

ENGINEERED STRUCTURES LIMITED



75 South Street, Epsom, KT18 7PY. Design Calculations & Sketches Rev 0 December 2021

General Notes:



- 1. This is permanent work design only, the contractor is responsible for all temporary work design and the structure stability during the construction work,
- 2. All timber work New and Existing is assumed to be of grade C24
- 3. All new Steel members are S335 Grade, unless noted otherwise.
- 4. The contractor is responsible for timber and steel connections design.
- 5. All beams and lintels to have 200mm end bearing minimum, unless noted otherwise.
- 6. Compressive strength of masonry unit assumed to be 5.2 N/mm²
- 7. Mortar of class M4 is assumed for all new and old work.
- 8. Design is carried out to EuroCodes.

8. These calculations and details are to be read in conjunction with all relevant architects and engineers' drawings and specifications.

9. Full building regulation approval should be obtained prior to the commencement of works on site/before any construction materials are ordered . 10. Any works carried out prior to this are undertaken at the clients/contractor's own risk.

11. The works are to be carried out to the approval and satisfaction of the building control officer, to accepted good building practice and with full compliance and in accordance with all relevant British Annexes of Euro Standards and Codes of Practice.

12. All lengths and spans used in these calculations should be verified on site prior to commencement of any construction works. Contractor and/or steel fabricator to take their dimensions on site.

13. Builder/contractor is to check that the structural engineer's proposal (i.e. location of steel beams/trimmers etc) is feasible and necessary before ordering materials.

14. This calculation is property of ESL (Engineered Structures Limited), and intended for the foresaid property only, this pack can not be redistributed without a prior written confirmation from ESL.

Job Description:



Insertion of 2 No. conservation style rooflights in rear roof slope as shown in the Architectural drawing included in this document.

Loading:



1. Roof Loads:

 Live Loads Slates, timber battens & felt Timber rafters & insulation Ceiling & services 	0.75 kN/m ² 0.55 kN/m ² 0.2 kN/m ² <u>0.15 kN/m²</u>
 Plan Dead Load 0.9/cos45= Plan Live Load = 	1.30 kN/m² 0.75 kN/m²
 <u>2. Floor Loads:</u> Live loads Timber Boards Timber Joists Ceiling & Services 	1.5 kN/m ² 0.15 kN/m ² 0.2 kN/m ² 0.15 kN/m ²
✤ DL = ✤ LL =	0.5 kN/m² 1.5 kN/m²
External Brick Wall • Brick • 100mm Lightweight Block • Plaster	2.10 kN/m² 1.00 kN/m² 0.25 kN/m²
☆ DL=	3.35 kN/m²

Proposed First Floor:





Existing Roof Structural Arrangement:







A structural assessment has been carried out on the existing roof rafters to assess the impact of introducing a new roof light in the lower part of the roof.

The existing spacing between the rafters is 400mm, and the new Velux roof window to be introduced is 780mm, meaning a <u>single</u> existing roof rafter will need to be cut, to introduce the window.

As such, the existing roof rafters has been analysed for an increase equivalent spacing of 600mm (copy of rafter analysis and design attached to this document appendix) proving that the rafter is still safe and within its design capacity despite the additional loads imposed to it. **Conclusion:**



Introducing the proposed Velux window, and trimming a single roof rafter within a distance of 1.6m, would not require structural intervention.



Design Calculations:

	Project				Job no.	
	75 South Street					
	Calcs for			Start page no./Revision		
Engineered Structures Limited	Roof Rafter			1		
	Calcs by MKA	Calcs date 07/12/2021	Checked by	Checked date	Approved by	Approved date

TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex Tedds calculation version 1.0.05

Rafter details

Description Rafter spacing Rafter inclination 63 x 100 C24 timber rafters s_{Rafter} = **600** mm





600	
	lii
600	

FG Rafter= 0.85 kN/m²

Fq_Rafter= 0.75 kN/m²

Fs_Rafter= 0.60 kN/m²

Forces input on Rafter

Permanent load on slope Imposed load on plan Snow load on plan

Rafter loading details

Distributed loads

Permanent load on slope Imposed load on slope Snow load on slope pg = Fg_Rafter × SRafter= 0.51 kN/m

 $p_{Q} = F_{Q_Rafter} \times S_{Rafter} \times COS(\theta_{Rafter}) = 0.32 \text{ kN/m}$ $p_{S} = F_{S_Rafter} \times S_{Rafter} \times COS(\theta_{Rafter}) = 0.25 \text{ kN/m}$

ANALYSIS

Loading

Self weight included (Permanent x 1)

Tedds calculation version 1.0.37

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ENGINEERED STRUCTURES LIMITED	Roof Rafter			2		
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Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.35G + 1.50Q + ψs1.50S (Strength)	1.35	1.50	0.75	0.00
1.35G + ψ01.50Q + 1.50S (Strength)	1.35	1.05	1.50	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00
1.00G + 1.00Q + ψs1.00S (Service)	1.00	1.00	0.50	0.00
1.00G + ψ21.00Q (Quasi)	1.00	0.30	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.51 kN/m at 0 m to 3.5 m
Member	Imposed	UDL	GlobalZ	0.32 kN/m at 0 m to 3.5 m
Member	Snow	UDL	GlobalZ	0.25 kN/m at 0 m to 3.5 m

Results

Total deflection









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Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X Z			
	(mm)	(mm)	(°)	
1	0	0	0.1324	Member
2	0	0	-0.04509	Member
3	-0.1	0	-0.01935	Member

Load combination: 1.00G + 1.00Q + ys1.00S (Service)

Node	Deflection		Rotation	Co-ordinate system
	х	Z		
	(mm)	(mm)	(°)	
1	0	0	0.15223	Member
2	-0.1	0	-0.05184	Member
3	-0.1	0	-0.02225	Member

Load combination: 1.00G + y21.00Q (Quasi)

Node	Deflection		Rotation	Co-ordinate system
	X Z			
	(mm)	(mm)	(°)	
1	0	0	0.0977	Member
2	0	0	-0.03327	Member
3	0	0	-0.01428	Member

Total base reactions

Load case/combination	Fo	rce
	FX	FZ
	(kN)	(kN)
1.35G + 1.50Q (Strength)	0	4.2
1.35G + 1.50Q + ψs1.50S (Strength)	0	4.9
1.35G + ψ₀1.50Q + 1.50S (Strength)	0	5
1.00G + 1.00Q (Service)	0	3
1.00G + 1.00Q + ψs1.00S (Service)	0	3.4
1.00G + ψ21.00Q (Quasi)	0	2.2

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	2.05	1	-3	-0.7	0
		2	1.2	-1	-0.3
2	1.45	2	-1.2	-0.9	0.3
		3	0	-0.4	0

Load combination: 1.35G + 1.50Q + ψ s1.50S (Strength)

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ENGINEERED STRUCTURES LIMITED		Roof	7			
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Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-3.4	-0.8	0
		2	1.4	-1.2	-0.4
2	1.45	2	-1.4	-1	0.4
		3	0	-0.4	0

Load combination: $1.35G + \psi_0 1.50Q + 1.50S$ (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	2.05	1	-3.5	-0.8	0
		2	1.5	-1.2	-0.4
2	1.45	2	-1.5	-1	0.4
		3	0	-0.4	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.05	1	-2.1	-0.5	0
		2	0.9	-0.7	-0.2
2	1.45	2	-0.9	-0.6	0.2
		3	0	-0.3	0

Load combination: 1.00G + 1.00Q + ys1.00S (Service)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	2.05	1	1 -2.4		0
		2	1	-0.8	-0.3
2	1.45	2	-1	-0.7	0.3
		3	0	-0.3	0

Load combination: 1.00G + ψ₂1.00Q (Quasi)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(kNm)
1	2.05	1	-1.6	-0.4	0
		2	0.6	-0.5	-0.2
2	1.45	2	-0.6	-0.4	0.2
		3	0	-0.2	0

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Member	Position (m)	5	Shear force (kN)	Moment (kNm)				
Member	0.825	0		0.3 (max)	0.3			
	2.05	1	-1.2 (max abs)	-0.3	-0.4 (min)			
					Tedds calculation			
lember - Spa	<u>n 1</u>							
artial factor f	or material proper	ties and resi	stances					
artial factor fo	r material properties	s - Table 2.3	γм = 1.300					
lember detail	S		Mar diama di					
oad duration -	CI.2.3.1.2		iviedium-term	2				
			2					
Imper section	n details	har	N – 1					
uniber of unit		ber	N = 1					
oneduli of section			b = 03 mm					
imbor strongth	115 D clocc EN 228-20	16 Toblo 1	() = 100 mm					
			024					
		-1						
1 Ť		7	63x100 timber section Cross-sectional area, A, 6300 n	nm=				
	\setminus /		Section modulus, W, 105000 n	nm ^a				
	\sim		Second moment of area, 1, 525	nº 60000 mm4				
			Second moment of area, 1, 208	3725 mm4				
	\sim		Radius of gyration, 1, 28.9 mm Radius of gyration, 1, 18.2 mm					
²	Х		Timber strength class C24					
			Characteristic bending strength Characteristic shear strength, f	, f _{mk} , 24 N/mm² ., 4 N/mm²				
			Characteristic compression stre	ngth parallel to grain, f _{c.0.6} , 21 f	N/mm²			
			Characteristic compression stre Characteristic tension strength	ngth perpendicular to grain, f	₂₀₄ , 2.5 N/mm ² m ²			
	$/ \qquad \land$		Mean modulus of elasticity, E	nen, 11000 N/mm ²				
<u>+</u>	/	7	Fifth percentile modulus of elas Shear modulus of elasticity. G	ticity, E _{bos} , 7400 N/mm ²				
			Characteristic density, p., 350 k	g/m ^a				

Span details

Bearing length

L_b = **100** mm

Member span 1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	14.2	0.6	0.040	PASS
Bending stress	N/mm ²	17.6	4.0	0.225	PASS
Shear stress	N/mm ²	2.7	0.4	0.163	PASS
Bending and axial force				0.225	PASS
Column stability check				0.251	PASS
Deflection	mm	8.2	4.0	0.493	PASS

	I				1	
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Consider Combination 3 - 1.3	5G + ψ₀1.50Q +	1.50S (Strengt	h)		-	-
Modification factors						
Duration of load and moisture c	ontent - Table 3.	1 kmod = 0.8				
Deformation factor - Table 3.2	avia ava 2.1	Kdef = 0.8	((150 mm / h))	2 1 2) - 1 094		
Bending stress re-distribution fa	axis - exp.s. i	$k_{m,m,y} = 11111$ $k_{m} = 0.7$	((150 mm / n)*	-, 1.3) = 1.064		
Crack factor for shear resistance	e - cl.6.1.7(2)	kcr = 0.67				
System strength factor - cl.6.6	e ener (<u></u>)	ksys = 1.1				
Check compression parallel to	o the grain - cl f	\$14				
Design axial compression		Pd = 3.548	kN			
Design compressive stress		$\sigma_{c,0,d} = P_d / d$	A = 0.563 N/m	im²		
Design compressive strength		$f_{c.0.d} = k_{mod}$	\times k _{svs} \times fc.0.k / γ M	M = 14.215 N/mm	2	
		σc.0.d / fc.0.d	= 0.040			
PAS	S - Design paral	llel compressio	on strength ex	ceeds design p	arallel compr	ession stress
Chook docian at and of anon	0 1		0	0.1		
Check shear force - Section 6	.1.7	F 4.94				
Design shear torce		$F_{y,d} = 1.242$	2 KIN E//k	h) = 0.441 N/mm	2	
Design shear strength		$\tau_{y,d} = 1.5 \times$	$Fy,d/(Kcr \times D \times f)$	$(1) = 0.441 \text{ N/mm}^2$	1-	
Design shear strength		Iv,y,d = Kmod	× Ksys × Iv.k / γΜ	= 2.706 N/IIIII ²		
		$\tau_{y,d} / I_{v,y,d} = D\Lambda$	V.103	boar strongth ov	vcoods dosiar	shoar stross
	(i.e., 0.4.0		55 - Design si		CCCUS UCSIGI	
Check bending moment - Sec	tion 6.1.6	M 0 41	6 kNm			
Design bending moment		$\mathbf{M}_{\mathbf{M},\mathbf{d}} = \mathbf{M}_{\mathbf{M}}$	U NNIII J / W/ 3 958 M	N/mm ²		
Design bending strength		$f_{m,v,d} = k_{h,m,v}$	$\mathbf{x} \times \mathbf{k}_{\text{mod}} \times \mathbf{k}_{\text{suc}} \times \mathbf{k}_{\text{suc}}$	fm k / vm - 17 618	N/mm ²	
Design bending strength		$f_{m,y,u} = K_{n,m,y}$	y ~ Killou ~ Ksys ~		/ IN/11111	
		PASS - [Desian bendin	a strenath exce	eds design b	endina stress
Check combined bonding and		nian Castian		ig strength exec	eus uesigir b	
Combined leading checks ave	6 10 8 6 20	$(\pi_{\rm out}/f_{\rm out})$	0.2.4	- 0 226		
Combined loading checks - exp	.0.19 & 0.20	$(\sigma_{c,0,d} / f_{c,0,d})$	$)^2 \pm k_m \times \sigma_m$	/ f= 0.220		
	PAS	S - Combined	hending and a	axial compressi	on utilisation	is accentable
Choole columno cubicated to						
Effective length for v evic handli	eitner compress		2050 mm = 1	ion and bending	- CI.6.3.2	
Slandernoon ratio	ig	$L_{e,y} = 0.9 \times$	2050 mm = 10	043		
Belativa elepdorpage ratio	6.01	$\lambda y = Le, y / ly$	= 03.913	-) - 1 091		
Effective length for z-axis bendu	. 0.21	$\lambda_{\text{rel},y} = \lambda_y / y$	λι×ν(Ic.0.κ/⊏0.0:	5) = 1.004		
Slenderness ratio	ig	$\lambda_z = 1 \text{ oz} / \text{iz}$, – 0			
Relative slenderness ratio - exp	6 22	$\lambda_{rel z} = \lambda_z / z$. – ♥ π × √(fcoκ / Εοο	5) = 0		
	. 0.22	70101,2 - 702 7 1	λ	$s_{rely} > 0.3$ column	n stability che	ck is required
Straightness factor		βc = 0.2	20		r stability che	en is required
Instability factors - exp.6.25_6.2	6, 6,27 & 6,28	$k_v = 0.5 \times 0.5$	$1 + \beta_c \times (\lambda_{relv} -$	$(0,3) + \lambda_{rel,v^2} = 1$	166	
	-,	$k_z = 0.5 \times 0.5$	$1 + \beta_c \times (\lambda_{relz} -$	$(0.3) + \lambda_{rel} z^2 = 0$.470	
		$k_{c.v} = 1 / (k_{c.v})$	$v + \sqrt{(kv^2 - \lambda_{rel}v^2)}$)) = 0.627		
		$k_{c_7} = 1 / (k_{c_7})$	$7 + \sqrt{(k_{7}^{2} - \lambda_{rol})^{2}}$)) = 1.064		
		, z = 1 / (10)	- , , , , , , , , , , , , , , , , , , ,	,,		

	Project						Job no.		
			75 Sout	h Street					
	Calcs for		Roof	Rafter			Start page no./R	evision 11	
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Column stability checks - exp.6	o.23 & 6.24	c	5c,0,d / (kc,y >	< fc,0,d) + c	5m,y,d / fm,y	y,d = 0.288			
		c	. 5c,0,d / (k c,z >	< fc,0,d) + k	$m \times \sigma_{m,y,q}$	d / fm,y,d = 0.1	94		
						PASS - Co	lumn stability	is acceptable	
Consider Combination 2 - 1.3	5G + 1.50Q +	ψs1.50	S (Strengt	h)					
Check design 923 mm along	span								
Check y-y axis deflection - Se	ection 7.2								
Instantaneous deflection		8	_{5y} = 2.2 mm	ı					
Quasi-permanent variable load	factor	ſ	12 = 0.3						
Final deflection with creep		8	$\delta_{y,Final} = \delta_{y} \times$	(1 + k _{def})) = 4 mm				
Allowable deflection		8	$\delta_{y,Allowable} = I$, _m1_s1 / 25	50 = 8.2 I	mm			
		8	δy,Final / δy,Allo	wable = 0.4	493				
				PASS	S - Allow	able deflect	ion exceeds fir	nal deflectior	
Member - Snan 2									
Partial factor for material pro	perties and re	sistanc	es d aco						
Partial factor for material prope	mies - Table 2.	.3 γ	M = 1.300						
Member details									
Load duration - cl.2.3.1.2		Ν	Medium-ter	m					
Service class - cl.2.3.1.3		2	2						
Timber section details									
Number of timber sections in m	nember	1	V = 1						
Breadth of sections		t	o = 63 mm						
Depth of sections		ł	n = 100 mm	ו					
Timber strength class - EN 338	:2016 Table 1	(224						
4 63	•I								
			63x100 timbe Cross-section Section modul Section modul Second mome Radius of gyra Radius of gyra Timber atreng Characteristic Characteristic Characteristic Characteristic Characteristic Characteristic Mean modulus Fifth percentilis Shear modulus Characteristic Mean density,	r section al area, A, 630 us, W, 10500 us, W, 16500 nt of area, L, 4 nt of area, L, 2 ton, L, 28.9 m tion, L, 29.0 m tion, L,	0 mm ² 0 mm ³ 5250000 mm 2083725 mm 1m gth, f _{ma} , 24 N 1, f _{ma} , 24 N 1000 1asticity, E _{mm} 690 Ni 0 kg/m ³	۲ ۱۹ ۱۶ Ilei to grain, f _{ene} , 21 endicular to grain, f grain, f _{ene} , 14.6 N/n N/mm ⁹ , 7400 N/mm ⁹ imm ⁹	N/mm² .50x² 2.5 N/mm² m²		
Span details Bearing length		I		n					
		L							
Member span 2 results summ	nary	Unit	Capacity		Maximu	um l	Jtilisation	Result	
Compressive stress	1	N/mm ²	14.2		0.2	0	0.016	PASS	

N/mm²

N/mm²

17.6

2.7

4.0

0.4

Bending stress

Shear stress

PASS

PASS

0.225

0.134

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Bending and avial force		1			1		0.22	25	1	PASS		
Column stability check							0.22	223		PASS		
Deflection		mm	5.8		0.3		0.05	59		PASS		
					0.0		10.00					
Consider Combination 3 - 1	35G + wo1 50	0 + 150	S (Strengt	h)								
Medification factors	000 i quillou	<u>u</u> 1 1.00	o (on onge	<u></u>								
Duration of load and moisture	contont Tab	021	k									
Deformation factor - Table 3.2			$k_{def} = 0.8$									
Depth factor for bending - Mai	or axis - exp 3	: 1	$k_{\rm hmv} = min$	((150 mm	n / h) ^{0.2}	1 3) = 1.084	L					
Bending stress re-distribution	factor - cl.6.1.	6(2)	km = 0.7	((100 1111	.,, ,							
Crack factor for shear resistar	nce - cl.6.1.7(2	2)	kcr = 0.67									
System strength factor - cl.6.6	System strength factor - cl.6.6											
Check compression parallel	to the grain -	- cl.6.1.4	Ļ									
Design axial compression	•		Pd = 1.47 k	N								
Design compressive stress		$\sigma_{c,0,d} = P_d /$	A = 0.233	3 N/mn	1 ²							
Design compressive strength	Design compressive strength			$f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M = 14.215 \text{ N/mm}^2$								
			$\sigma_{c,0,d} / f_{c,0,d} = 0.016$									
PA	SS - Design p	oarallel o	compressio	on streng	th exc	eeds desigr	n para	allel compre	ess	ion stre		
Check design at start of spa	n											
Check shear force - Section	<u></u> 617											
Design shear force	0.1.7		$F_{vd} = 1.022$	2 kN								
Design shear stress - exp.6.6()		$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.363 \text{ N/mm}^2$									
Design shear strength	-		$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.708 \text{ N/mm}^2$									
g			$T_{V,V} = 1000 \times 1000 \times 1000 + 10000 + 10000 + 10000 + 10000 + 1000 + 1000 + 1000 + 1000 + 1000 + $									
			PA	SS - Desi	ign she	ear strength	exce	eeds design	n sh	ear stre		
Check bending moment - Se	ction 6.1.6				-	-		-				
Design bending moment			M _{y,d} = 0.41	6 kNm								
Design bending stress			$\sigma_{m,y,d} = M_{y,d}$	u / Wy = 3 .	.958 N/	/mm²						
Design bending strength			$f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 17.618 N/mm^2$									
			σm.y,d / fm.y,d = 0.225									
			PASS - [Design be	ending	strength ex	ceed	ls design be	end	ing stre		
Check combined bending ar	nd axial comp	pression	- Section	6.2.4								
Combined loading checks - e>	(p.6.19 & 6.20		(σc,0,d / fc,0,d) ² + σ _{m,y,d}	/ f m,y,d =	= 0.225						
-			(σc,0,d / fc,0,d) ² + km × (5 m,y,d / 1	fm,y,d = 0.158						
	I	PASS - (Combined	bending	and ax	kial compres	sion	utilisation	is a	cceptab		
Check columns subjected to	either comp	ression	or combin	ed comp	ressio	n and bendi	ng - (cl.6.3.2				
Effective length for y-axis ben	ding		Le,y = 0.9 ×	1450 mm	n = 1 3	05 mm	5					
Slenderness ratio	-		λy = L _{e,y} / i _v	= 45.207								
Relative slenderness ratio - e>	vp. 6.21		$\lambda_{rel,y} = \lambda_y / x$	π × √ (f c.0.k	/ E0.05)	= 0.767						
Effective length for z-axis ben	ding		L _{e,z} = 0 mm	ì	,							
Slenderness ratio	-		λz = Le,z / iz	= 0								
Relative slenderness ratio - ex	(p. 6.22		$\lambda_{rel,z} = \lambda_z / z$	$\pi \times \sqrt{fc.0.k}$	/ E0.05)	= 0						
					λrel	,y > 0.3 colur	nn si	tability che	ck i	s requir		
Straightness factor			βc = 0.2					-		·		

	Project				Job no.		
	75 South Street						
	Calcs for				Start page no./Revision		
Engineered Structures Limited	Roof Rafter				13		
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
	MKA	07/12/2021					
Instability factors - exp. 6.25, 6.26, 6.27 & 6.28 $K_y = 0.5 \times (1 + \beta c \times (\Lambda rel, y - 0.3) + \Lambda rel, y^2) = 0.840$							
$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$							
$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y^2})}) = 0.84$				= 0.844			
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$						
Column stability checks - exp.6.23 & 6.24		$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = 0.244$					
	$\sigma_{c,0,d}$ / (k _{c,z} :	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.173$					
	PASS - Column stability is acceptable						
Consider Combination 2 - 1 35G + 1 50Q + ws1 50S (Strength)							
Check design 976 mm along s	pan						
Check y-y axis deflection - Se	ction 7.2						
Instantaneous deflection		δ _y = 0.2 mm					
Quasi-permanent variable load factor		$\psi_2 = 0.3$					
Final deflection with creep		$\delta_{y,\text{Final}} = \delta_y \times (1 + k_{\text{def}}) = 0.3 \text{ mm}$					
Allowable deflection		δ y,Allowable = Lm1_s2 / 250 = 5.8 mm					
		$\delta_{y,Final} / \delta_{y,Allowable} = 0.059$					
		PASS - Allowable deflection exceeds final deflection					