Date: 26/7/21 19:09 PM
Designed By: Lovett Construction
Designer: Tim Lovett
Email: tlovett2007@gmail.com
Phone: 0447685925
29 Wyoming Ave
VALLEY HEIGHTS, New South Wales, 2777,
Australia

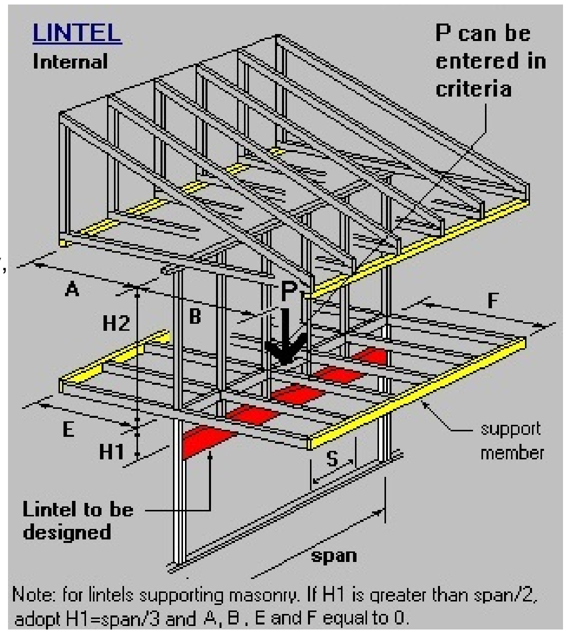
Single Span 150x50x2.5 RHS Steel RHS C350LO to AS/NZS 1163

Provide 10mm stiffeneres to steel beams at points of concentrated load and supports

Computations and certificates produced by SpanMan must be submitted to a building surveyor/certifier for approval of the member sizes prior to installation. Installation can only occur when a Building Permit has been granted.

Design Parameters

Country: Australia
Building type: House - domestic dwelling
Design working life: 50 years
Building Type Importance: 2 - Normal structures and structures not falling into other levels
Roof Use: Normal roof
Roof live load: 0.25 kPa, Roof live load: 1.1 kN
Floor Use: General eg living rooms, bedrooms, corridors, kitchen, toilets, study, rumpus
Floor live load: 1.5 kPa, Floor live load: 1.8 kN



Span = 3,000 mm
A = 3,600 mm (roof span left)
B = 450 mm (roof span right)
H1 = 0 mm (lower wall height)
H2 = 2,700 mm (upper wall height)
E = 3,600 mm (floor span)
F = 0 mm (floor span right)
S = 600 mm (stud spacing)

Roof weight(0.80 mm steel sheet) = 10 kg/m^2
Roof ceiling(13 mm plaster, pink batt insulation, wiring + sisalation + fittings) = 18 kg/m^2
Roof self weight(trusses/rafters) = 10.17 kg/m^2
H1 weight(10 mm plasterboard one side, pink batt insulation, 15 mm avg. weatherboards, wiring + sisalation + fittings) = 25 kg/m^2
H2 weight(10 mm plasterboard one side, pink batt insulation, 15 mm avg. weatherboards, wiring + sisalation + fittings) = 25 kg/m^2
Floor weight(19 mm particle board) = 13 kg/m^2
Floor ceiling(13 mm plaster, pink batt insulation, wiring + sisalation + fittings) = 18 kg/m^2
Floor self weight(joists) = 12.88 kg/m^2

The top of the Lintel is to be restrained by studs at 600 mm centres

Section Properties

Depth = 150 mm
Width = 50 mm
E = 200,000 MPa
A = 959 mm^2
I_{xx} = 2.54e6 mm^4
Z_{xx} = 43,500 mm^3

Dead Load

w(self weight) = 0.000959 x 7.9 x 9.81 = 0.0743 kN/m
w(dead load) = roof + walls + floor
= ((3.6/2 + 0.45/2)/cos(25)x(29.03 + 10.17) + (0x(25+ 25) + 2.7x(25 + 25)) + (3.6/2 + 0/2)x(31+12.88))x0.00981 = 2.959 kN/m
w(total dead load) = 0.0743 + 2.959 = 3.033 kN/m

Live Load

Imposed Loads
Q(UDL floor live load) = Floor load width x floor liveload = (3.6/2 + 0/2) x 1.5 = 2.7 kN/m

Q(Roof point live load) = 1.1 kN
Q(Interior point live load) = 1.8 kN
Q(Maximum point live load) = 1.8 kN

Designed By: Lovett Construction
Designer: Tim Lovett
Email: tlovett2007@gmail.com
Phone: 0447685925
29 Wyoming Ave
VALLEY HEIGHTS, New South Wales, 2777,
Australia

Q(Imposed point load) = 1.8 kN

Short-Term and Long-Term Live Loads
Q(Short-term UDL) = Imposed UDL x ψ_s = 2.7 x 0.7 = 1.89 kN/m
Q(Short-term point load) = Imposed point load x ψ_s = 1.8 x 1 = 1.8 kN
Q(Long-term UDL) = Imposed UDL x ψ_l = 2.7 x 0.4 = 1.08 kN/m
Q(Long-term point load) = Imposed point load x ψ_l = 1.8 x 0.4 = 0.72 kN

CALCULATIONS

Deflections, bending moments, shear forces and support reactions are calculated by the principles of structural analysis and match the output of any standard structural analysis software.

Where deflections, bending moments and shear forces are within 3% of allowable values they are marked in red.

(1) Deflection - Long-Term Dead Load
(+ downward deflection, - upward deflection)

w_1(long-term dead load) = 3.033 kN/m
w_1(long-term live load) = 1.08 kN/m

Deflection(1,500 mm from support) = 8.539 mm <= 9.167 (lesser 12 mm & span/360 ± 10%)

(2) Deflection - Short-Term Point Live Load
(+ downward deflection, - upward deflection)

$P_{1(live\ load)}$(1.5 m into span) = 1.8 kN

Deflection(1,500 mm from support) = 1.993 mm <= 13.2 (lesser 15 mm & span/250 ± 10%)

(3) Deflection - Short-Term UDL Midspan
(+ downward deflection, - upward deflection)

w_1(live load) = 1.89 kN/m

Deflection(1,500 mm from support) = 3.924 mm <= 13.2 (lesser 15 mm & span/250 ± 10%)

(4) Bending Strength - 1.35xDead Load Only

w_1(dead load) = 1.35x3.033 = 4.094 kN/m

Span Strength

Moment(span) = 4.606 kNm

l_e = 600 mm
α_m = 1
M_s(5.2.1) = $f_y Z_e$ = 350 x 43,500 x 1e-06 = 15.225 kNm
M_{oa}(5.6.1.1.(3)) = $[(\pi^2 EI_y / l_e^2)[GJ + (\pi^2 EI_w/l_e^2)]]^{0.5}$
= $[(\pi^2$ x 200,000 x 452,000 / $600^2)$ x [80,000 x 1.28e6 + (π^2 x 200,000 x 0/600^2)]]$^{0.5}$ * 1e-06 = 503.8 kNm
α_s(5.6.1.1(2)) = $0.6[((M_s/M_{oa})^2 + 3)^{0.5} - M_s/M_{oa}]$= 0.6 x $[((15.23/503.8)^2 + 3)^{0.5}$ - 15.23 / 503.8] = 1.021
M_b(AS4100 5.6.1.1(a)) = α_m x α_s x M_s = 1 x 1.021 x 15.23 = 15.55 kNm
M_s = 15.23 kNm

$\Phi^* M_b$ = 0.9 x 15.23 = 13.7 kNm >= 4.606 kNm

Reactions (+downward, -upward)

Maximum limit state reaction at x = 6.142 kN
Maximum limit state reaction at y = 6.142 kN

Designed By: Lovett Construction
Designer: Tim Lovett
Email: tlovett2007@gmail.com
Phone: 0447685925
29 Wyoming Ave
VALLEY HEIGHTS, New South Wales, 2777,
Australia

(5) Bending Strength - 1.2xDead Load + 1.5xImposed Point Live Load Span

w_1(dead load) = 1.2x3.033 = 3.64 kN/m
$P_{1(live\ load)}$(1.5 m into span) = 1.5x1.8 = 2.7 kN

Span Strength

Moment(span) = 6.119 kNm

l_e = 600 mm
α_m = 1
M_s(5.2.1) = $f_y Z_e$ = 350 x 43,500 x 1e-06 = 15.225 kNm
M_{oa}(5.6.1.1.(3)) = $[(\pi^2 EI_y / l_e^2)[GJ + (\pi^2 EI_w/l_e^2)]]^{0.5}$
= $[(\pi^2$ x 200,000 x 452,000 / $600^2)$ x [80,000 x 1.28e6 + (π^2 x 200,000 x 0/600^2)]$]^{0.5}$ * 1e-06 = 503.8 kNm
α_s(5.6.1.1(2)) = $0.6[((M_s/M_{oa})^2 + 3)^{0.5} - M_s/M_{oa}]$= 0.6 x $[((15.23/503.8)^2 + 3)^{0.5}$ - 15.23 / 503.8] = 1.021
M_b(AS4100 5.6.1.1(a)) = α_m x α_s x M_s = 1 x 1.021 x 15.23 = 15.55 kNm
M_s = 15.23 kNm

Φ* M_b = 0.9 x 15.23 = 13.7 kNm >= 6.119 kNm

Reactions (+downward, -upward)

Maximum limit state reaction at x = 6.809 kN
Maximum limit state reaction at y = 6.809 kN

(6) Bending Strength - 1.2xDead Load + 1.5xImposed Live Load UDL

w_1(dead load) = 1.2x3.033 = 3.64 kN/m
w_1(live load) = 1.5x2.7 = 4.05 kN/m

Span Strength

Moment(span) = 8.651 kNm

l_e = 600 mm
α_m = 1
M_s(5.2.1) = $f_y Z_e$ = 350 x 43,500 x 1e-06 = 15.225 kNm
M_{oa}(5.6.1.1.(3)) = $[(\pi^2 EI_y / l_e^2)[GJ + (\pi^2 EI_w/l_e^2)]]^{0.5}$
= $[(\pi^2$ x 200,000 x 452,000 / $600^2)$ x [80,000 x 1.28e6 + (π^2 x 200,000 x 0/600^2)]$]^{0.5}$ * 1e-06 = 503.8 kNm
α_s(5.6.1.1(2)) = $0.6[((M_s/M_{oa})^2 + 3)^{0.5} - M_s/M_{oa}]$= 0.6 x $[((15.23/503.8)^2 + 3)^{0.5}$ - 15.23 / 503.8] = 1.021
M_b(AS4100 5.6.1.1(a)) = α_m x α_s x M_s = 1 x 1.021 x 15.23 = 15.55 kNm
M_s = 15.23 kNm

Φ* M_b = 0.9 x 15.23 = 13.7 kNm >= 8.651 kNm

Reactions (+downward, -upward)

Maximum limit state reaction at x = 11.53 kN
Maximum limit state reaction at y = 11.53 kN

(7) Shear Strength - 1.35xDead Load Only

w_1(dead load) = 1.35x3.033 = 4.094 kN/m

Shear(at 0.001mm from x) = 6.142kN
Shear(at 0.001mm from y) = -6.142kN

Shear(maximum) = 6.142 kN

Shear(Design shear with AS 1684.1-1999(3.4.2.4) reduction) = 6.142 - 7x0.57 = 2.152 kN

$V^*(5.11.1) = \Phi V_v$
$V_v(5.11.2) = V_u$
d_p/t_w = 150 / 2.5 = 60
$82/(f_y/250)^{0.5}$ = 82 / $(350/250)^{0.5}$ = 69.3
$V_u(5.11.2(a)) = V_w$
$V_w(5.11.4) = 0.6f_yA_w$ = 0.6x350x725x0.001 = 152.3 kN

V*(5.11.1) = 0.9 x 152.3 = 137 kN >= 2.152 kN

Reactions (+downward, -upward)

Maximum limit state reaction at x = 6.142 kN
Maximum limit state reaction at y = 6.142 kN

(8) Shear Strength - 1.2xDead Load + 1.5xImposed Shear Point Live Load 1.001 mm from X

w_1(dead load) = 1.2x3.033 = 3.64 kN/m
$P_{1(live\ load)}$(0.001 m into span) = 1.5x1.8 = 2.7 kN

Shear(at 0.001mm from x) = 8.158kN
Shear(at 0.001mm from y) = -5.46kN

Shear(maximum) = 8.158 kN

Shear(Design shear with AS 1684.1-1999(3.4.2.4) reduction) = 8.158 - 7x0.8 = 2.558 kN

$V^*(5.11.1) = \Phi V_v$
$V_v(5.11.2) = V_u$
d_p/t_w = 150 / 2.5 = 60
$82/(f_y/250)^{0.5}$ = 82 / $(350/250)^{0.5}$ = 69.3
$V_u(5.11.2(a)) = V_w$
$V_w(5.11.4) = 0.6f_yA_w$ = 0.6x350x725x0.001 = 152.3 kN

V*(5.11.1) = 0.9 x 152.3 = 137 kN >= 2.558 kN

Reactions (+downward, -upward)

Maximum limit state reaction at x = 8.158 kN
Maximum limit state reaction at y = 5.46 kN

(9) Shear Strength - 1.2xDead Load + 1.5xImposed Live Load UDL

w_1(dead load) = 1.2x3.033 = 3.64 kN/m
w_1(live load) = 1.5x2.7 = 4.05 kN/m

Shear(at 0.001mm from x) = 11.53kN
Shear(at 0.001mm from y) = -11.53kN

Shear(maximum) = 11.53 kN

Shear(Design shear with AS 1684.1-1999(3.4.2.4) reduction) = 11.53 - 7x0.8 = 5.934 kN

$V^*(5.11.1) = \Phi V_v$
$V_v(5.11.2) = V_u$
d_p/t_w = 150 / 2.5 = 60
$82/(f_y/250)^{0.5}$ = 82 / $(350/250)^{0.5}$ = 69.3
$V_u(5.11.2(a)) = V_w$
$V_w(5.11.4) = 0.6f_yA_w$ = 0.6x350x725x0.001 = 152.3 kN

V*(5.11.1) = 0.9 x 152.3 = 137 kN >= 5.934 kN

Designed By: Lovett Construction
Designer: Tim Lovett
Email: tlovett2007@gmail.com
Phone: 0447685925
29 Wyoming Ave
VALLEY HEIGHTS, New South Wales, 2777,
Australia

Reactions (+downward, -upward)

Maximum limit state reaction at x = 11.53 kN
Maximum limit state reaction at y = 11.53 kN