{0.000}$

Maximum segment length - cl.5.3.2.4 $L_{s1\_max} = r_y \times (80 + 50 \times \beta_m) \times \sqrt{[250 \text{ N/mm}^2 / f_y]} = \mathbf{2188}$ mm

Segment is not considered to have full lateral restraint

Member capacity of segments without full lateral restraint - Section 5.6

Moment at quarter point of segment $M_2^* = \mathbf{23}$ kNm

Moment at center-line of segment $M_3^* = \mathbf{30.6}$ kNm

Moment at three quarter point of segment $M_4^* = \mathbf{23}$ kNm

Maximum moment in segment $M_m^* = \mathbf{30.6}$ kNm

Moment modification factor $\alpha_m = \min(1.7 \times M_m^* / \sqrt{[M_2^{*2} + M_3^{*2} + M_4^{*2}]}, 2.5) = \mathbf{1.166}$

Twist restraint factor - Table 5.6.3(1) $k_t = 1 + [2 \times (d_1 / L_{s1}) \times (t_f / (2 \times t_w))^3] = \mathbf{1.031}$

Load height factor - Table 5.6.3(2) $k_l = \mathbf{1.000}$

Lateral rotation restraint factor - Table 5.6.3(3) $k_r = \mathbf{1.000}$

Effective length $l_e = k_t \times k_l \times k_r \times L_{s1} = \mathbf{4229}$ mm

Reference buckling moment - eq.5.6.1.1(3) $M_{oa} = M_o = \sqrt{[(\pi^2 \times E \times I_y / l_e^2) \times [G \times J + (\pi^2 \times E \times I_w / l_e^2)]]} = \mathbf{44.3}$ kNm

Slenderness reduction factor - eq.5.6.1.1(2) $\alpha_s = 0.6 \times [\sqrt{[(M_s / M_{oa})^2 + 3]} - (M_s / M_{oa})] = \mathbf{0.447}$

Nominal member moment capacity - eq.5.6.1.1(1) $M_b = \min(\alpha_m \times \alpha_s, 1) \times M_s =$ **37.9** kNm

Design member moment capacity $M_{bc} = \phi \times M_b =$ **34.1** kNm

PASS - Design member moment capacity exceeds design bending moment

Interaction of shear and bending - Section 5.12

Nominal shear capacity with bending - cl.5.12.3 $V_{vm} = V_v =$ **193.9** kN

Design shear capacity with bending $V_{vmc} = \phi \times V_{vm} =$ **174.5** kN

PASS - Design shear capacity with bending exceeds design shear force

Serviceability limit state - Section 3.5

Consider deflection due to permanent and imposed loads

Limiting deflection $\delta_{lim} = \min(14\text{ mm}, L_{s1} / 250) =$ **14** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **9.26** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL MEMBER DESIGN (AS4100)

In accordance with AS4100-1998 incorporating Amendment No.1 2012

TEDDS calculation version 3.0.10

Section details

Section type **89x89x5 SHS C350 (BHP 1999)**

Steel grade **C350**

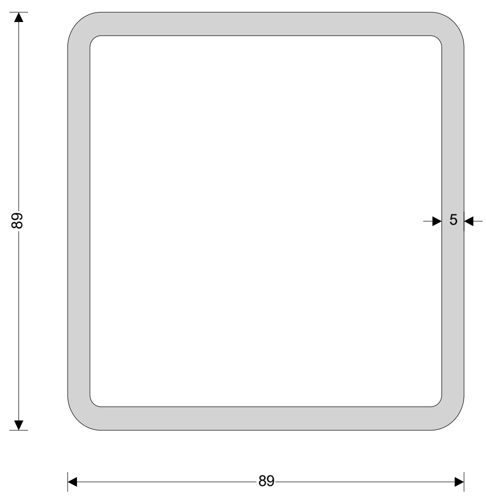
From table 2.1: Strengths of steels

Thickness of material t = **5.0** mm

Yield stress f_y = **350** N/mm^2

Tensile strength f_u = **430** N/mm^2

Modulus of elasticity E = **200000** N/mm^2



Capacity factors (ϕ) for strength limit states - Table 3.4

Capacity factor ϕ = **0.90**

Lateral restraint

Distance between major axis restraints L_x = **2700** mm

Distance between minor axis restraints L_y = **2700** mm

Effective length factors

Effective length factor in major axis k_{ex}= **1.200**

Effective length factor in minor axis k_{ey} = **1.200**

Section slenderness - Section 5.2.2

Flange slenderness $\lambda_{ef} = (b - 2 \times t) / t \times \sqrt{[f_y / 250 \text{ N/mm}^2]}$ = **18.7**

Flange yield slenderness limit - Table 5.2 λ_{eyf} = **40** $\lambda_{ef} / \lambda_{eyf}$ = **0.467**

Web slenderness $\lambda_{ew} = (d - 2 \times t) / (t) \times \sqrt{[f_y / 250 \text{ N/mm}^2]}$ = **18.7**

Web yield slenderness limit - Table 5.2 λ_{eyw} = **115** $\lambda_{ew} / \lambda_{eyw}$ = **0.163**

Section slenderness λ_s = **18.7**

Section plasticity limit - Table 5.2 λ_{sp} = **30**

Yield slenderness limit - Table 5.2 λ_{sy} = **40**

$\lambda_s < \lambda_{sp}$ - Section is compact

Design for bending moment - Section 5.1

Design bending moment $M^* = \mathbf{0.4}$ kNm

Section moment capacity for bending about a principal axis - Section 5.2

Effective section modulus - cl.5.2.3 $Z_e = \min(S_x, 1.5 \times Z_x) = \mathbf{49210}$ mm^3

Nominal section moment capacity - cl.5.2.1 $M_s = f_y \times Z_e = \mathbf{17.2}$ kNm

Design section moment capacity $M_{sc} = \phi \times M_s = \mathbf{15.5}$ kNm

Segments with full lateral restraint - Section 5.3.2

End moment ratio $\beta_m = \mathbf{-1.000}$

Maximum segment length - cl.5.3.2.4 $L_{s\_max} = r_y \times (1800 + 1500 \times \beta_m) \times (b / d) \times (250 \text{ N/mm}^2 / f_y) = \mathbf{7236}$ mm

Segment considered to have full lateral restraint

PASS - Design section moment capacity exceeds design bending moment

Members subject to axial compression - Section 6

Design compression force $N^* = \mathbf{59.8}$ kN

Cross-sectional area of holes $A_h = \mathbf{0}$ mm^2

Net area of cross-section $A_n = A_g - A_h = \mathbf{1594}$ mm^2

Nominal section capacity - Section 6.2

Flange yield slenderness limit - Table 6.2.4 $\lambda_{eyf} = \mathbf{40}$

Effective width of flanges - cl.6.2.4 $b_{ef} = \min(\lambda_{eyf} / \lambda_{ef}, 1) \times (b - 2 \times t) = \mathbf{79.0}$ mm

Web yield slenderness limit - Table 6.2.4 $\lambda_{eyw} = \mathbf{40}$

Effective width of web - cl.6.2.4 $b_{ew} = \min(\lambda_{eyw} / \lambda_{ew}, 1) \times (d - 2 \times t) = \mathbf{79.0}$ mm

Effective area of section $A_e = A_g - 2 \times [(b - 2 \times t) - b_{ef} + (d - 2 \times t) - b_{ew}] \times t = \mathbf{1594}$ mm^2

Form factor - cl.6.2.2 $k_f = A_e / A_g = \mathbf{1.000}$

Nominal section capacity in compression - cl.6.2.1 $N_s = k_f \times A_n \times f_y = \mathbf{557.9}$ kN

Design section capacity in compression $N_{sc} = \phi \times N_s = \mathbf{502.1}$ kN

Nominal member capacity in major (x-x) axis - Section 6.3

Effective length for buckling $l_{ex} = L_x \times k_{ex} = \mathbf{3240}$ mm

Modified compression member slenderness $\lambda_{nx} = l_{ex} / r_x \times \sqrt{[k_f]} \times \sqrt{[f_y / 250 \text{ N/mm}^2]} = \mathbf{113.521}$

Compression member factor $\alpha_{ax} = 2100 \times (\lambda_{nx} - 13.5) / (\lambda_{nx}^2 - 15.3 \times \lambda_{nx} + 2050) = \mathbf{15.912}$

Member section constant - Table 6.3.3(1) $\alpha_b = \mathbf{-0.5}$

Slenderness ratio $\lambda_x = \lambda_{nx} + \alpha_{ax} \times \alpha_b = \mathbf{105.565}$

Compression member imperfection factor $\eta_x = 0.00326 \times (\lambda_x - 13.5) = \mathbf{0.300}$

Compression member factor $\xi_x = ((\lambda_x / 90)^2 + 1 + \eta_x) / (2 \times (\lambda_x / 90)^2) = \mathbf{0.972}$

Member slenderness reduction factor $\alpha_{cx} = \xi_x \times [1 - \sqrt{[1 - (90 / (\xi_x \times \lambda_x))^2]}] = \mathbf{0.505}$

Nominal member capacity in compression - cl.6.3.3 $N_{cx} = \alpha_{cx} \times N_s = \mathbf{281.5}$ kN

Design member capacity in compression $N_{cxc} = \phi \times N_{cx} = \mathbf{253.4}$ kN

Nominal member capacity in minor (y-y) axis - Section 6.3

Effective length for buckling $l_{ey} = L_y \times k_{ey} = \mathbf{3240}$ mm

Modified compression member slenderness $\lambda_{ny} = l_{ey} / r_y \times \sqrt{[k_f]} \times \sqrt{[f_y / 250 \text{ N/mm}^2]} = \mathbf{113.521}$

Compression member factor $\alpha_{ay} = 2100 \times (\lambda_{ny} - 13.5) / (\lambda_{ny}^2 - 15.3 \times \lambda_{ny} + 2050) = \mathbf{15.912}$

Member section constant - Table 6.3.3(1) $\alpha_b = \mathbf{-0.5}$

Slenderness ratio $\lambda_y = \lambda_{ny} + \alpha_{ay} \times \alpha_b = \mathbf{105.565}$

Compression member imperfection factor $\eta_y = 0.00326 \times (\lambda_y - 13.5) = \mathbf{0.300}$

Compression member factor $\xi_y = ((\lambda_y / 90)^2 + 1 + \eta_y) / (2 \times (\lambda_y / 90)^2) = \mathbf{0.972}$

Member slenderness reduction factor $\alpha_{cy} = \xi_y \times [1 - \sqrt{[1 - (90 / (\xi_y \times \lambda_y))^2]}] = \mathbf{0.505}$

Nominal member capacity in compression - cl.6.3.3 $N_{cy} = \alpha_{cy} \times N_s = \mathbf{281.5}$ kN

Design member capacity in compression $N_{cyc} = \phi \times N_{cy} = \mathbf{253.4}$ kN

PASS - Design capacity in compression exceeds design compression force

Members subject to combined actions - Section 8

Nominal section moment capacity - cl.8.3.2 $M_r = M_s = \mathbf{17.2}$ kNm

Design section moment capacity $M_{rc} = \phi \times M_r = \mathbf{15.5}$ kNm

End moment ratio $\beta_m = \mathbf{-1.000}$

Nominal member moment capacity - cl.8.4.2.2 $M_i = \min(M_s \times ([1 - ((1 + \beta_m) / 2)^3] \times (1 - N^* / (\phi \times N_{cx})) + 1.18 \times ((1 + \beta_m) / 2)^3 \times \sqrt{(1 - N^* / (\phi \times N_{cx}))}), M_r) = \mathbf{13.2}$ kNm

Design member moment capacity $M_{ic} = \phi \times M_i = \mathbf{11.8}$ kNm

PASS - Combined axial and bending check is satisfied

TIMBER MEMBER DESIGN TO AS1720.1-2010

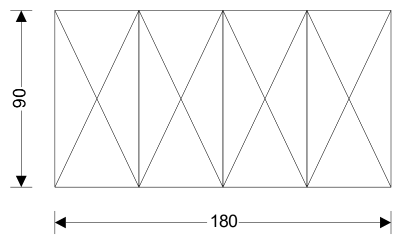
Tedds calculation version 1.7.04

Analysis results

Design moment in major axis — M^*_x = **0.300** kNm

Design moment in minor axis — M^*_y = **0.300** kNm

Design axial compression — N^*_c = **59.800** kN



Timber section details

Breadth of timber sections — b = **45** mm

Depth of timber sections — d = **90** mm

Number of timber sections in member — N = **4**

Overall breadth of timber member — $b_b = N \times b$ = **180** mm

Timber species — **Mixed softwood species (excl.Pinus species)**

Moisture condition — **Seasoned**

Timber strength grade - Table H3.1 — **MGP10**

Member details

Load duration - cl.2.4.1 — **Standard test**

Equilibrium moisture content — **15 %**

Overall length of member — L_x = **2700** mm

Effective length factor - Table 3.2 — g_{13} = **0.75**

Distance between lateral restraints in major axis — L_{ax} = **2700** mm

Distance between lateral restraints in minor axis — L_{ay} = **2700** mm

Section properties

Cross sectional area of member — $A = N \times b \times d$ = **16200** mm^2

Section modulus — $Z_x = N \times b \times d^2 / 6$ = **243000** mm^3

$Z_y = d \times (N \times b)^2 / 6$ = **486000** mm^3

Second moment of area — $I_x = N \times b \times d^3 / 12$ = **10935000** mm^4

$I_y = d \times (N \times b)^3 / 12$ = **43740000** mm^4

Radius of gyration — $r_x = \sqrt{(I_x / A)}$ = **26.0** mm

$r_y = \sqrt{(I_y / A)}$ = **52.0** mm

Modification factors

Duration of load factor for strength - Table 2.3 — k_1 = **1.00**

Moisture condition factor - cl.2.4.2.3 — k_4 = **1.00**

Temperature factor - cl.2.4.3 — k_6 = **1.00**

Length and position of bearing factor - cl.2.4.4 — k_7 = **1.00**

Strength sharing factor - Table 2.7 — k_9 = **1.24**

Temporary design action ratio — $r = \mathbf{0.25}$

Material constant - exp.E2(1) — $\rho_b = 14.71 \times (E / f'_b)^{-0.480} \times r^{-0.061} = \mathbf{0.75}$

Distance between discrete lateral restraints — $L_{ay} = \mathbf{2700}$ mm — $L_{ay} / d < 64 \times [N \times b / (\rho_b \times d)]^2$

Major axis slenderness coefficient - cl.3.2.3.2(b) — $S_1 = \mathbf{0.00}$

Major axis bending stability factor - exp.3.2(10) — $k_{12bx} = \mathbf{1.00}$

Minor axis slenderness coefficient - cl.3.2.3.2 (c) — $S_2 = \mathbf{0.00}$

Minor axis bending stability factor - cl.3.2.4 — $k_{12by} = \mathbf{1.00}$

Material constant - exp.E2(3) — $\rho_c = 11.39 \times (E / f'_c)^{-0.408} \times r^{-0.074} = \mathbf{0.96}$

Major axis slenderness coefficient - 3.3(6) — $S_3 = g_{13} \times L_x / d = \mathbf{22.50}$

Major axis comp.stability factor - exp.3.3(11c) — $k_{12cx} = 200 / (\rho_c \times S_3)^2 = \mathbf{0.43}$

Minor axis slenderness coeff. - exp.3.3(8) & (9) — $S_4 = \min(L_{ay} / (N \times b), g_{13} \times L_x / (N \times b)) = \mathbf{11.25}$

Minor axis comp.stability factor - exp.3.3(11b) — $k_{12cy} = 1.5 - 0.05 \times \rho_c \times S_4 = \mathbf{0.96}$

Bending strength - cl.3.2.1

Capacity factor - Table 2.1 — $\phi_b = \mathbf{0.9}$

Design capacity in major axis bending - cl.3.2(2) — $\phi M_x = \phi_b \times k_1 \times k_4 \times k_6 \times k_9 \times k_{12bx} \times f'_b \times Z_x = \mathbf{4.610}$ kNm

Design capacity in minor axis bending - cl.3.2(2) — $\phi M_y = \phi_b \times k_1 \times k_4 \times k_6 \times k_9 \times k_{12by} \times f'_b \times Z_y = \mathbf{9.220}$ kNm

PASS - Design capacity in bending exceeds design bending moment

Compressive strength - cl.3.3.1

Capacity factor - Table 2.1 — $\phi_c = \mathbf{0.9}$

Cross-sectional area of member — $A_c = N \times b \times d = \mathbf{16200}$ mm^2

Major axis design capacity in compression - exp.3.3(2)

$\phi N_{cx} = \phi_c \times k_1 \times k_4 \times k_6 \times k_{12cx} \times f'_c \times A_c = \mathbf{113.042}$ kN

Minor axis design capacity in compression - exp.3.3(2)

$\phi N_{cy} = \phi_c \times k_1 \times k_4 \times k_6 \times k_{12cy} \times f'_c \times A_c = \mathbf{252.283}$ kN

PASS - Design capacity in compression exceeds design compression

Beam-column bent about both axes - Appendix E5

Beam-column bent about both axes check - exp.E5(1) and E5(2)

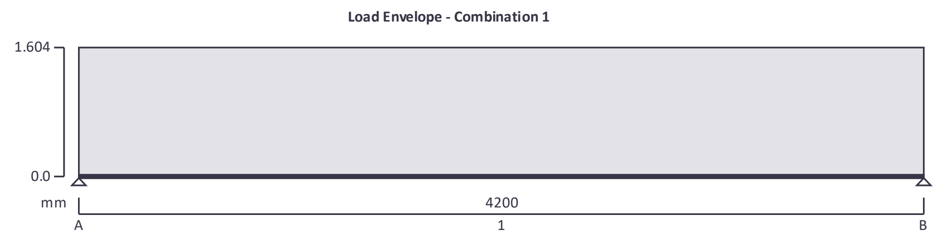
$[M^*_x / \phi M_x]^2 + [M^*_y / \phi M_y] + [N^*_c / \phi N_{cy}] = \mathbf{0.274} < 1$

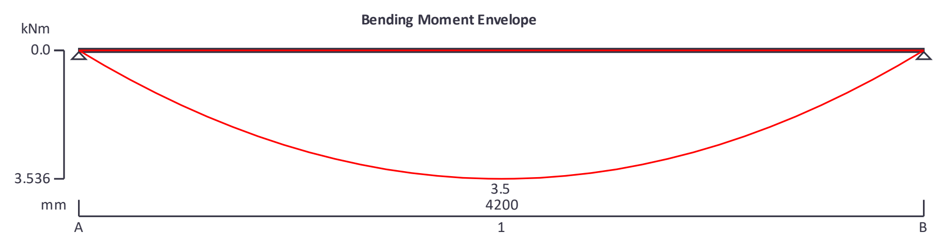
$[M^*_x / \phi M_x] + [M^*_y / \phi M_y]^2 + [N^*_c / \phi N_{cx}] = \mathbf{0.595} < 1$

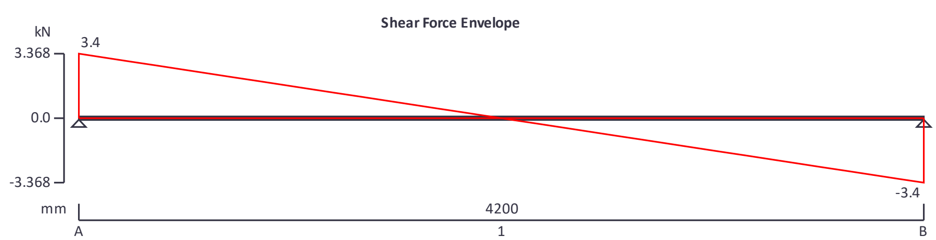
PASS - Beam design meets combined bending and compression criteria

TIMBER BEAM ANALYSIS & DESIGN TO AS1720.1-2010

Tedds calculation version 1.7.04







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 0.450 kN/m

Live full UDL 0.675 kN/m

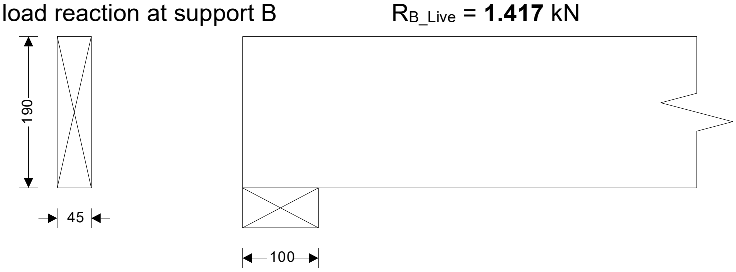
Load combinations

Load combination 1	Support A	Permanent × 1.20
		Live × 1.50
	Span 1	Permanent × 1.20
		Live × 1.50
	Support B	Permanent × 1.20
		Live × 1.50

Analysis results

Maximum moment M_{max} = **3.536** kNm M_{min} = **0.000** kNm

Design moment	$M^* = max(abs(M_{max}),abs(M_{min}))$ = **3.536** kNm
Maximum shear	V_{max} = **3.368** kN V_{min} = **-3.368** kN
Design shear	$V^* = max(abs(V_{max}),abs(V_{min}))$ = **3.368** kN
Total load on member	W_{tot} = **6.736** kN
Reactions at support A	R_{A_max} = **3.368** kN R_{A_min} = **3.368** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **1.035** kN
Unfactored live load reaction at support A	R_{A_Live} = **1.418** kN
Reactions at support B	R_{B_max} = **3.368** kN R_{B_min} = **3.368** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **1.035** kN
Unfactored live load reaction at support B	R_{B_Live} = **1.417** kN



Timber section details

Breadth of timber sections	b = **45** mm
Depth of timber sections	d = **190** mm
Number of timber sections in member	N = **1**
Overall breadth of timber member	$b_b = N \times b$ = **45** mm
Timber species	**Mixed softwood species (excl.Pinus species)**
Moisture condition	**Seasoned**
Timber strength grade - Table H3.1	**MGP10**

Member details

Load duration - cl.2.4.1	**Standard test**
Equilibrium moisture content	**15 %**
Length of span	L_{s1} = **4200** mm
Length of bearing	L_b = **100** mm
Number of discrete parallel systems	N_{mem} = **10**
Centre-to-centre spacing of discrete systems	s = **450** mm

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8550** mm^2
Section modulus	$Z_x = N \times b \times d^2 / 6$ = **270750** mm^3
	$Z_y = d \times (N \times b)^2 / 6$ = **64125** mm^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **25721250** mm^4
	$I_y = d \times (N \times b)^3 / 12$ = **1442812** mm^4
Radius of gyration	$r_x = \sqrt{(I_x / A)}$ = **54.8** mm
	$r_y = \sqrt{(I_y / A)}$ = **13.0** mm

Modification factors

Duration of load factor for strength - Table 2.3	k_1 = **1.00**
Moisture condition factor - cl.2.4.2.3	k_4 = **1.00**

Temperature factor - cl.2.4.3 k_6 = **1.00**

Length and position of bearing factor - Table 2.6 k_7 = **1.00**

Geometric factor appropriate to the number of members in a combined parallel system - Table 2.7

g_{31} = **1.00**

Geometric factor appropriate to the number of members in a discrete system - Table 2.7

g_{32} = **1.33**

Strength sharing factor (cl. 2.4.5.3) $k_9 = \max(1.0, g_{31} + (g_{32} - g_{31}) \times [1 - 2 \times s / L_x])$ = **1.26**

Temporary design action ratio r = **0.25**

Material constant - exp.E2(1) $\rho_b = 14.71 \times (E / f'_b)^{-0.480} \times r^{-0.061}$ = **0.73**

Distance between discrete lateral restraints L_{ay} = **0** mm $L_{ay} / d < 64 \times [N \times b / (\rho_b \times d)]^2$

Major axis slenderness coefficient - cl.3.2.3.2(b) S_1 = **0.00**

Major axis bending stability factor - exp.3.2(10) k_{12bx} = **1.00**

Minor axis slenderness coefficient - cl.3.2.3.2 (c) S_2 = **0.00**

Minor axis bending stability factor - cl.3.2.4 k_{12by} = **1.00**

Bearing strength - cl.3.2.6

Capacity factor - Table 2.1 ϕ_p = **0.9**

Bearing area for loading perpendicular to grain $A_p = N \times b \times L_b$ = **4500** mm^2

Design capacity in bearing - exp.3.2(16) $\phi N_p = \phi_p \times k_1 \times k_4 \times k_6 \times k_7 \times f'_p \times A_p$ = **27.540** kN

PASS - Design capacity in bearing perpendicular to the grain exceeds design bearing load

Bending strength - cl.3.2.1

Capacity factor - Table 2.1 ϕ_b = **0.9**

Design capacity in bending - cl.3.2(2) $\phi M = \phi_b \times k_1 \times k_4 \times k_6 \times k_9 \times k_{12bx} \times f'_b \times Z_x$ = **4.910** kNm

PASS - Design capacity in bending exceeds design bending moment

Flexural shear strength - cl.3.2.5

Capacity factor - Table 2.1 ϕ_s = **0.9**

Shear plane area $A_s = N \times b \times d \times 2 / 3$ = **5700** mm^2

Design shear capacity - exp.3.2(14) $\phi V = \phi_s \times k_1 \times k_4 \times k_6 \times f'_s \times A_s$ = **12.825** kN

PASS - Design shear capacity exceeds design shear force

Deflection - AS/NZS 1170.0

Deflection limit - Table C1 $\delta_{lim} = \text{Min}(20\text{ mm}, 0.004 \times L_{s1})$ = **16.800** mm

Deflection due to permanent load δ_G = **7.990** mm

Deflection due to imposed load δ_Q = **10.945** mm

Load factor - Table 4.1 ψ = **0.7**

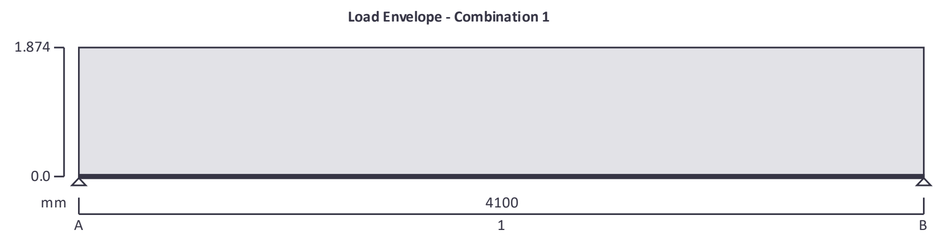
Creep factor (Standard test) j_2 = **1.000**

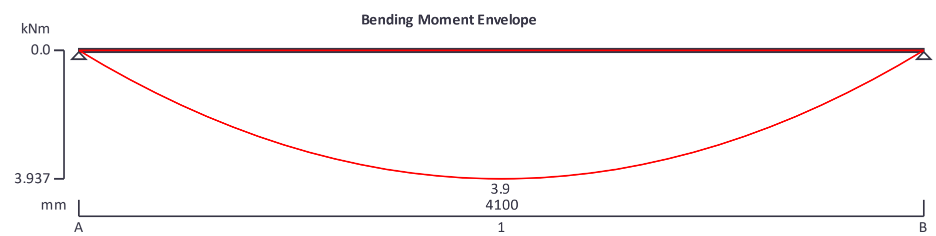
Total deflection $\delta_{tot} = j_2 \times (\delta_G + \psi \times \delta_Q)$ = **15.651** mm

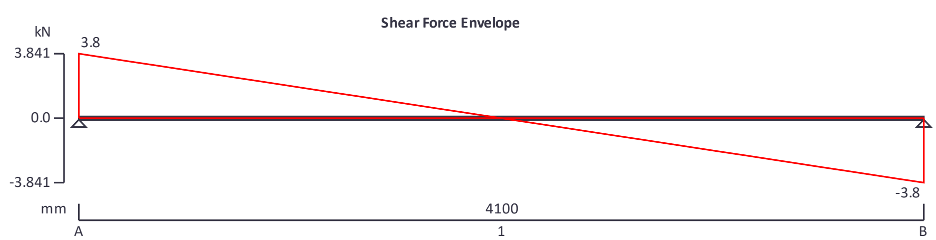
PASS - Total deflection is less than the deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO AS1720.1-2010

Tedds calculation version 1.7.04







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 0.675 kN/m

Live full UDL 0.675 kN/m

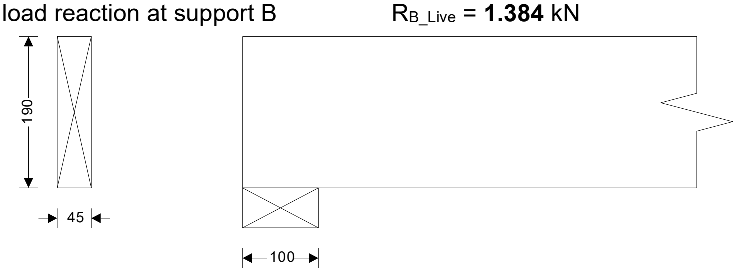
Load combinations

Load combination 1	Support A	Permanent × 1.20
		Live × 1.50
	Span 1	Permanent × 1.20
		Live × 1.50
	Support B	Permanent × 1.20
		Live × 1.50

Analysis results

Maximum moment M_{max} = **3.937** kNm M_{min} = **0.000** kNm

Design moment	$M^* = \max(\text{abs}(M_{max}),\text{abs}(M_{min}))$ = **3.937** kNm
Maximum shear	V_{max} = **3.841** kN V_{min} = **-3.841** kN
Design shear	$V^* = \max(\text{abs}(V_{max}),\text{abs}(V_{min}))$ = **3.841** kN
Total load on member	W_{tot} = **7.683** kN
Reactions at support A	R_{A_max} = **3.841** kN R_{A_min} = **3.841** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **1.471** kN
Unfactored live load reaction at support A	R_{A_Live} = **1.384** kN
Reactions at support B	R_{B_max} = **3.841** kN R_{B_min} = **3.841** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **1.471** kN
Unfactored live load reaction at support B	R_{B_Live} = **1.384** kN



Timber section details

Breadth of timber sections	b = **45** mm
Depth of timber sections	d = **190** mm
Number of timber sections in member	N = **1**
Overall breadth of timber member	$b_b = N \times b$ = **45** mm
Timber species	**Mixed softwood species (excl.Pinus species)**
Moisture condition	**Seasoned**
Timber strength grade - Table H3.1	**MGP10**

Member details

Load duration - cl.2.4.1	**Standard test**
Equilibrium moisture content	**15 %**
Length of span	L_{s1} = **4100** mm
Length of bearing	L_b = **100** mm
Number of discrete parallel systems	N_{mem} = **10**
Centre-to-centre spacing of discrete systems	s = **450** mm

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8550** mm^2
Section modulus	$Z_x = N \times b \times d^2 / 6$ = **270750** mm^3
	$Z_y = d \times (N \times b)^2 / 6$ = **64125** mm^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **25721250** mm^4
	$I_y = d \times (N \times b)^3 / 12$ = **1442812** mm^4
Radius of gyration	$r_x = \sqrt{(I_x / A)}$ = **54.8** mm
	$r_y = \sqrt{(I_y / A)}$ = **13.0** mm

Modification factors

Duration of load factor for strength - Table 2.3	k_1 = **1.00**
Moisture condition factor - cl.2.4.2.3	k_4 = **1.00**

Temperature factor - cl.2.4.3 k_6 = **1.00**

Length and position of bearing factor - Table 2.6 k_7 = **1.00**

Geometric factor appropriate to the number of members in a combined parallel system - Table 2.7

g_{31} = **1.00**

Geometric factor appropriate to the number of members in a discrete system - Table 2.7

g_{32} = **1.33**

Strength sharing factor (cl. 2.4.5.3) $k_9 = \max(1.0, g_{31} + (g_{32} - g_{31}) \times [1 - 2 \times s / L_x])$ = **1.26**

Temporary design action ratio r = **0.25**

Material constant - exp.E2(1) $\rho_b = 14.71 \times (E / f'_b)^{-0.480} \times r^{-0.061}$ = **0.73**

Distance between discrete lateral restraints L_{ay} = **0** mm $L_{ay} / d < 64 \times [N \times b / (\rho_b \times d)]^2$

Major axis slenderness coefficient - cl.3.2.3.2(b) S_1 = **0.00**

Major axis bending stability factor - exp.3.2(10) k_{12bx} = **1.00**

Minor axis slenderness coefficient - cl.3.2.3.2 (c) S_2 = **0.00**

Minor axis bending stability factor - cl.3.2.4 k_{12by} = **1.00**

Bearing strength - cl.3.2.6

Capacity factor - Table 2.1 ϕ_p = **0.9**

Bearing area for loading perpendicular to grain $A_p = N \times b \times L_b$ = **4500** mm²

Design capacity in bearing - exp.3.2(16) $\phi N_p = \phi_p \times k_1 \times k_4 \times k_6 \times k_7 \times f'_p \times A_p$ = **27.540** kN

PASS - Design capacity in bearing perpendicular to the grain exceeds design bearing load

Bending strength - cl.3.2.1

Capacity factor - Table 2.1 ϕ_b = **0.9**

Design capacity in bending - cl.3.2(2) $\phi M = \phi_b \times k_1 \times k_4 \times k_6 \times k_9 \times k_{12bx} \times f'_b \times Z_x$ = **4.903** kNm

PASS - Design capacity in bending exceeds design bending moment

Flexural shear strength - cl.3.2.5

Capacity factor - Table 2.1 ϕ_s = **0.9**

Shear plane area $A_s = N \times b \times d \times 2 / 3$ = **5700** mm²

Design shear capacity - exp.3.2(14) $\phi V = \phi_s \times k_1 \times k_4 \times k_6 \times f'_s \times A_s$ = **12.825** kN

PASS - Design shear capacity exceeds design shear force

Deflection - AS/NZS 1170.0

Deflection limit - Table C1 $\delta_{lim} = \mathrm{Min}(20\ mm, 0.004 \times L_{s1})$ = **16.400** mm

Deflection due to permanent load δ_G = **10.583** mm

Deflection due to imposed load δ_Q = **9.953** mm

Load factor - Table 4.1 ψ = **0.7**

Creep factor (Standard test) j_2 = **1.000**

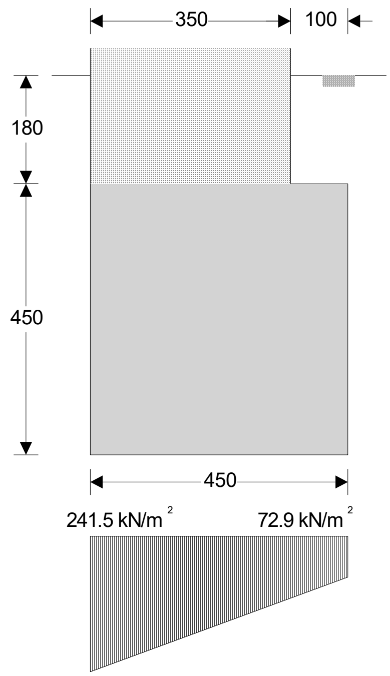
Total deflection $\delta_{tot} = j_2 \times (\delta_G + \psi \times \delta_Q)$ = **17.550** mm

FAIL - Total deflection exceeds the deflection limit

STRIP FOOTING ANALYSIS AND DESIGN (AS3600)

In accordance with AS3600-2018 incorporating Amendment No. 1

TEDDS calculation version 2.0.07



Strip footing details

Width of strip footing	B = **450** mm
Depth of strip footing	h = **450** mm
Depth of soil over strip footing	h_{soil} = **180** mm
Density of concrete	ρ_{conc} = **23.6** kN/m^3

Load details

Load width	b = **350** mm
Load eccentricity	e_P = **-50** mm

Soil details

Density of soil	ρ_{soil} = **20.0** kN/m^3
Design shear strength	ϕ' = **25.0** deg
Design base friction	δ = **19.3** deg
Ultimate design bearing capacity	$P_{bearing}$ = **250** kN/m^2

Load factors for stability

Dead load factor - stabilizing	γ_{sG} = **0.90**
Dead load factor - destabilizing	γ_{dG} = **1.35**
Imposed load factor - destabilizing	γ_{dQ} = **1.50**
Wind load factor - destabilizing	γ_{dW} = **1.00**

Axial loading on strip footing

Dead axial load	P_G = **20.0** kN/m

Imposed axial load P_Q = **20.0** kN/m

Wind axial load P_W = **0.0** kN/m

Total axial load $P = \gamma_{dG} \times P_G + \gamma_{dQ} \times P_Q + \gamma_{dW} \times P_W$ = **56.9** kN/m

Foundation loads

Dead surcharge load F_{Gsur} = **0.000** kN/m^2

Imposed surcharge load F_{Qsur} = **7.700** kN/m^2

Strip footing self weight $F_{swt} = h \times \rho_{conc}$ = **10.620** kN/m^2

Soil self weight $F_{soil} = h_{soil} \times \rho_{soil}$ = **3.600** kN/m^2

Total foundation load $F = [\gamma_{dG} \times (F_{Gsur} + F_{swt} + F_{soil}) + \gamma_{dQ} \times F_{Qsur}] \times B$ = **13.8** kN/m

Calculate base reaction

Total base reaction $T = F + P$ = **70.8** kN/m

Eccentricity of base reaction in x $e_T = (P \times e_P + M + H \times h) / T$ = **-40** mm

Base reaction acts within middle third of base

Calculate base pressures

$q_1 = (T / B) \times (1 - 6 \times e_T / B)$ = **241.541** kN/m^2

$q_2 = (T / B) \times (1 + 6 \times e_T / B)$ = **72.906** kN/m^2

Minimum base pressure $q_{min} = \min(q_1, q_2)$ = **72.906** kN/m^2

Maximum base pressure $q_{max} = \max(q_1, q_2)$ = **241.541** kN/m^2

Factor of safety for base pressure $F_{sb} = P_{bearing} / q_{max}$ = **1.035**

PASS - Maximum base pressure is less than ultimate design bearing capacity