Design Report

Proposed House, 45 Hangahai Drive, Flat Bush

TIMBER beam								a
Consider a simply support floor beam spanning =						3 m		
	Loading							
Dead load(udl)	G=			0.672	kN/m	(wall load & roof load)		
Live load(udl)	Q=			0	kN/m			
Load combinations from AS1170.0								
Strength limit state								
1.35G=		0.9072	kN/m					
1.2G+1.5Q=		0.8064	kN/m					
Serviceability limit state					where			
G+ Ψs Q =		0.672	kN/m		short term deflection		Ψs =	0.7
G+ Ψl Q =		0.672	kN/m		long term deflection		Ψl =	0.4

Try section	2 sections				
	d=		190	mm	
	b=	2*45	90	mm	

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:					
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber	
ϕ=		0.8			
k1=		0.6		for permanent load or	
k4=		1.14		0.8	for medium load
k5=		1			
Lay=		400	mm	distance b/w restraints	
S= slender coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.89			
k8=		1		from table 2.8	
fb		27.7		N/mm^2	
Z=bd^2/6		541500	mm^3		
ϕMn long=		8.21	kNm	long term loading	
ϕMn med=		10.95	kNm	short term loading	

Compare with design load

M*(1.35G) =	1.03	kNm	<	8.21	kNm	OK
M*(1.2G+1.5Q) =	0.91	kNm	<	10.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		11400	mm^2
ϕVn long		23.71	kN
ϕVn med		31.61	kN

Compare with design load

V*(1.35G) =	1.37	kN	<	23.71	kN	OK
V*(1.2G+1.5Q) =	1.21	kN	<	31.61	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	1.37	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	1.21	kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	1.32	mm				
Δ_{Q}	0	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =	1.32	mm				
$\Delta_{k2(G+\Psi lQ)}$ =	2.63	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		1.49	mm	span/400=	7.5	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		2.98	mm	span/250=	12	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam b

Consider a simply support floor beam spanning = 1.2 m

Loading

Dead load(udl)	G=		0.672	kN/m	(wall load & roof load)	
Live load(udl)	Q=		0	kN/m		
Load combinations from AS1170.0						
Strength limit state						
1.35G=	0.9072	kN/m				
1.2G+1.5Q=	0.8064	kN/m				
Serviceability limit state			where			
G+ Ψs Q =	0.672	kN/m	short term deflection	Ψs =		0.7
G+ Ψl Q =	0.672	kN/m	long term deflection	Ψl =		0.4

Try section	2 sections			
	d=		140	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:

ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender				
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.11		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		294000	mm^3	
ϕMn long=		4.46	kNm	long term loading
ϕMn med=		5.95	kNm	short term loading

Compare with design load

M*(1.35G) =	0.17	kNm	<	4.46	kNm	OK
M*(1.2G+1.5Q) =	0.15	kNm	<	5.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:

ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		8400	mm^2
ϕVn long		17.47	kN
ϕVn med		23.29	kN

Compare with design load

V*(1.35G) =	0.55	kN	<	17.47	kN	OK
V*(1.2G+1.5Q) =	0.49	kN	<	23.29	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	0.55	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	0.49	kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	0.09	mm				
Δ_{Q}	0	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\psi sQ}$ =	0.09	mm				
$\Delta_{k2(G+\psi lQ)}$ =	0.17	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\psi sQ}$ =		0.1	mm	span/400=	3	mm	OK
$\Delta_{k2(G+\psi lQ)}$ =		0.2	mm	span/250=	4.8	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

Timber Beam c

Consider a simply support floor beam spanning =				3	m	
Loading						
Dead load(udl)	G=		3.87	kN/m	(wall load,roof load&floor load)	
Live load(udl)	Q=		3.89	kN/m		
Load combinations from AS1170.0						
Strength limit state						
1.35G=	5.2245	kN/m				
1.2G+1.5Q=	10.479	kN/m				
Serviceability limit state				where		
G+ Ψs Q =	6.593	kN/m		short term deflection	Ψs =	0.7
G+ Ψl Q =	5.426	kN/m		long term deflection	Ψl =	0.4

Try section	2 sections			
	d=		240	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:				
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	4.48		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		864000	mm^3	
ϕMn long=		13.1	kNm	long term loading
ϕMn med=		17.47	kNm	short term loading

Compare with design load

M*(1.35G) =	5.88	kNm	<	13.1	kNm	OK
M*(1.2G+1.5Q) =	11.79	kNm	<	17.47	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		14400	mm^2
ϕVn long		29.95	kN
ϕVn med		39.93	kN

Compare with design load

V*(1.35G) =	7.84	kN	<	29.95	kN	OK
V*(1.2G+1.5Q) =	15.72	kN	<	39.93	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	7.84	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	15.72	kN	<	49.99	kN	OK

Check Serviceability design

E=		10.5	Gpa	For SG8, table 2.3			
Elb=		8	Gpa	lower bound E, table2.3			
Δ_{G}		3.75	mm				
Δ_{Q}		3.77	mm				
k_2		2		creep factor for solid timber, table 2.5			
$\Delta_{G+\psi sQ}$ =		6.39	mm				
$\Delta_{k2(G+\psi lQ)}$ =		10.52	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\psi sQ}$ =		7.26	mm	span/400=	7.5	mm	OK
$\Delta_{k2(G+\psi lQ)}$ =		11.94	mm	span/250=	12	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam						d
Consider a simply support floor beam spanning =			2.2 m			
Loading						
Dead load(udl)	G=		3.17 kN/m	(wall,roof&floor load)		
Live load(udl)	Q=		0.4 kN/m			
Load combinations from AS1170.0						
Strength limit state						
1.35G=		4.2795 kN/m				
1.2G+1.5Q=		4.404 kN/m				
Serviceability limit state			where			
G+ Ψs Q =		3.45 kN/m	short term deflection	Ψs =		0.7
G+ Ψl Q =		3.33 kN/m	long term deflection	Ψl =		0.4

Try section	2 sections			
	d=		240	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:				
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender				
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	4.48		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		864000	mm^3	
ϕMn long=		13.1	kNm	long term loading
ϕMn med=		17.47	kNm	short term loading

Compare with design load

M*(1.35G) =	2.59 kNm	<	13.1	kNm	OK
M*(1.2G+1.5Q) =	2.67 kNm	<	17.47	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		14400	mm^2
ϕVn long		29.95	kN
ϕVn med		39.93	kN

Compare with design load

V*(1.35G) =	4.71 kN	<	29.95	kN	OK
V*(1.2G+1.5Q) =	4.85 kN	<	39.93	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	4.71	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	4.85	kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	0.89	mm				
Δ_{Q}	0.12	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =	0.97	mm				
$\Delta_{k2(G+\Psi lQ)}$ =	1.87	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		1.1	mm	span/400=	5.5	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		2.12	mm	span/250=	8.8	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam

Consider a simply support floor beam spanning =			2.2 m			
Loading						
Dead load(udl)	G=		3.77	kN/m	(wall ,roof&floor load)	
Live load(udl)	Q=		3.38	kN/m		
Load combinations from AS1170.0						
Strength limit state						
1.35G=		5.0895 kN/m				
1.2G+1.5Q=		9.594 kN/m				
Serviceability limit state				where		
G+ Ψs Q =		6.136 kN/m		short term deflection	Ψs =	0.7
G+ Ψl Q =		5.122 kN/m		long term deflection	Ψl =	0.4

Try section	2 sections			
	d=		240	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:				
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	4.48		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		864000	mm^3	
ϕMn long=		13.1	kNm	long term loading
ϕMn med=		17.47	kNm	short term loading

Compare with design load

M*(1.35G) =	3.08 kNm	<	13.1	kNm	OK
M*(1.2G+1.5Q) =	5.81 kNm	<	17.47	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		14400	mm^2
ϕVn long		29.95	kN
ϕVn med		39.93	kN

Compare with design load

V*(1.35G) =	5.6 kN	<	29.95	kN	OK
V*(1.2G+1.5Q) =	10.56 kN	<	39.93	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	5.6	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	10.56	kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	1.06	mm				
Δ_{Q}	0.95	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\psi sQ}$ =	1.72	mm				
$\Delta_{k2(G+\psi lQ)}$ =	2.88	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\psi sQ}$ =		1.96	mm	span/400=	5.5	mm	OK
$\Delta_{k2(G+\psi lQ)}$ =		3.26	mm	span/250=	8.8	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam f

Consider a simply support floor beam spanning = 1 m

Loading

Dead load(udl)	G=	1.012	kN/m	(wall ,roof load&floor load)		
Live load(udl)	Q=	1.7	kN/m			
Load combinations from AS1170.0						
Strength limit state						
1.35G=		1.3662	kN/m			
1.2G+1.5Q=		3.7644	kN/m			
Serviceability limit state				where		
G+ Ψs Q =		2.202	kN/m	short term deflection	Ψs =	0.7
G+ Ψl Q =		1.692	kN/m	long term deflection	Ψl =	0.4

Try section	2 sections			
	d=		140	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:

ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender				
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.11		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		294000	mm^3	
ϕMn long=		4.46	kNm	long term loading
ϕMn med=		5.95	kNm	short term loading

Compare with design load

M*(1.35G) =	0.18	kNm	<	4.46	kNm	OK
M*(1.2G+1.5Q) =	0.48	kNm	<	5.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:

ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		8400	mm^2
ϕVn long		17.47	kN
ϕVn med		23.29	kN

Compare with design load

V*(1.35G) =	0.69	kN	<	17.47	kN	OK
V*(1.2G+1.5Q) =	1.89	kN	<	23.29	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	75	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			6750		mm^2	
ϕNnbp long			37.49		kN	
ϕNnbp med			49.99		kN	
N*(1.35G) =	0.69	kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	1.89	kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	0.07	mm				
Δ_{Q}	0.11	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =	0.14	mm				
$\Delta_{k2(G+\Psi lQ)}$ =	0.21	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		0.16	mm	span/400=	2.5	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		0.24	mm	span/250=	4	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam j

Consider a simply support floor beam spanning = 2.6 m

Loading

Dead load(udl)	G=	0.86	kN/m	(wall load,roof load)	
Live load(udl)	Q=	1.3	kN/m		

Load combinations from AS1170.0

Strength limit state

1.35G=	1.161	kN/m			
1.2G+1.5Q=	2.982	kN/m			
Serviceability limit state			where		
G+ Ψs Q =	1.77	kN/m	short term deflection	Ψs =	0.7
G+ Ψl Q =	1.38	kN/m	long term deflection	Ψl =	0.4

Try section 2 sections

d=		190	mm	
b=	2*45	90	mm	

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:

ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber
ϕ=		0.8		
k1=		0.6		for permanent load or
k4=		1.14		0.8 for medium load
k5=		1		
Lay=		400	mm	distance b/w restraints
S= slender				
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.89		
k8=		1		from table 2.8
fb		27.7		N/mm^2
Z=bd^2/6		541500	mm^3	
ϕMn long=		8.21	kNm	long term loading
ϕMn med=		10.95	kNm	short term loading

Compare with design load

M*(1.35G) =	0.99	kNm	<	8.21	kNm	OK
M*(1.2G+1.5Q) =	2.52	kNm	<	10.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:

ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		11400	mm^2
ϕVn long		23.71	kN
ϕVn med		31.61	kN

Compare with design load

V*(1.35G) =	1.51	kN	<	23.71	kN	OK
V*(1.2G+1.5Q) =	3.88	kN	<	31.61	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length= 75 mm (see table 2.6, cl 2.8, NZS 3603-1993)

Design strength:

ϕNnbp=	Øx k1x k3x fpx Ap				
k1=			from above		
k3=			1.3	(table 2.6, cl 2.8)	
fp=			8.9	N/mm^2	
Ap=			6750	mm^2	
ϕNnbp long			37.49	kN	
ϕNnbp med			49.99	kN	
N*(1.35G) =	1.51 kN	<	37.49	kN	OK
N*(1.2G+1.5Q) =	3.88 kN	<	49.99	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	0.95	mm				
Δ_{Q}	1.44	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =	1.95	mm				
$\Delta_{k2(G+\Psi lQ)}$ =	3.05	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		2.22	mm	span/400=	6.5	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		3.46	mm	span/250=	10.4	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam							k	
Consider a simply support floor beam spanning =					2	m		
	Loading							
Dead load(udl)	G=				0.08	kN/m	(wall load,roof load)	
Live load(udl)	Q=				0.4	kN/m		
Load combinations from AS1170.0								
Strength limit state								
1.35G=			0.108	kN/m				
1.2G+1.5Q=			0.696	kN/m				
Serviceability limit state						where		
G+ Ψs Q =			0.36	kN/m		short term deflection	Ψs =	0.7
G+ Ψl Q =			0.24	kN/m		long term deflection	Ψl =	0.4

Try section	2 sections			
	d=		140	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:					
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber	
ϕ=		0.8			
k1=		0.6		for permanent load or	
k4=		1.14		0.8	for medium load
k5=		1			
Lay=		400	mm	distance b/w restraints	
S= slender					
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.11			
k8=		1		from table 2.8	
fb		27.7		N/mm^2	
Z=bd^2/6		294000	mm^3		
ϕMn long=		4.46	kNm	long term loading	
ϕMn med=		5.95	kNm	short term loading	

Compare with design load

M*(1.35G) =	0.06	kNm	<	4.46	kNm	OK
M*(1.2G+1.5Q) =	0.35	kNm	<	5.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		8400	mm^2
ϕVn long		17.47	kN
ϕVn med		23.29	kN

Compare with design load

V*(1.35G) =	0.11	kN	<	17.47	kN	OK
V*(1.2G+1.5Q) =	0.7	kN	<	23.29	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	50	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			4500		mm^2	
ϕNnbp long			25		kN	
ϕNnbp med			33.33		kN	
N*(1.35G) =	0.11	kN	<	25	kN	OK
N*(1.2G+1.5Q) =	0.7	kN	<	33.33	kN	OK

Check Serviceability design

E=		10.5	Gpa	For SG8, table 2.3			
Elb=		8	Gpa	lower bound E, table2.3			
Δ_{G}		0.08	mm				
Δ_{Q}		0.39	mm				
k_2		2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =		0.35	mm				
$\Delta_{k2(G+\Psi lQ)}$ =		0.47	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		0.4	mm	span/400=	5	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		0.53	mm	span/250=	8	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

timber beam L

Consider a simply support floor beam spanning =			1.3	m	

Loading

Dead load(udl)	G=		0.42	kN/m	(wall load,roof load)	
Live load(udl)	Q=		2.12	kN/m		
Load combinations from AS1170.0						
Strength limit state						
1.35G=	0.567	kN/m				
1.2G+1.5Q=	3.684	kN/m				
Serviceability limit state			where			
G+ Ψs Q =	1.904	kN/m	short term deflection	Ψs =		0.7
G+ Ψl Q =	1.268	kN/m	long term deflection	Ψl =		0.4

Try section	2 sections			
	d=		140	mm
	b=	2*45	90	mm

Check Bending strength(NZS 3603, cl 3.2.4.)

Design strength:					
ϕMn=	Øxk1xk4xk5xk8xfbxZ			for sawn timber	
ϕ=		0.8			
k1=		0.6		for permanent load or	
k4=		1.14		0.8	for medium load
k5=		1			
Lay=		400	mm	distance b/w restraints	
S= slender					
coefficient	1.35(Lay/b((d/b)^2 -1)^0.5)^0.5=	3.11			
k8=		1		from table 2.8	
fb		27.7		N/mm^2	
Z=bd^2/6		294000	mm^3		
ϕMn long=		4.46	kNm	long term loading	
ϕMn med=		5.95	kNm	short term loading	

Compare with design load

M*(1.35G) =	0.12	kNm	<	4.46	kNm	OK
M*(1.2G+1.5Q) =	0.78	kNm	<	5.95	kNm	OK

Check Shear strength(NZS 3603, cl 3.2.3)

Design strength:			
ϕVn=	Øxk1xk4xk5xfsxAs		
ϕ,k1,k4,k5		same as above	
fs=	(table 2.2 - NZS 3603)	3.8	N/mm^2
As=(2/3*bd)		8400	mm^2
ϕVn long		17.47	kN
ϕVn med		23.29	kN

Compare with design load

V*(1.35G) =	0.37	kN	<	17.47	kN	OK
V*(1.2G+1.5Q) =	2.4	kN	<	23.29	kN	OK

Check Bearing strength(NZS 3603 3.2.9)

assuming bearing length=	50	mm	(see table 2.6, cl 2.8, NZS 3603-1993)			
Design strength:						
ϕNnbp=	Øx k1x k3x fpx Ap					
k1=			from above			
k3=			1.3	(table 2.6, cl 2.8)		
fp=			8.9		N/mm^2	
Ap=			4500		mm^2	
ϕNnbp long			25		kN	
ϕNnbp med			33.33		kN	
N*(1.35G) =	0.37	kN	<	25	kN	OK
N*(1.2G+1.5Q) =	2.4	kN	<	33.33	kN	OK

Check Serviceability design

E=	10.5	Gpa	For SG8, table 2.3			
Elb=	8	Gpa	lower bound E, table2.3			
Δ_{G}	0.08	mm				
Δ_{Q}	0.37	mm				
k_2	2		creep factor for solid timber, table 2.5			
$\Delta_{G+\Psi sQ}$ =	0.33	mm				
$\Delta_{k2(G+\Psi lQ)}$ =	0.44	mm				

Its is possible that timber members may have a lower stiffness than the avg, from NZS 3603, 2.34.2.3

$E_{(modified)}$ =	(E+Elb)/2	9.25	Gpa				
therefore,							
$\Delta_{G+\Psi sQ}$ =		0.38	mm	span/400=	3.25	mm	OK
$\Delta_{k2(G+\Psi lQ)}$ =		0.5	mm	span/250=	5.2	mm	OK

(Refer to AS/NZS 1170.0, table C1 for suggested serviceability limits)

Check internal balustrade post calculations

HorizontAL Lateral Load =		0.35	kN/m
For Timber Balustrade post height at =		1	m
Timber balustrade c/c spacing =		0.6	m

M* = 0.35 x 1.5 x 1 x 0.6
= 0.315 kN.m

Try, SG8 studs = 90 x 45

ØM = Øxk1xk4xk5xk8xfbxZ

Where,

Ø =	0.8	
k1 =	0.8	
k4 =	1	
k5 =	1	
k8 =	1	
fb =	14	N/mm2 (Table 1. NZS 3604)

= 0.8 x 0.8 x 1 x 1 x 1 x 14 x (90^2 x 45/6)
= 0.544 kN.m **Hence Ok**

Check Fixing = 0.315/0.09
= 3.5 kN

Provide sheet brace strap and GIB handibrac bracket which can take 4.2kN OK

Check fixing resistant to the boundary joists
0.315/0.24= 1.3 Kn/m top and bottom

The top to be resisted by CPC80 and the resisted by Lumberlok 6kN strap> 1.3kN ok.

Glass balustrade
Check fixing resistant to the boundary joists
M = 0.35x1.5x1x0.6 = 0.315kN.m
Check fixing resistant to the boundary joists
0.315/0.24= 1.3 Kn/m top and bottom

The top to be resisted by CPC80 and the resisted by Lumberlok 6kN strap> 1.3kN ok.

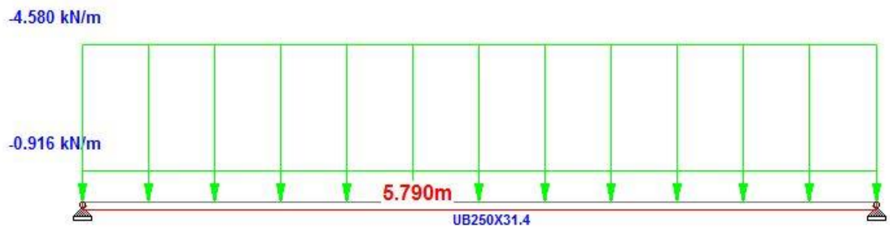
Check balustrade fix to the deck joists structure to resist the lateral load
Deck joists at 400mm c/c

Balustrade moment M = 0.75x1.5x1.0.4m = 0.45kNm

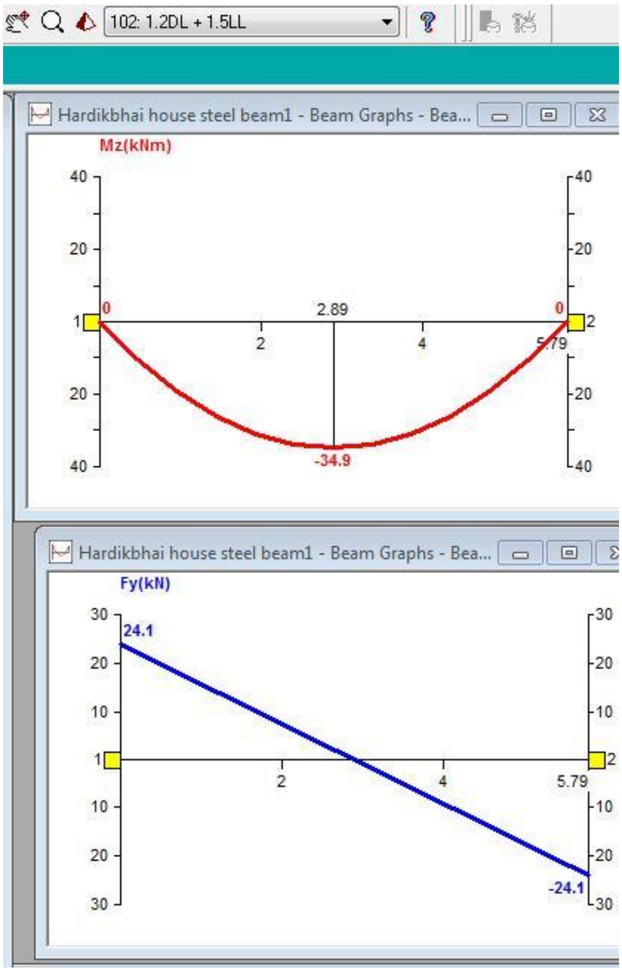
Check fixing
0.45/0.19 = 2.4kN
Provide sheet brace strap brace at top and bottom which can take 0.7x6kN =4.2kN, OK

STEEL BEAMS CALCULATIONS:

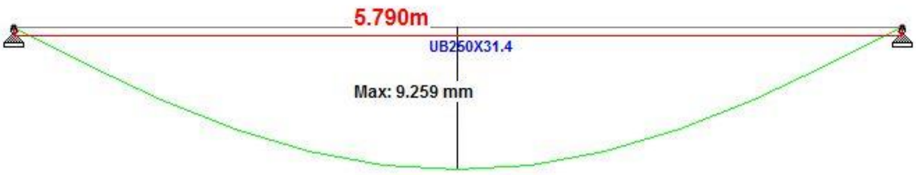
Steel beam B1



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL)



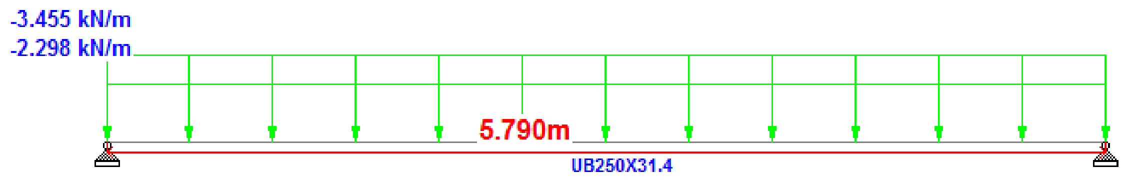
Deflection(DL+LL)

Member Design – B1

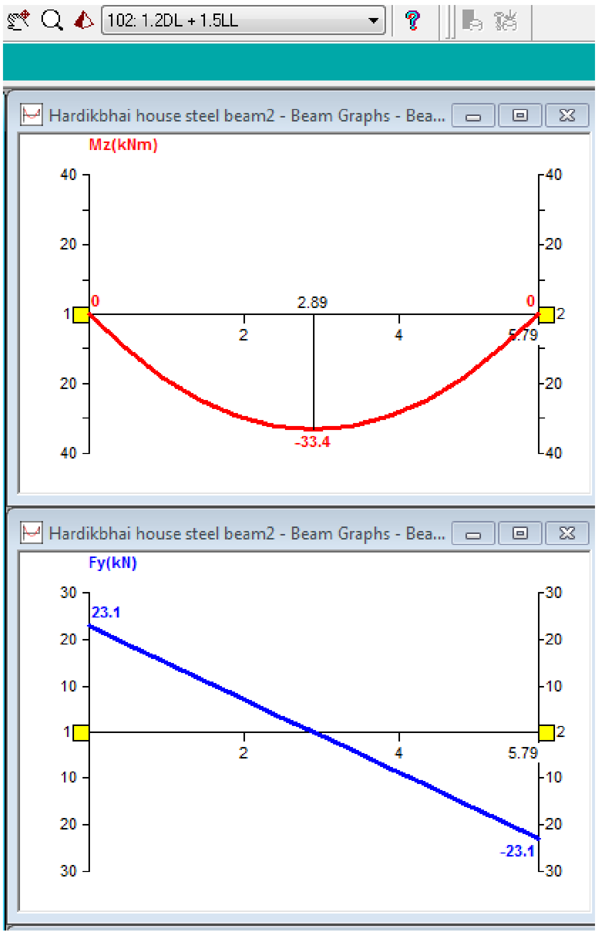
```
MemDes  Calculations  @  13:54:28 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B1

Section : 250UB31  Grade 300+
   d = 252 mm          b = 146 mm        tf =  8.6 mm         tw =  6.1 mm
Area = 4010 mm2     fyf = 320 MPa      fyw = 320 MPa         fu = 440 MPa
 Ixx = 44.50 E6mm4  Sx = 397.00 E3mm3  Zx = 354.00 E3mm3   rx = 105.00 mm
 Iyy =  4.47 E6mm4  Sy = 94.20 E3mm3  Zy = 61.20 E3mm3   ry = 33.40 mm
  Iw = 65.90 E9mm6   J = 89.30 E3mm4    kf = 1.000      Manufact. Type = HR
   Compactness(x,y) = N, N
 Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  9.20,    Allowable = 10.00      [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202 ]
    Web   :  le : Actual = 43.55,    Allowable = 101.00      [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr   = 5.79 * 1.00 * 1.00 * 1.00 = 5.79m

Major Axis Bending
    Design Action M*x =  35.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.40 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5          [Eq 5.6.1.1(2)]
      = 1.7 *   35.0 / [(  18.0)2 + (  35.0)2 + (  18.0)2 ]0.5 = 1.375
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 5.7902 =   60.4 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo =  54.8 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.35     [ (Ms/Mo) = 126.4 /  54.8 = 2.308 ]
     am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.37 * 0.35 *  126.4 =   60.2
  Major axis capacity Ratio = M*x   /  f Mbx
                           = 0.65,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  24.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
    Nominal Shear Yield capacity  Vw = 295.1 kN
      av = 3.55 >= 1.0 => full web shear capacity
     Vu = Vw = 295.1 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                          = 0.09,              ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.48,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================     SUMMARY     =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.65    ---- OK ----
```

Deflection allowable = span /400= 5790/400= 14.475 (as per Table C1 ,AS NZS 1170.0-2002) OK

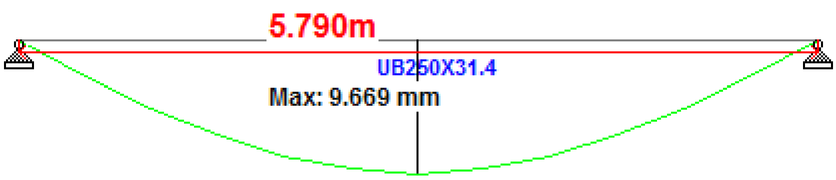
STEEL BEAM B-2



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL)



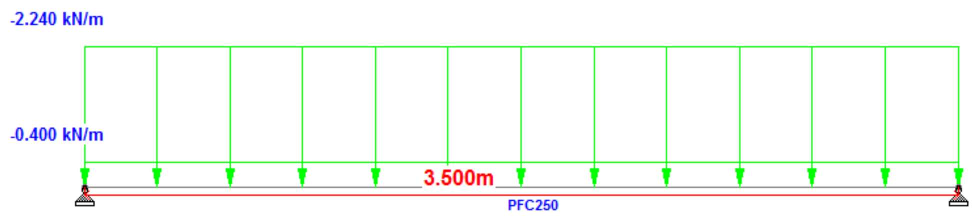
Deflection(DL+LL)

Member Design B2

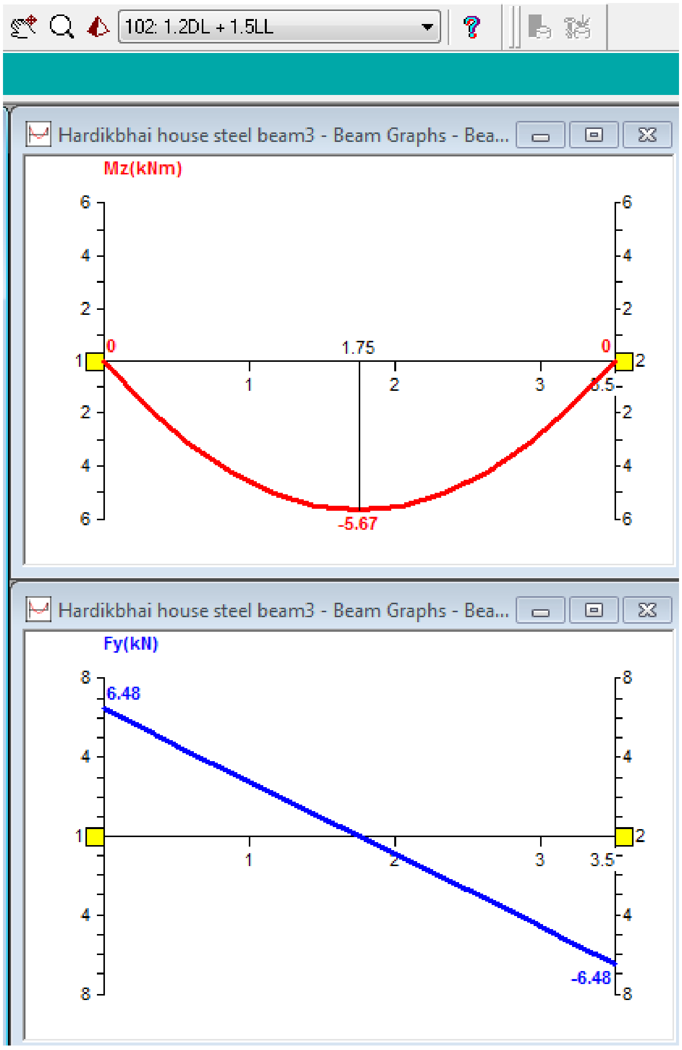
```
MemDes  Calculations  @  13:57:55 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B2

Section : 250UB31  Grade 300+
   d = 252 mm        b = 146 mm       tf =  8.6 mm        tw =  6.1 mm
Area = 4010 mm2    fyf = 320 MPa     fyw = 320 MPa       fu = 440 MPa
 Ixx = 44.50 E6mm4  Sx = 397.00 E3mm3  Zx = 354.00 E3mm3   rx = 105.00 mm
 Iyy =  4.47 E6mm4  Sy = 94.20 E3mm3  Zy = 61.20 E3mm3   ry = 33.40 mm
  Iw = 65.90 E9mm6   J = 89.30 E3mm4    kf = 1.000      Manufact. Type = HR
   Compactness(x,y) = N, N
 Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  9.20,    Allowable = 10.00     [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202 ]
    Web   :  le : Actual = 43.55,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 5.79 * 1.00 * 1.00 * 1.00 = 5.79m

Major Axis Bending
    Design Action M*x =  34.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.40 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5        [Eq 5.6.1.1(2)]
      = 1.7 *   34.0 / [(  16.0)2 + (  34.0)2 + (  16.0)2 ]0.5 = 1.415
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2   = 3.1422 x   205 E6 / 5.7902 =  60.4 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo =  54.8 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.35     [ (Ms/Mo) = 126.4 /  54.8 = 2.308 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.42 * 0.35 *  126.4 =    62.0
  Major axis capacity Ratio = M*x  /  f Mbx
                            = 0.61,             ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  23.1 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
    Nominal Shear Yield capacity  Vw = 295.1 kN
      av = 3.55 >= 1.0 => full web shear capacity
     Vu = Vw = 295.1 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.09,              ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.49,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================    SUMMARY    =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.61    ---- OK ----
```

Deflection allowable = span /400= 5790/400= 14.475mm (as per Table C1 ,AS NZS 1170.0-2002) OK

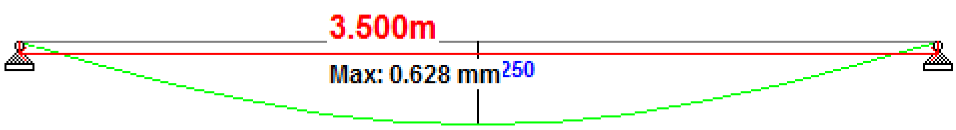
STEEL BEAM B3



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL)



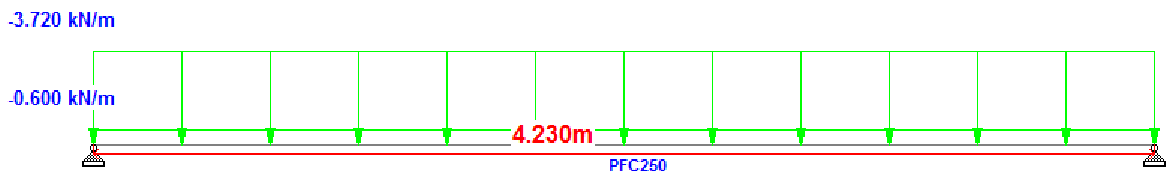
Deflection(DL+LL)

Member design B3

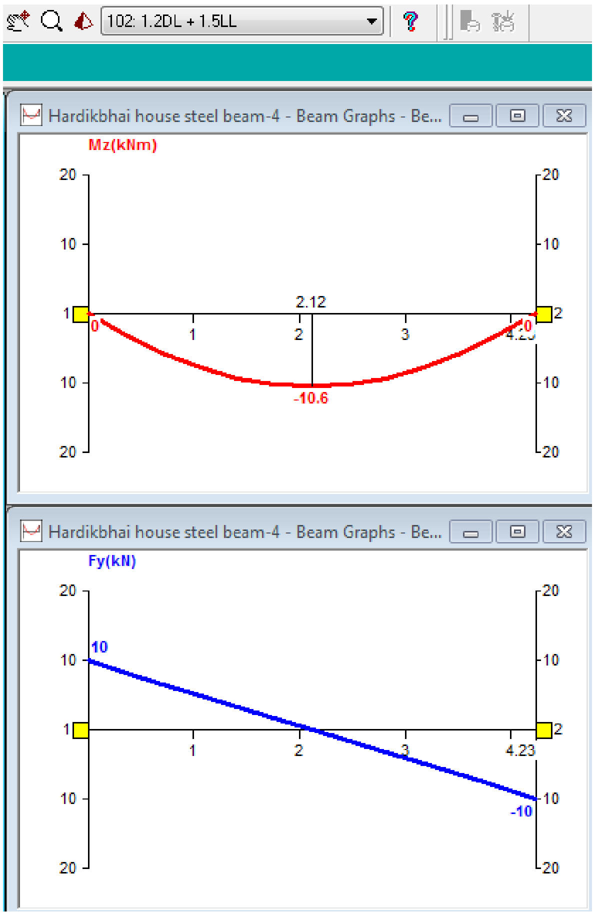
```
MemDes  Calculations  @  12:16:23 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B3

Section : 250PFC  Grade 300+
   d = 250 mm         b =  90 mm        tf = 15.0 mm        tw =  8.0 mm
Area = 4520 mm2    fyf = 300 MPa      fyw = 320 MPa         fu = 440 MPa
 Ixx = 45.10 E6mm4  Sx = 421.00 E3mm3  Zx = 361.00 E3mm3   rx = 99.90 mm
 Iyy =  3.64 E6mm4  Sy = 107.00 E3mm3  Zy = 59.30 E3mm3   ry = 28.40 mm
  Iw = 35.90 E9mm6   J =238.00 E3mm4    kf = 1.000      Manufact. Type = HR
 Zex = 421.00 E3mm3 Zey = 88.70 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  5.99,    Allowable = 10.00      [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.082 / 0.015 * 1.095 = 5.988 ]
    Web   :  le : Actual = 31.11,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.220 / 0.008 * 1.131 = 31.113 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                 => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 3.50 * 1.00 * 1.00 * 1.00 = 3.50m

Major Axis Bending
    Design Action M*x =   6.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.30 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5        [Eq 5.6.1.1(2)]
      = 1.7 *    6.0 / [(   3.0)2 + (   6.0)2 + (   3.0)2 ]0.5 = 1.388
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 3.5002 = 165.2 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo = 122.5 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.59     [ (Ms/Mo) = 126.3 / 122.5 = 1.031 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.39 * 0.59 *  126.3 =  103.6
  Major axis capacity Ratio = M*x   /  f Mbx
                            = 0.06,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =   7.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.250 * 0.008
    Nominal Shear Yield capacity  Vw = 384.0 kN
      av = 6.09 >= 1.0 => full web shear capacity
     Vu = Vw = 384.0 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.02,               ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.82,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 =======================     SUMMARY     =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.06     ---- OK ----
```

Deflection allowable = span /400= 3500/400= 8.75mm (as per Table C1 ,AS NZS 1170.0-2002) OK

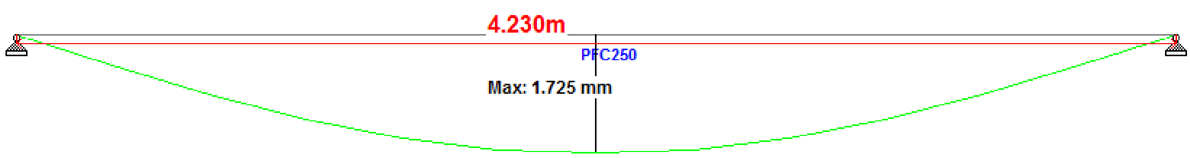
STEEL BEAM B4



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL)



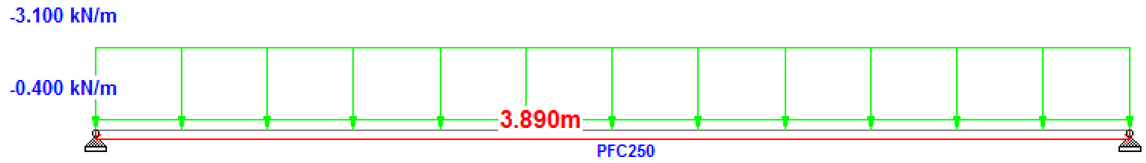
Deflection(DL+LL)

Member design B4

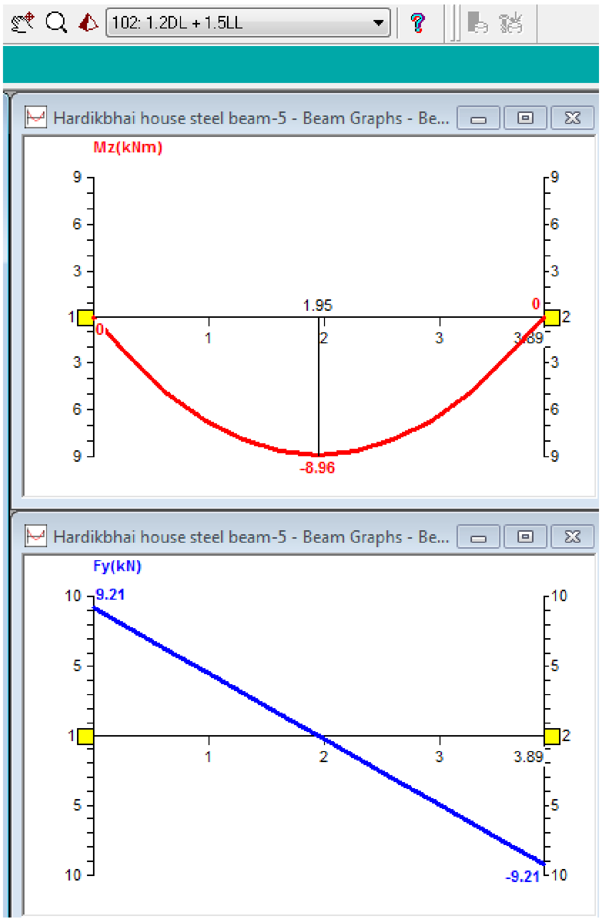
```
MemDes  Calculations  @  12:20:34 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B4

Section : 250PFC  Grade 300+
   d = 250 mm          b =  90 mm        tf = 15.0 mm        tw =  8.0 mm
Area = 4520 mm2    fyf = 300 MPa      fyw = 320 MPa        fu = 440 MPa
 Ixx = 45.10 E6mm4  Sx = 421.00 E3mm3  Zx = 361.00 E3mm3   rx = 99.90 mm
 Iyy =  3.64 E6mm4  Sy = 107.00 E3mm3  Zy = 59.30 E3mm3   ry = 28.40 mm
  Iw = 35.90 E9mm6   J =238.00 E3mm4    kf = 1.000      Manufact. Type = HR
 Zex = 421.00 E3mm3 Zey = 88.70 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  5.99,    Allowable = 10.00     [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.082 / 0.015 * 1.095 = 5.988 ]
    Web    :  le : Actual = 31.11,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.220 / 0.008 * 1.131 = 31.113 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                 => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 4.23 * 1.00 * 1.00 * 1.00 = 4.23m

Major Axis Bending
    Design Action M*x =  11.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.30 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5        [Eq 5.6.1.1(2)]
      = 1.7 *   11.0 / [(   6.0)2 + (  11.0)2 + (   6.0)2 ]0.5 = 1.346
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2   = 3.1422 x   205 E6 / 4.2302 = 113.1 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo =  97.5 kNm         ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.52     [ (Ms/Mo) = 126.3 /  97.5 = 1.295 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.35 * 0.52 *  126.3 =   88.5
  Major axis capacity Ratio = M*x  /  f Mbx
                           = 0.14,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  10.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.250 * 0.008
    Nominal Shear Yield capacity  Vw = 384.0 kN
      av = 6.09 >= 1.0 => full web shear capacity
     Vu = Vw = 384.0 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                          = 0.03,              ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.70,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================      SUMMARY      ====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.14    ---- OK ----
```

Deflection allowable = span /400= 4230/400= 10.575 mm (as per Table C1, AS NZS 1170.0-2002) OK

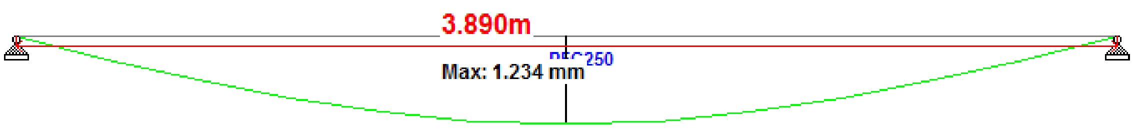
STEEL BEAM B5



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL



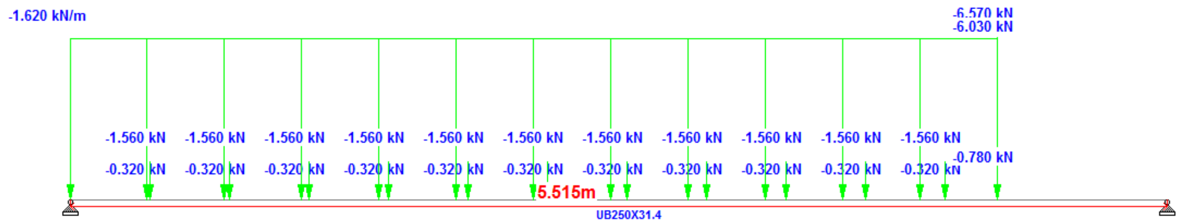
Deflection(DL+LL)

Member design B5

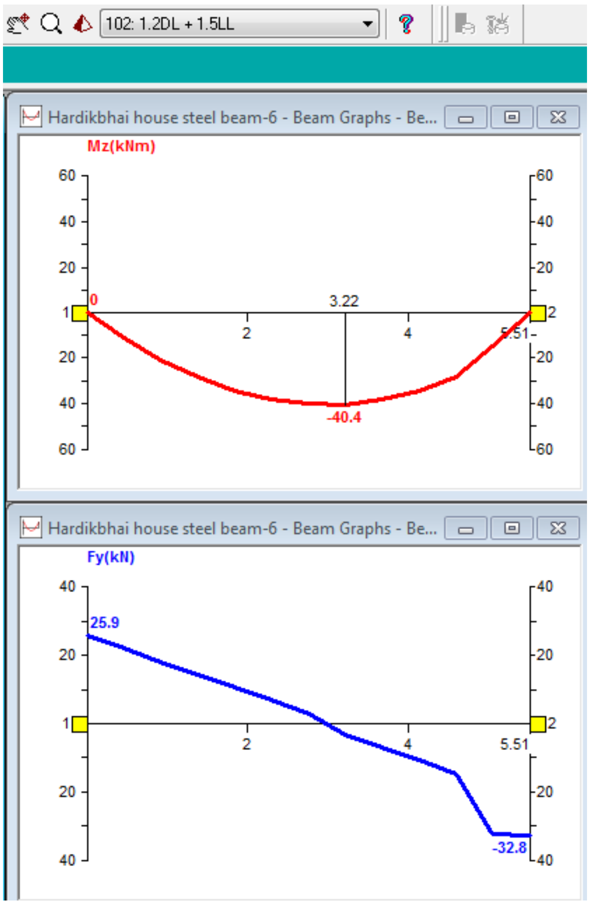
```
MemDes  Calculations  @  12:33:53 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B5

Section : 250PFC  Grade 300+
   d = 250 mm          b =  90 mm        tf = 15.0 mm        tw =  8.0 mm
Area = 4520 mm2    fyf = 300 MPa     fyw = 320 MPa        fu = 440 MPa
 Ixx = 45.10 E6mm4  Sx = 421.00 E3mm3  Zx = 361.00 E3mm3   rx = 99.90 mm
 Iyy =  3.64 E6mm4  Sy = 107.00 E3mm3  Zy = 59.30 E3mm3   ry = 28.40 mm
  Iw = 35.90 E9mm6   J =238.00 E3mm4   kf = 1.000      Manufact. Type = HR
 Zex = 421.00 E3mm3 Zey = 88.70 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  5.99,    Allowable = 10.00     [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.082 / 0.015 * 1.095 = 5.988 ]
    Web   :  le : Actual = 31.11,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.220 / 0.008 * 1.131 = 31.113 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 3.89 * 1.00 * 1.00 * 1.00 = 3.89m

Major Axis Bending
    Design Action M*x =   9.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.30 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5         [Eq 5.6.1.1(2)]
      = 1.7 *    9.0 / [(   5.0)2 + (   9.0)2 + (   5.0)2 ]0.5 = 1.337
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 3.8902 = 133.7 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo = 107.7 kNm         ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.55     [ (Ms/Mo) = 126.3 / 107.7 = 1.173 ]
     am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.34 * 0.55 *  126.3 =   93.1
  Major axis capacity Ratio = M*x  /  f Mbx
                           = 0.11,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  10.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.250 * 0.008
    Nominal Shear Yield capacity  Vw = 384.0 kN
      av = 6.09 >= 1.0 => full web shear capacity
     Vu = Vw = 384.0 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                          = 0.03,              ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.74,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================   SUMMARY   =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.11    ---- OK ----
```

Deflection allowable = span /400= 3890/400= 9.75 mm (as per Table C1 ,AS NZS 1170.0-2002) OK

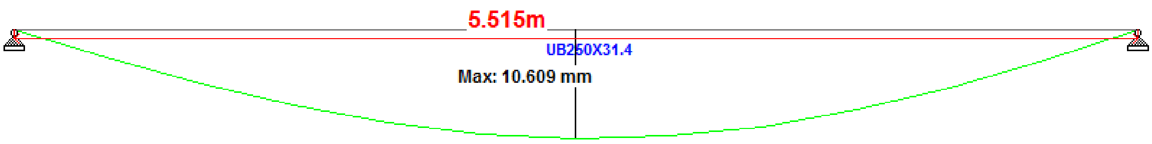
STEEL BEAM B6



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL



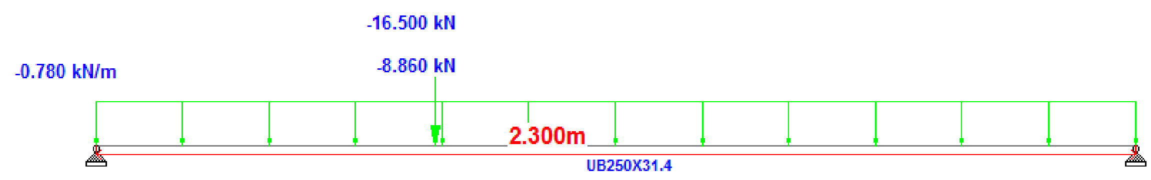
Deflection(DL+LL)

Member design B6

```
MemDes  Calculations  @  12:49:44 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B6

Section : 250UB31  Grade 300+
   d = 252 mm          b = 146 mm        tf =  8.6 mm        tw =  6.1 mm
Area = 4010 mm2    fyf = 320 MPa      fyw = 320 MPa      fu = 440 MPa
 Ixx = 44.50 E6mm4  Sx = 397.00 E3mm3  Zx = 354.00 E3mm3   rx = 105.00 mm
 Iyy =  4.47 E6mm4  Sy = 94.20 E3mm3  Zy = 61.20 E3mm3   ry = 33.40 mm
  Iw = 65.90 E9mm6    J = 89.30 E3mm4    kf = 1.000      Manufact. Type = HR
   Compactness(x,y) = N, N
 Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual  =  9.20,     Allowable = 10.00      [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202 ]
    Web    :  le : Actual = 43.55,     Allowable = 101.00      [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549 ]
                  Section Geometry Check [Major] : Passed    ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                  => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr   = 5.51 * 1.00 * 1.00 * 1.00 = 5.51m

Major Axis Bending
    Design Action M*x =  41.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.40 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5         [Eq 5.6.1.1(2)]
      = 1.7 *   41.0 / [(  20.0)2 + (  41.0)2 + (  20.0)2 ]0.5 = 1.399
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 5.5152 =   66.5 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo =  58.5 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.37      [ (Ms/Mo) = 126.4 /   58.5 = 2.159 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.40 * 0.37 *  126.4 =   64.6
  Major axis capacity Ratio = M*x   /  f Mbx
                           = 0.70,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  33.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
    Nominal Shear Yield capacity  Vw = 295.1 kN
      av = 3.55 >= 1.0 => full web shear capacity
     Vu = Vw = 295.1 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                         = 0.12,              ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.51,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================     SUMMARY    =====================
 ****  U.L.S. Capacity Check Passed,    Load Cap. Ratio = 0.70    ---- OK ----
```

Deflection allowable = span /400= 5515/400= 13.79 mm (as per Table C1, AS NZS 1170.0-2002) OK

STEEL BEAM B7



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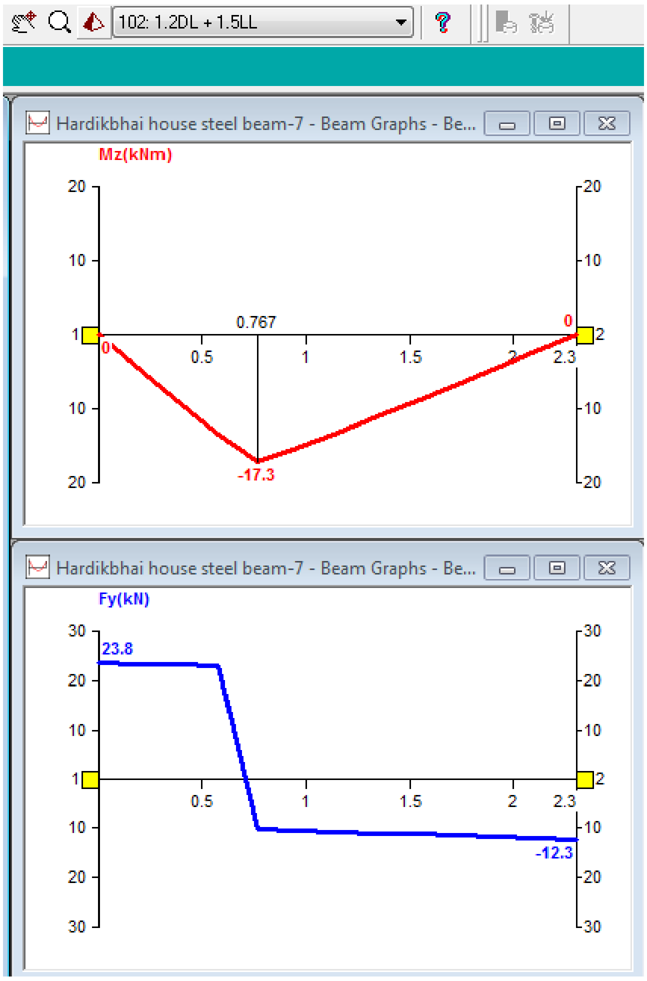
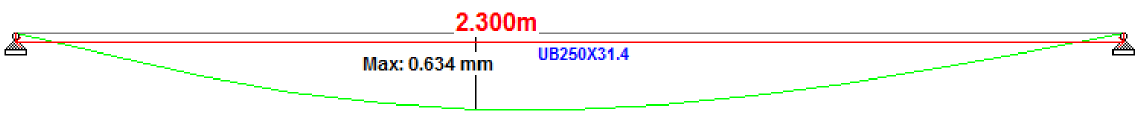


Moment and shear (1.2DL+1.5LL



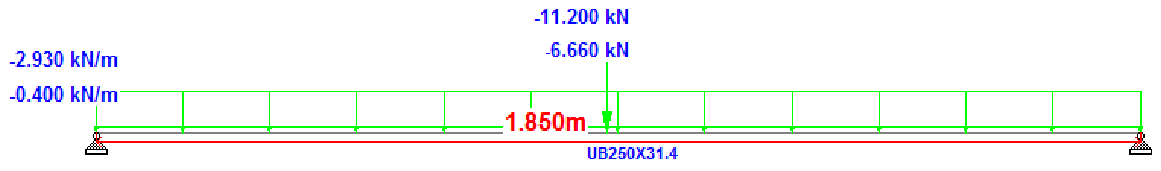
Deflection(DL+LL)

Member design B7

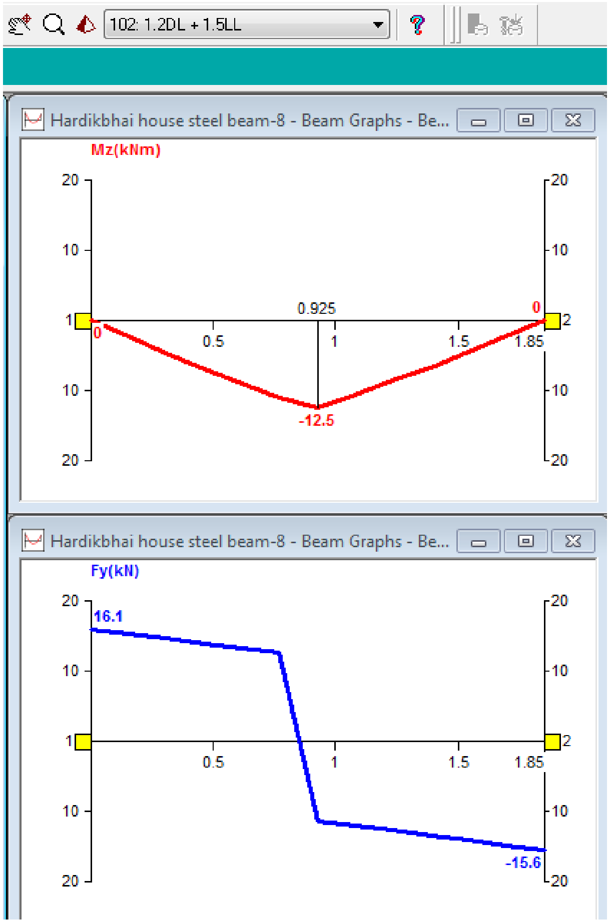
```
MemDes  Calculations  @  13:06:16 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B7

Section : 250UB31  Grade 300+
   d = 252 mm          b = 146 mm        tf =  8.6 mm        tw =  6.1 mm
Area = 4010 mm2    fyf = 320 MPa      fyw = 320 MPa        fu = 440 MPa
 Ixx = 44.50 E6mm4  Sx = 397.00 E3mm3  Zx = 354.00 E3mm3   rx = 105.00 mm
 Iyy =  4.47 E6mm4  Sy = 94.20 E3mm3  Zy = 61.20 E3mm3   ry = 33.40 mm
  Iw = 65.90 E9mm6    J = 89.30 E3mm4    kf = 1.000      Manufact. Type = HR
   Compactness(x,y) = N,  N
 Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  9.20,    Allowable = 10.00      [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202 ]
    Web   :  le : Actual = 43.55,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                  => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 2.30 * 1.00 * 1.00 * 1.00 = 2.30m

Major Axis Bending
    Design Action M*x =  18.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.40 kNm
    am   = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5         [Eq 5.6.1.1(2)]
       = 1.7 *   18.0 / [(  14.0)2 + (  18.0)2 + (   9.0)2 ]0.5 = 1.248
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 2.3002 = 382.5 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo = 235.2 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.77      [ (Ms/Mo) = 126.4 / 235.2 = 0.537 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.25 * 0.77 *  126.4 =  120.8
  Major axis capacity Ratio = M*x  /  f Mbx
                            = 0.17,               ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  24.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
    Nominal Shear Yield capacity  Vw = 295.1 kN
      av = 3.55 >= 1.0 => full web shear capacity
     Vu = Vw = 295.1 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.09,                ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.96,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================     SUMMARY    =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.17    ---- OK ----
```

Deflection allowable = span /400= 2300/400= 5.75 mm (as per Table C1, AS NZS 1170.0-2002) OK

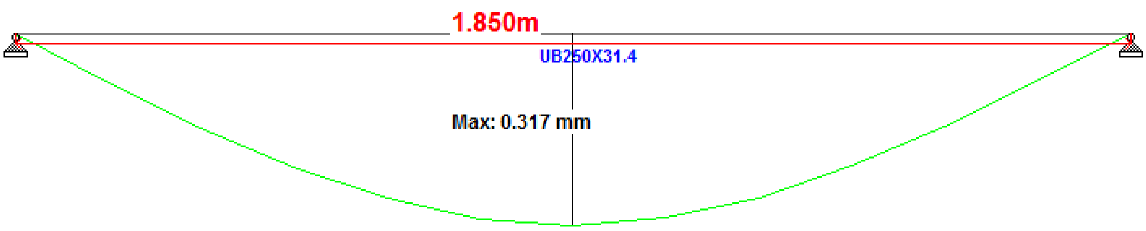
STEEL BEAM B8



Loading (DL & LL)



Moment and shear (1.2DL+1.5LL



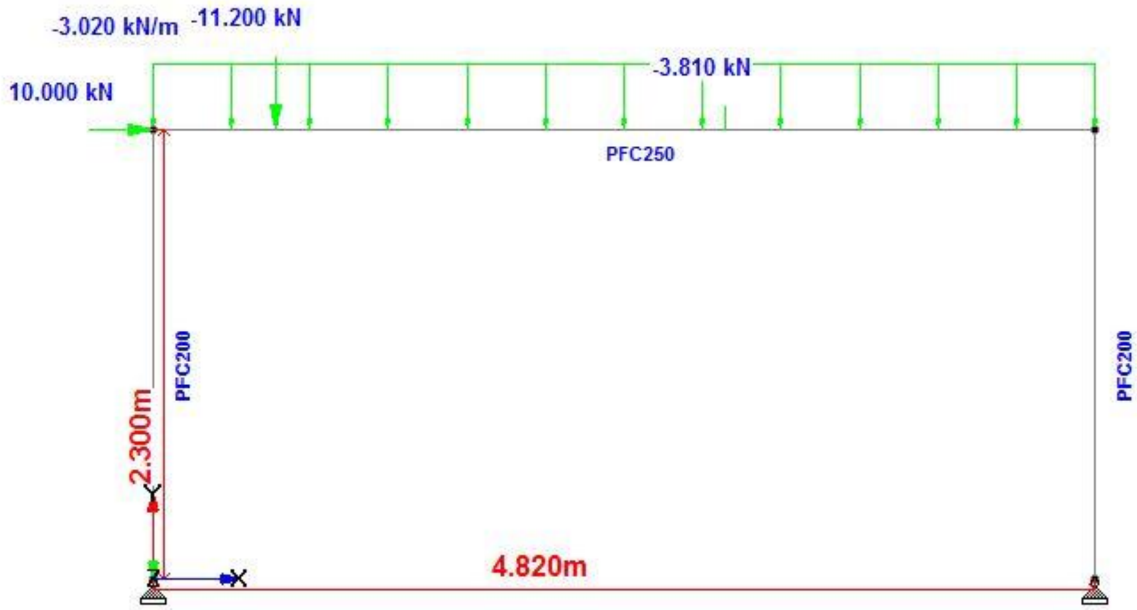
Deflection(DL+LL)

Member design B8

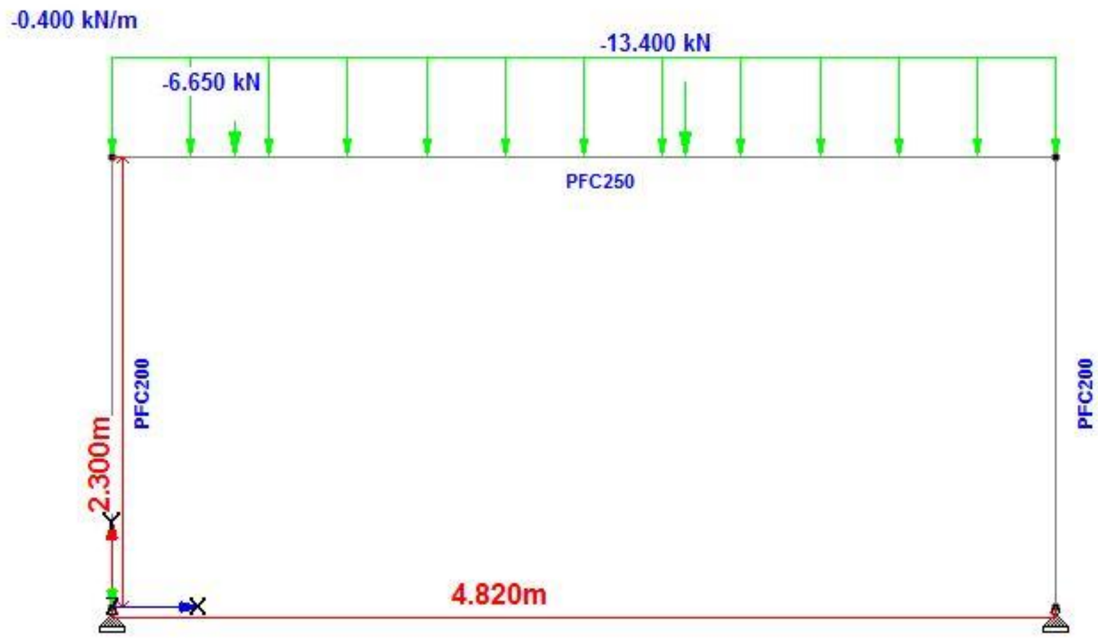
```
MemDes  Calculations  @  14:06:56 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B8

Section : 250UB31  Grade 300+
   d = 252 mm          b = 146 mm        tf =  8.6 mm        tw =  6.1 mm
Area = 4010 mm2     fyf = 320 MPa      fyw = 320 MPa       fu = 440 MPa
 Ixx = 44.50 E6mm4  Sx = 397.00 E3mm3  Zx = 354.00 E3mm3   rx = 105.00 mm
 Iyy =  4.47 E6mm4  Sy = 94.20 E3mm3  Zy = 61.20 E3mm3   ry = 33.40 mm
  Iw = 65.90 E9mm6   J = 89.30 E3mm4    kf = 1.000      Manufact. Type = HR
   Compactness(x,y) = N, N
 Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  9.20,     Allowable = 10.00     [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202 ]
    Web    :  le : Actual = 43.55,     Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                  => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 1.85 * 1.00 * 1.00 * 1.00 = 1.85m

Major Axis Bending
    Design Action M*x =  13.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.40 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5        [Eq 5.6.1.1(2)]
      = 1.7 *   13.0 / [(   6.0)2 + (  13.0)2 + (   6.0)2 ]0.5 = 1.424
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 1.8502 = 591.2 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo = 349.0 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.84     [ (Ms/Mo) = 126.4 / 349.0 = 0.362 ]
      am   as >= 1.0,  => Segment Fully Restrained
  Mbx = Msx = 126.40 kNm
  Major axis capacity Ratio = M*x  /  f Mbx
                            = 0.11,             ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  17.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
    Nominal Shear Yield capacity  Vw = 295.1 kN
      av = 3.55 >= 1.0 => full web shear capacity
     Vu = Vw = 295.1 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.06,              ---- OK ----

 Seismic Member Checks
  Cl 12.6.2.2 check,  am x  as > 1.00,   => OK
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================   SUMMARY   =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.11    ---- OK ----
```

Deflection allowable = span /400= 1850/400= 4.63 mm (as per Table C1 AS NZS 1170.0-2002) OK

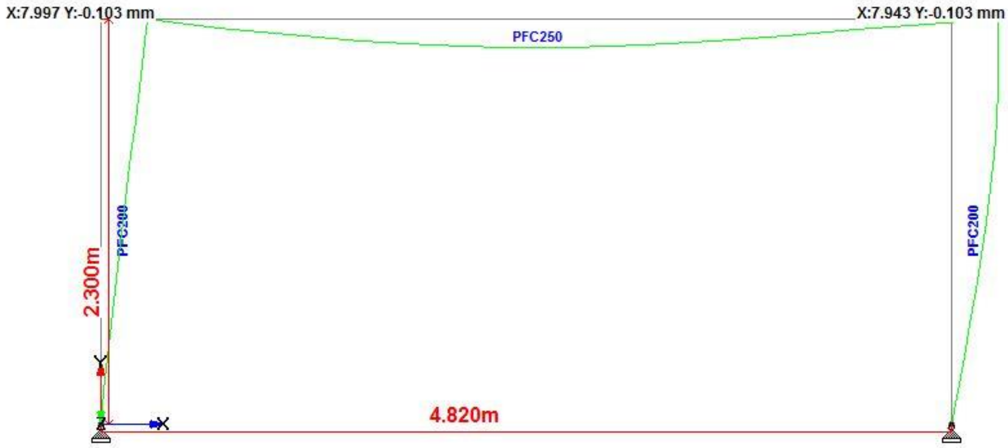
STEEL BEAM B9- Portal Frame



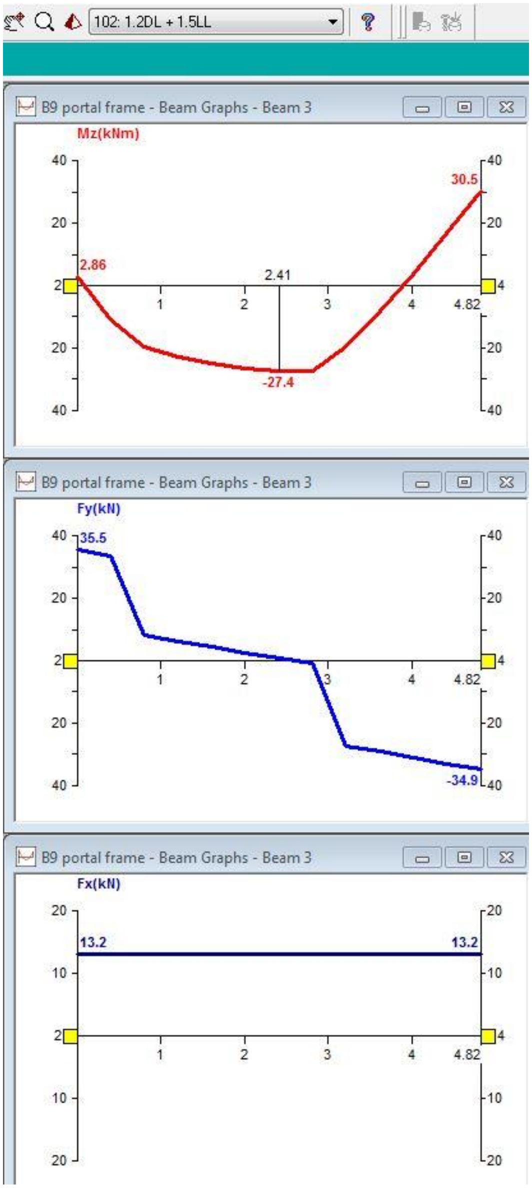
Loading DL



Loading LL



Deflection diagram (DL+LL)



Moment & shear diagram (1.2DL+1.5LL)

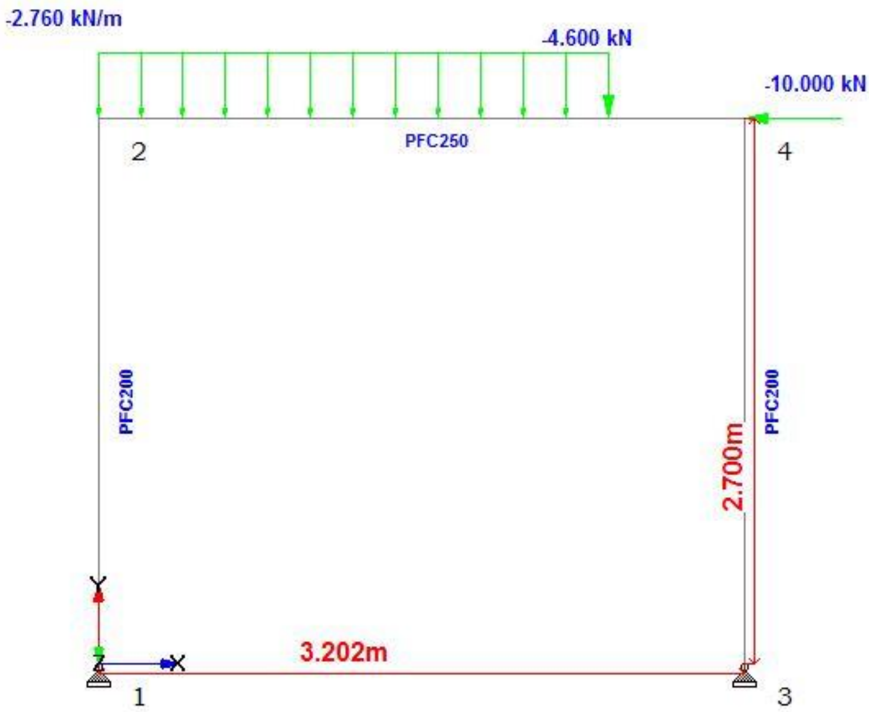
Portal beam design of B9 portal frame.- PFC250

```
MemDes  Calculations  @  14:57:54 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B9 for portal frame

Section : 250PFC  Grade 300+
   d = 250 mm         b =  90 mm       tf = 15.0 mm       tw =  8.0 mm
Area = 4520 mm2    fyf = 300 MPa      fyw = 320 MPa       fu = 440 MPa
 Ixx = 45.10 E6mm4  Sx = 421.00 E3mm3  Zx = 361.00 E3mm3   rx = 99.90 mm
 Iyy =  3.64 E6mm4  Sy = 107.00 E3mm3  Zy = 59.30 E3mm3   ry = 28.40 mm
  Iw = 35.90 E9mm6   J =238.00 E3mm4   kf = 1.000      Manufact. Type = HR
 Zex = 421.00 E3mm3 Zey = 88.70 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  5.99,    Allowable = 10.00     [ OK ]
        [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.082 / 0.015 * 1.095 = 5.988 ]
    Web    :  le : Actual = 31.11,    Allowable = 101.00     [ OK ]
        [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.220 / 0.008 * 1.131 = 31.113 ]
                  Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as LL
                                                 => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 4.82 * 1.00 * 1.00 * 1.00 = 4.82m

Major Axis Bending
    Design Action M*x =  31.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.30 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5         [Eq 5.6.1.1(2)]
      = 1.7 *   31.0 / [(  17.0)2 + (  28.0)2 + (  10.0)2 ]0.5 = 1.539
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2   = 3.1422 x   205 E6 / 4.8202 =  87.1 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo =  83.8 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.47     [ (Ms/Mo) = 126.3 /  83.8 = 1.507 ]
      am   as < 1.0, => Segment NOT Fully Restrained
  Mbx = 1.54 * 0.47 *  126.3 =   92.0
  Major axis capacity Ratio = M*x  /  f Mbx
                            = 0.37,               ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =  36.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.250 * 0.008
    Nominal Shear Yield capacity  Vw = 384.0 kN
      av = 6.09 >= 1.0 => full web shear capacity
     Vu = Vw = 384.0 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.10,                ---- OK ----

 Seismic Member Checks
   Cl. 12.6.2.2 check :  am x  as Limit
      am x  as = 0.73,  Tbl 12.6.2 Limit of 1.00 is NOT met,  => NOK
   Designer to check whether segment has yielding regions or not
   [ NOTE : If Member Category of 1, check if a value other than 1.35 may be used ]
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================     SUMMARY     =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.37    ---- OK ----
```

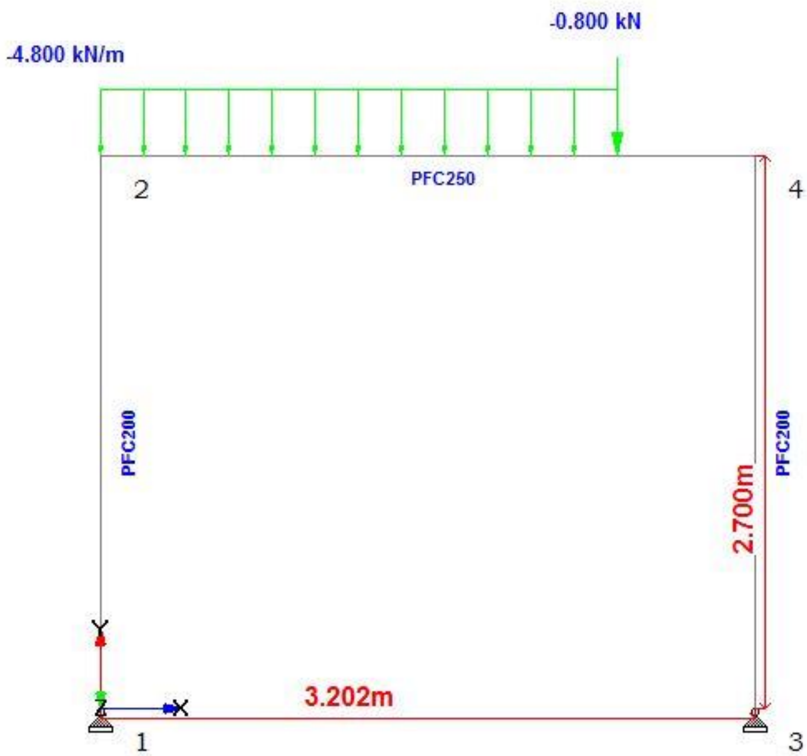
Horizontal Deflection allowable = height /200= 2300/200= 11.5 mm > 7.99mm actual (as per Table C1 AS NZS 1170.0-2002) OK

Deflection allowable = span /400= 4820/400= 12.05mm> 0.103mm actual (as per Table C1 AS NZS 1170.0-2002) OK

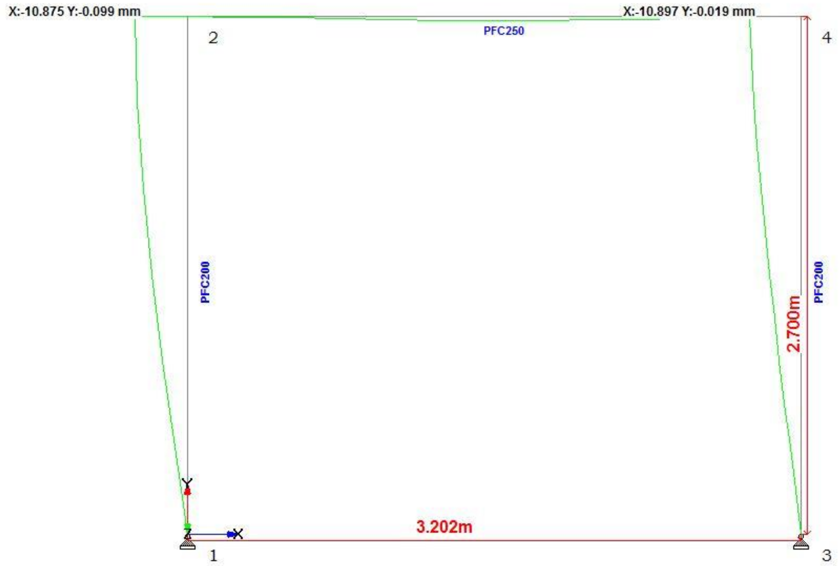
STEEL BEAM B10- Portal Frame



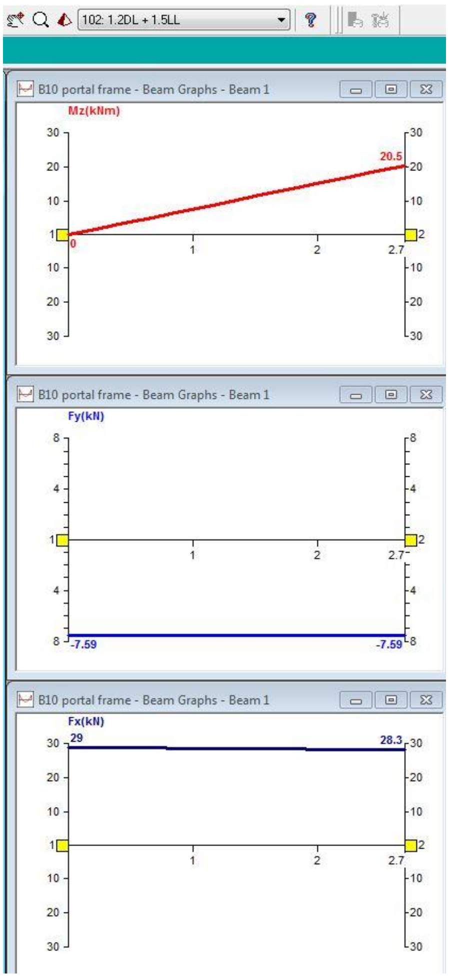
Loading DL



Loading LL



Deflection diagram (DL+LL)



Moment & shear diagram (1.2DL+1.5LL)

Portal beam design of B10 portal frame- PFC250

```
MemDes  Calculations  @  15:28:08 23-03-2018
========================================
Project : Proposed house, 45 Hangahai Flat Bush
Description : steel beam -B10 for portal frame

Section : 250PFC  Grade 300+
   d = 250 mm          b =  90 mm        tf = 15.0 mm        tw =  8.0 mm
Area = 4520 mm2    fyf = 300 MPa      fyw = 320 MPa        fu = 440 MPa
 Ixx = 45.10 E6mm4  Sx = 421.00 E3mm3  Zx = 361.00 E3mm3   rx = 99.90 mm
 Iyy =  3.64 E6mm4  Sy = 107.00 E3mm3  Zy = 59.30 E3mm3   ry = 28.40 mm
  Iw = 35.90 E9mm6   J =238.00 E3mm4    kf = 1.000      Manufact. Type = HR
 Zex = 421.00 E3mm3 Zey = 88.70 E3mm3

Check  of  Section  Category  Requirements
  Specified Minimum Section Category = 3
 Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
 NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
 ****  MAJOR axis bending ****
   Flange :  le : Actual =  5.99,    Allowable = 10.00     [ OK ]
          [  le = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.082 / 0.015 * 1.095 = 5.988 ]
    Web   :  le : Actual = 31.11,    Allowable = 101.00     [ OK ]
          [  le = WebDepth / WebThick * (fyw/250)1/2 = 0.220 / 0.008 * 1.131 = 31.113 ]
                    Section Geometry Check [Major] : Passed   ---- OK ----

Member Effective Length Calcs
 Torsional End Restraint Conds (TERC) are given as FF
                                                     => kt = 1.00
 User provided value of kl = 1.00
 User provided value of kr = 1.00
 Eff. Len Le = L * kt * kl * kr  = 3.20 * 1.00 * 1.00 * 1.00 = 3.20m

Major Axis Bending
    Design Action M*x =  21.0 kNm
  Section Bending Capacity  Msx =  fyf  Zex  =  126.30 kNm
    am  = 1.7 * M*m / [(M*2)2 + (M*3)2 + (M*4)2 ]0.5         [Eq 5.6.1.1(2)]
      = 1.7 *   21.0 / [(   6.0)2 + (  12.0)2 + (  17.0)2 ]0.5 = 1.648
  Reference Buckling Moment Calculation : Mo
    Temp. variable Pi2_E_Le2 = PI2 x E / Le2    = 3.1422 x   205 E6 / 3.2022 = 197.3 E06
    Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
    Mo = 137.0 kNm          ( Eqtn 5.6.1.1(4) )
    as = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.62     [ (Ms/Mo) = 126.3 / 137.0 = 0.922 ]
      am   as >= 1.0,  => Segment Fully Restrained
  Mbx = Msx = 126.30 kNm
  Major axis capacity Ratio = M*x   /  f Mbx
                            = 0.18,              ---- OK ----

Shear Calculations (Unstiffened Web)
    Design Action V*x =   8.0 kN
   Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.250 * 0.008
    Nominal Shear Yield capacity  Vw = 384.0 kN
      av = 6.09 >= 1.0 => full web shear capacity
     Vu = Vw = 384.0 kN
    Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
      Shear capacity ratio = V*x /  f Vu
                           = 0.02,               ---- OK ----

 Seismic Member Checks
  Cl 12.6.2.2 check,  am x  as > 1.00,    => OK
  Cl 12.6.2.5 check, Full Lateral Restraint is not required,  OK
   Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member
 ========================    SUMMARY     =====================
 ****  U.L.S. Capacity Check Passed,   Load Cap. Ratio = 0.18    ---- OK ----
```

Horizontal Deflection allowable = height /200= 2700/200= 13.5 mm > 10.875mm actual (as per Table C1 AS NZS 1170.0-2002) OK

Deflection allowable = span /400= 3202/400= 8.0mm> 0.1mm actual (as per Table C1 AS NZS 1170.0-2002) OK

GIB EzyBrace® Bracing Software

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Demand Calculation Sheet

Job Details

Name:	
Street and Number:	45 Hangahai drive, Flat Bush
Lot and DP Number:	Lot 57
City/Town/District:	
Designer:	
Company:	
Date:	

Building Specification

Number of Storeys	2	
Floor Loading	2 kPa	
Foundation Type	Slab	
	Upper	**Lower**
Cladding Weight	Light	Light
Roof Weight	Heavy	Light
Room in Roof Space	No	No
Roof Pitch (degrees)	25	25
Roof Height above Eaves (m)	1.9	1.9
Building Height to Apex (m)	7.6	
Ground to Lower Floor (m)	0.150	
Lower to Upper Floor (m)		2.7
Average Stud Height (m)	2.4	2.4
Building Length (m)	13.4	13.4
Building Width (m)	9.24	11
Building Plan Area (m²)	124	148

Building Location

Wind Zone = High

Earthquake Zone 1

Soil Type	D & E (Deep to Very Soft)
Annual Prob. of Exceedance:	1 in 500 (Default)

Bracing Units required for Wind

	Along	**Across**
Upper Level	**490**	**642**
Lower Level	**1152**	**1472**

Bracing Units required for Earthquake

	Along & Across
Upper Level	**835**
Lower Level	**1309**

Upper Level Along Resistance Sheet

Job Name:

Timber Floor Limit of 120 BUs/m Applied

	Wind	EQ
Demand	490	835
Achieved	1528	1347
	312%	161%

Line	Element	Length (m)	Angle (degrees)	Stud Ht. (m)	Type	Supplier	Wind (BUs)	EQ (BUs)	Wind	EQ
A	1	0.93		2.4	GS1-N	GIB®	59	55		
	2	1.60		2.4	GS1-N	GIB®	110	96		
	3	0.98		2.4	GS1-N	GIB®	63	58		
	4	0.64		2.4	GS1-N	GIB®	37	37		
	5	0.88		2.4	GS1-N	GIB®	55	52		
									324 OK	298 OK
B	1	1.68		2.4	GS1-N	GIB®	116	101		
	2	1.59		2.4	GS1-N	GIB®	109	95		
	3	1.09		2.4	GS1-N	GIB®	73	65		
									298 OK	261 OK
C	1	2.04		2.4	GS1-N	GIB®	140	122		
	2	2.55		2.4	GS1-N	GIB®	176	153		
									316 OK	275 OK
D	1	1.73		2.4	GS1-N	GIB®	119	104		
	2	2.72		2.4	GS1-N	GIB®	188	163		
	3	4.10		2.4	GS1-N	GIB®	283	246		
									589 OK	512 OK

GIB EzyBrace® Bracing Software

Upper Level Across Resistance Sheet

Job Name:

Timber Floor Limit of 120 BUs/m Applied

	Wind	EQ
Demand	642	835
Achieved	1206	1069
	188%	128%

Line	Element	Length (m)	Angle (degrees)	Stud Ht. (m)	Type	Supplier	Wind (BUs)	EQ (BUs)	Wind	EQ
M	1	0.68		2.4	GS1-N	GIB®	40	40		
	2	0.74		2.4	GS1-N	GIB®	44	43		
	3	2.80		2.4	GS1-N	GIB®	193	168		
									277 OK	251 OK
N	1	4.04		2.4	GS1-N	GIB®	278	242		
	2	3.26		2.4	GS1-N	GIB®	225	195		
									503 OK	437 OK
O	1	3.47		2.4	GS1-N	GIB®	239	208		
	2	1.11		2.4	GS1-N	GIB®	74	66		
									313 OK	274 OK
P	1	0.94		2.4	GS1-N	GIB®	60	56		
	2	0.86		2.4	GS1-N	GIB®	53	51		
									113 OK	106 OK

GIB EzyBrace® Bracing Software

Lower Level Along Resistance Sheet

Job Name:

									Wind	EQ
									Demand	
									1152	1309
									Achieved	
Line	Element	Length (m)	Angle (degrees)	Stud Ht. (m)	Type	Supplier	Wind (BUs)	EQ (BUs)	1714 149%	1496 114%
A	1	1.86		2.4	GS1-N	GIB®	128	111		
	2	1.85		2.4	GS1-N	GIB®	128	111		
									256 OK	222 OK
B	1	2.39		2.4	GS1-N	GIB®	165	143		
	2	2.53		2.4	GS1-N	GIB®	175	152		
	3	1.88		2.4	GS1-N	GIB®	130	113		
	4	5.20		2.4	GS1-N	GIB®	359	312		
									828 OK	720 OK
C	1	0.92		2.4	GS1-N	GIB®	58	55		
	2	1.10		2.4	GS1-N	GIB®	74	66		
	3	2.63		2.4	GS1-N	GIB®	181	158		
	4	2.13		2.4	GS1-N	GIB®	147	128		
	5	2.47		2.4	GS1-N	GIB®	170	148		
									631 OK	554 OK

GIB EzyBrace® Bracing Software

Lower Level Across Resistance Sheet

Job Name:

	Wind	EQ
Demand		
	1472	1309
Achieved		

Line	Element	Length (m)	Angle (degrees)	Stud Ht. (m)	Type	Supplier	Wind (BUs)	EQ (BUs)	1717 117%	1435 110%
M	1	1.35		2.4	BL1-H	GIB®	173	140		
	2	2.20		2.4	GS2-N	GIB®	216	189		
	3	0.56		2.4	GS2-N	GIB®	44	40		
									432 OK	369 OK
N	1	1.58		2.4	GS2-N	GIB®	155	136		
	2	1.92		2.4	GS2-N	GIB®	188	165		
									343 OK	301 OK
O	1	3.76		2.4	BL1-H	GIB®	481	391		
									481 OK	391 OK
P	1	3.60		2.4	BL1-H	GIB®	460	374		
									460 OK	374 OK

Custom Wall Elements

Supplier	System	Min. Length m	Wind BUs/m	EQ BUs/m