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Structural Calculations

Project:-	Lemsford Village Hall, Lemsford, Welwyn Garden City				
Design:-	Design of	Design of Foundation & Superstructure			
Job No:-	20655		Date:-	-	Apr-19
Prepared By:-	C. Silva	MEr	g		
Basis Of Design:- BS EN 1990 BS EN 1990 NA BS EN 1991 BS EN 1991 NA BS EN 1992 BS EN 1992 NA BS EN 1993 NA BS EN 1993 NA BS EN 1995 BS EN 1995 NA BS EN 1996 BS EN 1996 NA		Basis of Stru UK National Actions on S UK National Design of Co UK National Design of St UK National Design of Tiu UK National	actural Desig Annex for Ba Structures Annex for Ac Discrete Struct Annex for De eel Structure Annex for De mber Structure Annex for De asonry Structure	n asis ctio ctur esi esi esi esi ture	s of Structural Design ns on Structures es gn of Concrete Structures gn of Steel Structures gn of Timber Structures es

Notes:-

- 1. Full building regulations and checking engineer approval must be obtained prior to installation or fabrication.
- 2. Installation to be in accordance with current codes and standards.
- All lengths and dimensions in these calculations are for design purposes only and should not be used for setting out on site. Contractor/Builder must measure up lengths/heights for setting out before ordering of any materials.
- 4. All temporary works to be designed and undertaken by a suitably qualified contractor.
- 5. All loadings to existing structures have been calculated following a visual inspection on site and further investigative works may be required to verify the type of construction.
- 6. All construction work to comply with the Construction Design & Management (CDM) Regulations 2015.
- 7. All planning and other elements of Building Regulations by others.

Revisions	:-	
Rev	Date	Revision
Α	01/05/2019	Steel Beam C revised.

Image: Stand Structures 2nd Floor, Unit 1, Birchanger Industrial Estate, Stansted Road, Bishops Stortford, Herts, CM23 2TH Image: Structure Struct	Page By Date Rev	2 CS Apr-19 -
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DESIGN BRIEF/SPECIFICATION

Drawings used:- Drawings provided by client.

Scope of required design works:-

- 1 Strip Foundation
- 2 Superstructure Design

Please also refer to RCA drawings and sketchs where applicable.

STEELWORK SPECIFICATION

The Steelwork in the following calculations including UB & UC Sections have been designed as S355 grade in accordance with BS EN 1993. (Unless noted otherwise).

Grade of fixing, end, toe and baseplates as noted in calculations.

If cold rolled grade hollow Sections2 (2) (2) have been specified, client to be aware of increased radius sizes.

Where steelwork is to be used internally grade JR to be used and where external, grade J0 to be used to BS EN 10025:1993.

All bolts in connections to be Grade 8.8 to BS EN 1993 unless specified otherwise.

All welding to be in accordance with BS EN 1011-2:2001.

Where fillet welds are specified, these are generally to be 6mm fillet weld unless noted otherwise. Where partial penetration fillet welds are specified, these are generally to be installed to a depth of 2mm less than the thickness of the connected part. (Minimum throat size = $2\sqrt{t}$)

Partial penetration butt welds to be generally a minimum of 6mm thick.

All full and partial penetration butt welds should be made using matching electrodes with a specified minimum tensile strength, yield strength, elongation at failure and Charpy impact value equivalent or better than the parent material.

Should Hollo-bolts be used, installation and detailing in accordance with Table H61 of SCI "Joints in Steel construction:Simple Joints" guidelines.

Finishes to Steel work to Architects specification.

Finishes to Steel work to provide suitable fire resistance to building regulation requirements. All steelwork to be installed strictly in accordance with BS EN 1993.

Where steel bears on masonry, solid mass concrete padstone to be placed below. Size to RCA calculations.

STRUCTUR	2nd Floor, Unit Stansted Road, Bisho The Institution of Structural Engineers	1, Birchanger Industrial Estate, ps Stortford, Herts, CM23 2TH Telephone:(01279) 506721 Fax:(01279) 506724 mail: mail@rcastructures.co.uk	Page By Date Rev	3 CS Apr-19 -
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DESIGN BRIEF/SPECIFICATION (CONT.)

STEELWORK - CPR & CE Marking Compliance.

References:

The harmonized standard (hEN) covering fabricated structural steelwo Execution of steel structures and aluminium structures. (Part 1 & 2)		
Steel Sections and Plates		
Hot finished		
Cold formed welded		
Sheet, Plate & Strip		
Bars, Rods, Wire, Sections and Bright Products		
Aluminium		
Non-preloaded structural bolting assemblies		
High strength structural bolting assemblies for preloading		
95 Quality requirements for fusion welding of metallic materials –		
Part 2: Comprehensive quality requirements.		
Welding Consumables		

Execution Class to BS EN 1090-2

<u>Consequence Class from Table B1 - BS EN 1990 =</u> **CC1** Low Consequence for loss of human life Building use from BS EN 1991-1-7 :-Agricultural Buildings, Houses less than 4 storeys.

Service Category to BS EN 1090-2 =

SC1 Quasi static actions, low seismic activity, fatigue from crane actions.

Production Category to BS EN 1090-2 = PC2 Welded components grade S355 and above,

Execution Class from Table B.3 of BS EN 1090-2 =EXC2Therefore, fabricator to note execution classEXC2to BS EN 1090-2

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DESIGN BRIEF/SPECIFICATION (CONT.)

TIMBER SPECIFICATION

The Timber in the following calculations has generally been designed as C16 grade in accordance with BS EN 1995. (Unless noted otherwise).

Timber has been designed as generally covered by suitable finishes such as plasterboard to give suitable fire protection. Client to advise if charring rates are to be assessed for unprotected timbers.

All timber connections to be made in accordance with BS EN 1995 / NHBC Standards. Solid blocking to be used where joists are notched into steel beams.

Lateral Restraint straps required to walls parallel to joists and timber roof spans.

Straps to be at a maximum 1.2 c/c apart and fixed to a minimum of 3 No. Joists All straps installed to BS EN 1995 and BS8103.

Herringbone strutting or solid blocking required perpendicular to joist spans.

2.5m - 4.5m spans, 1 row required. Over 4.5m spans, 2 rows required equally spaced.

LINTEL SPECIFICATION

All Lintels to be installed in accordance with BS5977.

Individual manufacturers literature must be referred to for installation procedure.

All Lintels to bear 150mm where possible. No steel beams must bear directly on lintels.

CONCRETE SPECIFICATION

The Concrete in the following calculations has generally been designed as C28/35 grade in accordance with BS EN 1992 & BS8500. (Unless noted otherwise). (Or equivalent RC35 designated mix to NHBC standards) Concrete may be site mixed if in accordance with NHBC 2.1 tables 1 & 2. All reinforcement bars are taken as "H" type bars with a yield strength of 500N/mm². All concrete to be installed in accordance with BS EN 1992.

MASONRY SPECIFICATION

All masonry in the following calculations has been designed in accordance with BS EN 1996. All blockwork below DPC level to be constructed in mortar designation (i) in accordance with BS EN 1996 part 1, Table 1. (Cement : Sand 1:3)

All load bearing masonry to be minimum 100mm wide.

Movement joints placed in accordance with BS EN 1996.

Joints every 12m (6m from corners) in brickwork and every 6m (3m from corners) in blockwork. Wall ties to be installed and specified to DD140 & NHBC Standards.

No individual block to weigh more than 20Kg for Health & Safety purposes.

Blockwork designed to have a maximum density of 1400Kg/m³. RCA to be advised if different.

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Title of Scheme Lemsford Village H	all, Lemsford, Welwyn Garden City	Job No. 20655
PERMANENT & VARIABLE ACTIONS	TO BS EN 1991	
Roof Actions	Permanent G _k (kN/m ²) Variable C	0 _k (kN/m ²)
<u>Pitched Roof</u> Self weight of rafters, say Felt and Battens Roof Tiles Variable - Cat. H 12 mm Plasterboard / skim	0.10 0.12 0.65 <u>0.87</u> (on slope) <u>0.52</u> <u>0.20</u> 1.07	(on slope)
<u>Flat Roof</u> Joists Decking/Finishes 12 mm Plasterboard / skim Variable Action - Category H	0.15 0.30 0.20 0.65 0.60	

0.15	
0.20	
	0.35
0.35	0.35
	0.15 0.20

Wall Actions

External Cavity Wall (Brick outer skin)	
102 mm Brickwork	2.04
100 mm Blockwork (up to 10.5N/mm ²)	1.40
12 mm Plasterboard / skim	0.20
	3.64

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PERMANENT & VARIABLE ACTIONS TO BS EN 1991

	<u>Permanent G_k (kN/m²)</u>	<u>Variable Q_k (kN/m²)</u>
Wall Actions		
External Wall Below DPC		
102 mm Brickwork	2.04	
102 mm Brickwork	2.04	
100 Cavity Fill	2.50	
	6.58	
Internal Timber Stud Partitions		
Studwork	0.15	
24 mm Plasterboard / skim	0.40	
	0.55	

Floor Actions

<u>First Floor -</u>	Timber	<u>Floor</u>	(Jsts @	400 c	:/c)		
195	Х	50	Joists (esti	mated)	0.11		
12.5 n	nm Boar	ding			0.09		
12 n	nm Plast	erboard	/ skim		0.20		
Variable Ac	ction - C	ategory A	A (Domestic)			Category A1	1.50
					0.41	Private	1.50
Timber Stu	d Partitio	ons (Gen	eral)		0.35	_	
					0.76		

Bi-Fold Doors (Self Weight)

16 mm Glazing (2No 8mm Panels worst c	0.30
Framework	0.15
	0.45



Beam Reference =

Flat Roof Joists

<u>Actions</u>	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	Dista	ince(m)	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Flat Roof	0.65	0.60		C/C'S	0.40	0.26	0.24	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	0.26	0.24	0.00
Clear Span =	5400) mm.	Bearings	=	100	mm minir	num each	side
Design span =	5500	mm	(NB - Des	sign span	only - not	for setting	g out on si	ite)
Conservatively, use	equatior	n 6.10 =	1.35 G _k +	- 1.5 Q _{k 1}	$+ \Sigma \psi_{0i} 1.$	5 Q _{k i}		
Conservatively, take	e ψ _{0i} =	1.00						
Design UDL=	0.35	+	0.36	+	0	=	0.71	kN/m
Unfactored UDL =	0.50) kN/m						
Design Point Load	1=	0.00	kN					
Distance from A to	point loa	d 1=	0	mm				
Distance from B to	point loa	d 1=	5500	mm				
Design Point Load	2=	0.00	kN					
Distance from A to	point loa	d 2=	0	mm				
Distance from B to	, point loa	d 2=	5500	mm				
Design Point Load	3=	0.00	kN					
Distance from A to	point loa	d 3=	0	mm				
Distance from B to	point loa	d 3=	5500	mm				
Beam analysed with	n simple s	supports:-						
Design Moment =		2.69	+	0.00	+	0.00	+	0.00
Design Shear Force	e At A =	1.95	+	0.00	+	0.00	+	0.00
Design Shear Force	e At B =	1.95	+	0.00	+	0.00	+	0.00
	Design	lamont	0.00					

Design Moment =	2.69 kNm.	
Design. Shear A =	1.95 kN	
Design. Shear B =	1.95 kN	

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TIMBER BEAM DESIGN to BS	SEN 1995-1-1			
Beam Reference =	Flat Roof Joists			
From previous calculation :- Max. Shear =	1.95 kN	Maximum Moment	:=	2.69 kNm
Clear Span =5400Design span =5.50	mm. Bearings m = 5500	s = 100 0 mm) mm minir	num each side
Exposure condition = (Intermediate Floors, Warm Ro Load Duration =	Service Class 1 oofs, internal and par Short Term	rty timber frame walls (Snow, Maintenanc	s) ce on roofs.	.)
$\begin{tabular}{ll} \hline Design & Timber Grade = \\ Bending Strength, f_{m,k} = \\ Shear Strength, f_{v,k} = \end{tabular}$	C24 24.0 N/mm ² 4.00 N/mm ²	Material = Solid Tin	nber. Treate	ed or untreated
Try 1 Type of fire exposure = Charring rate = General R Required fire rating = Effective size for design = Elastic Modulus , W_{yy} = K_h Factor = 1.00 k_{mod} = 0.9	No. 220 Not Exposed ate 30 minutes 220 mm dee 4.0E+05 mm ³ . Type of Beam= $k_{crit} = 1$	deep x 50 = p x 50 Partial Safety facto Load Sharing (No LTB due to bo	wide men 0.60 0 mm wide or, $\gamma_m =$ $k_{sys} =$ arding)	nber mm/min member 1.3 1.1
$\frac{Bending}{Design bending strength f_{m,d}}$	= k _{mod} . k _h	. k _{crit} . k _{sys} . f _{m,k} /γ _m	=	18.28 N/mm ²
$M_{ult} = f_{m,d} \ . \ W_{yy} = \ensuremath{7.4}$	kNm >	2.69 kNm	O.K.	
<u>Shear</u> $k_{cr} =$ Applied shear stress =	$\frac{3V_{d}}{2bh k_{cr}} =$	0.40 N/mm ²		0.05 11/2225
Design shear strength $t_{v,d} =$	k _{mod} . k _{sy}	_{/s} . † _{v,k} /γ _m	=	3.05 IN/MM ⁻
As 3.0	N/mm ² >	0.40 N/mm ²	O.K.	

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TIMBER BEAM DE	SIGN to BS E	EN 1995-1-1	<u> </u>					
Beam Reference =	= FI	at Roof Jois	sts					
<u>Deflection</u>	A = I _{yy} = 2	11000 mr I.4E+07 mr	m ² m ⁴	E _{0,mean} = k _{def} =	. 11. 0.6	00 kN/mm ² $\psi_2 =$	0.0	
Building use =	Roofs	As	sume p	point loac	ls appro	ox 75	5 % imposed	
Equivalent unfacto Equivalent unfacto	ored UDL Peri ored PL Perm	m. = . =	0.26 0.00	kN/m kN/m		M(Perm)=	0.98 kNm	
$W_{fin,G}$ =	5wL ⁴ 384El	+ 19	9.2 M EA	х	(1 + k _o	_{def})		
$W_{fin,G}$ =	6.34	+	0.16	Х	1	.6 =	10.39 mm	
Equivalent unfacto Equivalent unfacto	ored UDL Vari ored PL Varial	able = ole =	0.24 0.00	kN/m kN/m	N	1(Variable)=	0.91 kNm	
$W_{fin,Q}$ =	5wL ⁴ 384El	+ _1	9.2 M EA	Х	(1 + ψ	r₂ k _{def})		
$W_{fin,Q} =$	5.86	+	0.14	х		1 =	6.00 mm	
Final Deflection (ir	ncluding cree	p and sheai	r) =	16.40) mm			
Type of finish:-Brittle FinishLimiting value =L /250Allowable deflection =21.6 mmNB Deflection not checked in a fire situation as serviceability only.As21.6 mm>16.40 mmO.K.								
Therefore, use:-								
1 No.	220	X	50	Joist/s,	C24	Grade Ti	mber	
Centres =	400 m	m						



Beam Reference =

Trimmer 1

<u>Actions</u>	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	Distar	nce(m)	Permanent Action, G _k (kN/m)	Variable Action, Q _{k1} (kN/m)	Variable Action, Q _{ki} (kN/m)
Flat Roof	0.65	0.60		3.4/2=	1.70	1.10	1.02	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
					TOTALS:	1.10	1.02	0.00
Clear Span =	1200) mm.	Bearings	=	100	mm minii	mum each	side
Design span =	1300) mm	(NB - Des	sign span	only - not	for setting	g out on si	ite)
Conservatively, use	equatior	n 6.10 =	1.35 G _k +	- 1.5 Q _{k 1} -	$+ \Sigma \psi_{0i} 1.$	5 Q _{ki}		
Conservatively, tak	$e \psi_{0i} =$	1.00						
Design UDI =	1.49	+	1.53	+	0	=	3.02	kN/m
Unfactored UDL =	2.12	2 kN/m						,
Design Point Load	1=	0.00	kN					
Distance from A to	point loa	d 1=	0	mm				
Distance from B to	point loa	d 1=	1300	mm				
Design Point Load	2=	0.00	kN					
Distance from A to	point loa	d 2=	0	mm				
Distance from B to	point loa	d 2=	1300	mm				
Design Point Load	3=	0.00	kN					
Distance from A to	noint loa	d.3=	0	mm				
Distance from R to			1000					
Beam analysed with	point ioa h simple s	a 3= supports:-	1300	mm				
Design Moment =		0.64	+	0.00	+	0.00	+	0.00
Design Shear Force	e At A =	1.96	+	0.00	+	0.00	+	0.00
Design Shear Force	e At B =	1.96	+	0.00	+	0.00	+	0.00
-								
	Decision	lamant	0.64	L(N Inco		1		

Design Moment =	0.64 kNm.
Design. Shear A =	1.96 kN
Design. Shear B =	1.96 kN

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<u>TIMBER BEAM D</u>	ESIGN to BS EN	N 1995-1-1			
Beam Reference	= Trir	mmer 1			
From previous ca	lculation :-				
Max. S	hear =	1.96 kN	Maximum Momen	t =	0.64 kNm
Clear Span = Design span =	1200 mm 1.30 m	n. Bearings = 1300	s = 100 0 mm) mm minir	num each side
Exposure conditi (Intermediate Flo Load Duration =	on = Ser ors, Warm Roofs Sho	rvice Class 1 s, <i>internal and pa</i> ort Term	rty timber frame walı (Snow, Maintenan	's) ce on roofs	.)
<u>Design</u> Timber Bending Strength Shear Strength, f	Grade = n, $f_{m,k} =$ $v_{v,k} =$	C24 24.0 N/mm ² 4.00 N/mm ²	Material = Solid Tir	nber. Treat	ed or untreated
Try Type of fire expo Charring rate = Required fire rati Effective size for Elastic Modulus K_h Factor = $k_{mod} = 0$	2 No sure = Not General Rate ng = design = $W_{yy} = 8.1$ 1.00 Typ .9 k_{crit}	t Exposed 30 minutes 220 mm dee 1E+05 mm ³ . be of Beam= x = 1	deep x 50 = p x 100 Partial Safety facto Load Sharing (No LTB due to bo	wide men 0.60 0 mm wide or, $\gamma_m =$ $k_{sys} =$ barding)	nber mm/min member 1.3 1.1
<u>Bending</u> Design bending	strength $f_{m,d} =$	k _{mod} . k _h	. k _{crit} . k _{sys} . f _{m,k} / γ _m	=	18.28 N/mm ²
$M_{ult} = f_{m,d}$. $W_{yy} =$	= 14.7 kNr	m >	0.64 kNm	O.K.	
<u>Shear</u> Applied shear str	$k_{cr} = 0.6$ ress =2	7 3V _d =	0.20 N/mm ²		
Design shear stre	ength $f_{v,d} =$	k_{mod} . k_{sy}	_{ys} . f _{v,k} /γ _m	=	3.05 N/mm ⁻
ŀ	As 3.0 N/r	nm ² >	0.20 N/mm ²	O.K.	

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<u>TIMBER BEAM DE</u>	SIGN to BS EN	<u>1995-1-1</u>				
Beam Reference =	= Trim	mer 1				
<u>Deflection</u>	$A = 2$ $I_{yy} = 8.9I$	22000 mm ² E+07 mm ⁴	E _{0,mean} = k _{def} =	: 11.00 k 0.6 v	$\sqrt{N/mm^2}$ $\psi_2 =$	0.0
Building use =	Roofs	Assume	point loac	ls approx	75	% imposed
Equivalent unfacto Equivalent unfacto	ored UDL Perm. ored PL Perm. =	= 1.10 0.00	kN/m kN/m	M	(Perm)=	0.23 kNm
$W_{fin,G}$ =	5wL ⁴ 384El	+ <u>19.2 M</u> EA	X	$(1 + k_{def})$		
W _{fin,G} =	0.04	+ 0.02	Х	1.6	=	0.10 mm
Equivalent unfacto Equivalent unfacto	ored UDL Variab ored PL Variable	de = 1.02 de = 0.00	kN/m kN/m	M(Va	riable)=	0.22 kNm
$W_{fin,Q}$ =	5wL ⁴ 384El	+ <u>19.2 M</u> EA	X	$(1 + \psi_2 k_{de})$	_{ef})	
$W_{fin,Q}$ =	0.04	+ 0.02	х	1	=	0.06 mm
Final Deflection (ir	ncluding creep a	and shear) =	0.15	ō mm		
Type of finish:- Allowable deflection NB Deflection not As 4.8	Brittle Finish on = checked in a fir 3 mm >	4.8 mm e situation as s 0.15	Limiting · erviceabil mm	value = <i>lity only.</i> O.K.	L/	250
Therefore, use:-						
2 No.	220	x 50	Joist/s,	C24 (Grade Tin	nber
Centres =	1700 mm					



Beam Reference =

Trimmer 2

<u>Actions</u>	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m²)	Distanc	ce(m)	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Flat Roof	0.65	0.60		nominal	0.20	0.13	0.12	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
				-	TOTALS:	0.13	0.12	0.00
Clear Span =	5375	mm.	Bearings	=	100	mm minir	num each	side
Design span =	5475	mm	(NB - Des	sign span o	only - not	for setting	g out on si	ite)
Conservatively, use	equatior	n 6.10 =	1.35 G _k +	- 1.5 Q _{k1} +	⊦Σψ _{0i} 1.	5 Q _{ki}		
Conservatively, take	eψ _{0i} =	1.00						
Design UDL=	0.18	+ kNl/m	0.18	+	0	=	0.36	kN/m
Design Point Load	1=	1.96	kN	Trimmer 1				
Distance from A to	point load	11=	940	mm				
Distance from B to	point load	d 1=	4535	mm				
Design Point Load	2=	1.96	kN	Trimmer 1				
Distance from A to	point load	d 2= d 2=	1945 3530	mm mm				
Design Point Load	3=	0.00	kN					
Distance from A to	- noint load	1.3=	0	mm				
Distance from R to		-0 -	E 47E					
Beam analysed with	n simple s	upports:-	5475					
Design Moment =		1.33	+	1 53	+	2 46	+	0.00
Design Shear Force	e At A =	0.97	+	1.63	+	1.27	+	0.00
Design Shear Force	e At B =	0.97	+	0.34	+	0.70	+	0.00
~	Desian N	Ioment =	5.32	kNm				

Design Moment =	5.32 kNm.
Design. Shear A =	3.86 kN
Design. Shear B =	2.01 kN

STRUCT	CA	The l of S Er	2nd I Stansted R Institution Spineers	Floor, Unit 1, Birch load, Bishops Stort ISO 9001 REGISTERED FIRM Email: ma	anger Indus ford, Herts phone:(012 Fax:(012 ill@rcastrue	strial Estate, , CM23 2TH 279) 506721 279) 506724 ctures.co.uk	Page By Date Rev	14 CS Apr-19 A
Title of Scheme	Lemsford	Village Hall	, Lemsford	d, Welwyn Ga	arden C	ity	Job No.	20655
<u>TIMBER BEAM D</u>	ESIGN to B	<u>S EN 1995-</u>	<u>1-1</u>					
Beam Reference	=	Trimmer 2						
From previous ca	lculation :-							
Max. Sl	near =	3.86	kN	Maximum M	loment	=	5.32	kNm
Clear Span = Design span =	5375 5.48	mm. m =	Bearings 5475	= mm	100	mm minir	mum each	side
Exposure condition (Intermediate Floor Load Duration =	on = ors, Warm R	Service Cla oofs, interna Short Term	ass 1 al and part 1	ty timber fran (Snow, Mair	ne walls ntenanc	;) e on roofs	s.)	
<u>Design</u> Timber Bending Strength Shear Strength, f	Grade = n, f _{m,k} = v,k =	C24 24.0 4.00	N/mm ² N/mm ²	Naterial = So	olid Tim	ber. Treat	ed or untre	ated
Try Type of fire expose Charring rate = Required fire ratin Effective size for Elastic Modulus,	2 sure = General F ng = design = W _{yy} =	No. Not Expose Rate 30 220 8.1E+05	220 ed minutes mm deep mm ³ .	deep x	50 = 100 ty factor	wide mer 0.60 mm wide r, γ _m =	mber mm/min member 1.3	
K_h Factor = $k_{mod} = 0$	1.00 .9	Type of Be k _{crit} =	am= 1	Load Sharir (No LTB due	ig e to boa	k _{sys} = arding)	1.1	
<u>Bending</u> Design bending s	strength f _{m,d}	=	k _{mod} . k _h .	k _{crit} . k _{sys} . f _m	, _k /γ _m	=	18.28	N/mm ²
$M_{ult} = f_{m,d}$. $W_{yy} =$	= 14.7	kNm	>	5.32 kN	١m	O.K.		
<u>Shear</u> Applied shear str	k _{cr} = ess =	0.67 3V _d 2bh k		0.39 N/	/mm²			
Design shear stre	ength $f_{v,d} =$	COLL VCL	k _{mod} . k _{sys}	$_{\rm s}$. ${\rm f}_{\rm v,k}/\gamma_{\rm m}$		=	3.05	N/mm ⁻
F	As 3.0	N/mm ²	>	0.39 N/	/mm ²	O.K.		

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Title of Scheme	Lemsford Villag	ge Hall, Lemstol	rd, Welwyn	Garden Cit	У	Job No. 20655
<u>TIMBER BEAM DE</u>	ESIGN to BS EN	<u>1995-1-1</u>				
Beam Reference	= Trimi	mer 2				
<u>Deflection</u>	$A = 2$ $I_{yy} = 8.9E$	22000 mm ² E+07 mm ⁴	${ m E_{0,mean}}={ m k_{def}}={ m }$	11.00 kl 0.6 ψ	N/mm^2 $r_2 =$	0.0
Building use =	Roofs	Assume	point loads	s approx	75	% imposed
Equivalent unfacto Equivalent unfacto	ored UDL Perm. ored PL Perm. =	= 0.13 0.27	kN/m kN/m	M(I	^{>} erm)=	1.48 kNm
$W_{fin,G}$ =	5wL ⁴ - 384El	+ <u>19.2 M</u> EA	X	$(1 + k_{def})$		
$W_{fin,G}$ =	4.75 -	+ 0.12	х	1.6	=	7.78 mm
Equivalent unfacto Equivalent unfacto	ored UDL Variab ored PL Variable	le = 0.12 = 0.80	kN/m kN/m	M(Var	iable)=	3.44 kNm
$W_{fin,Q}$ =	5wL ⁴ - 384El	+ <u>19.2 M</u> EA	X	$(1 + \psi_2 k_{de})$	F)	
$W_{fin,Q}$ =	11.01 -	+ 0.27	х	1	=	11.28 mm
Final Deflection (i	ncluding creep a	nd shear) =	19.07	mm		
Type of finish:- Allowable deflection NB Deflection not As 21.	Brittle Finish on = * checked in a fir 5 mm >	21.5 mm e situation as s 19.07	Limiting v erviceabili mm	value = <i>ity only.</i> O.K.	L/	250
Therefore, use:-						
2 No.	220	x 50	Joist/s,	C24 G	irade Tin	nber
Centres =	200 mm					



Beam Reference = Rafters Permanent Leading Permanent Variable Action Variable Action, Actions Action, G_k Action, G_k Variable Action, Variable Action, Distance(m) Q_{k1} Q_{ki} (kN/m) (kN/m²) (kN/m) (kN/m) Q_{k1} (kN/m²) Q_{ki} (kN/m²) Pitched Roof 0.00 0.21 1.07 0.52 c/c's 0.40 0.43 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 TOTALS: 0.43 0.00 0.21 Clear Span = 2200 mm. Bearings = 100 mm minimum each side (NB - Design span only - not for setting out on site) Design span = 2300 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0,i} =$ 1.00 Design UDL= 0.58 0.00 0.31177 0.89 kN/m ++= Unfactored UDL = 0.64 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1 =2300 mm Design Point Load 2= 0.00 kN Distance from A to point load 2= 0 mm Distance from B to point load 2=2300 mm Design Point Load 3= 0.00 kN Distance from A to point load 3= 0 mm Distance from B to point load 3=2300 mm Beam analysed with simple supports:-Design Moment = 0.00 0.00 0.59 +0.00 +Design Shear Force At A =1.02 +0.00 0.00 0.00 +Design Shear Force At B = 1.02 +0.00 +0.00 0.00 +

Design Moment =	0.59 kNm.	
Design. Shear A =	1.02 kN	
Design. Shear B =	1.02 kN	

Title of Scheme Lemsford	2nd Stansted F The Institution of Structural Engineers Village Hall, Lemsford	Floor, Unit 1, Birchanger Indu Road, Bishops Stortford, Herts Telephone:(01 Fax:(01 Email: mail@rcastru d, Welwyn Garden C	ustrial Estate, s, CM23 2TH 279) 506721 279) 506724 uctures.co.uk City	Page 17 By CS Date Apr-19 Rev A Job No. 20655
TIMBER BEAM DESIGN to BS	<u>EN 1995-1-1</u>			
Beam Reference =	Rafters			
From previous calculation :- Max. Shear =	1.02 kN	Maximum Moment	. =	0.59 kNm
Clear Span =2200 rDesign span =2.30 r	mm. Bearings m = 2300	= 100 mm	mm minir	num each side
Exposure condition = 9 (Intermediate Floors, Warm Ro Load Duration = 9	Service Class 1 oofs, internal and part Short Term	ty timber frame walls (Snow, Maintenanc	s) se on roofs	.)
$\label{eq:bestern} \begin{array}{l} \underline{\textit{Design}} & \textit{Timber Grade} = \\ \\ \text{Bending Strength, } f_{m,k} = \\ \\ \text{Shear Strength, } f_{v,k} = \end{array}$	C24 N 24.0 N/mm ² 4.00 N/mm ²	Material = Solid Tim	nber. Treat	ed or untreated
Try1IType of fire exposure =ICharring rate =General RRequired fire rating =Effective size for design =Elastic Modulus , W_{yy} =K_h Factor =1.00k_{mod} =0.9BendingDesign bending strength fm d	No. 200 Not Exposed tate 30 minutes 200 mm deep 3.3E+05 mm ³ . Type of Beam= $k_{crit} = 1$	deep x 50 = 5 x 50 Partial Safety facto Load Sharing (No LTB due to bo	wide men 0.60 mm wide r, $\gamma_m =$ $k_{sys} =$ arding)	nber mm/min member 1.3 1.1 18.28 N/mm ²
$M_{ult} = f_{m,d} \cdot W_{yy} = 6.1 $	kNm >	0.59 kNm	О.К.	10.20
<u>Shear</u> $k_{cr} = 0$ Applied shear stress =	$\frac{3V_{d}}{2bh k_{cr}} =$	0.23 N/mm ²		
Design shear strength $f_{v,d} =$ As 3.0 I	k _{mod} . k _{sys} N/mm ² >	_s . f _{v,k} /γ _m 0.23 N/mm ²	= O.K.	3.05 N/mm ⁻

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Title of Scheme	Lemsford Vil	lage Hall, Lemsfo	ord, Welwy	n Garden C	City	Job No. 20655
TIMBER BEAM L	DESIGN to BS E	<u>N 1995-1-1</u>				
Beam Reference	e = Ra	lfters				
<u>Deflection</u>	A = I _{yy} = 3	10000 mm ² .3E+07 mm ⁴	E _{0,mean} = k _{def} =	= 11.00 0.6	kN/mm^2 $\psi_2 =$	0.0
Building use =	Roofs	Assume	point load	ds approx	75	% imposed
Equivalent unfac Equivalent unfac	ctored UDL Perr ctored PL Perm.	n. = 0.43 = 0.00	3 kN/m) kN/m	N	I(Perm)=	0.28 kNm
$W_{fin,G}$ =	5wL ⁴ 384EI	+ <u>19.2 M</u> EA	_ X	$(1 + k_{def})$		
$W_{fin,G}$ =	0.43	+ 0.05	х	1.6	=	0.76 mm
Equivalent unfac Equivalent unfac	ctored UDL Varia ctored PL Variab	able = 0.2 ⁻ le = 0.00	I kN/m) kN/m	M(V	ariable)=	0.14 kNm
$W_{fin,Q}$ =	5wL ⁴ 384EI	+ <u>19.2 M</u> EA	X	$(1 + \psi_2 k)$	def)	
$W_{fin,Q}$ =	0.21	+ 0.02	х	1	=	0.23 mm
Final Deflection	(including creep	and shear) =	0.99	9 mm		
Type of finish:- Allowable deflect NB Deflection n As As Therefore, use:-	Brittle Finish etion = ot checked in a 8.8 mm >	8.8 mm <i>fire situation as s</i> 0.99	Limiting serviceabi 9 mm	value = <i>lity only.</i> O.K.	L/	250
1 No.	200	x 50	Joist/s,	C24	Grade Tin	nber
Centres =	400 mi	m				



Beam Reference = **Ceiling Joists** Permanent Leading Permanent Variable Action, Variable Action, Actions Action, G_k Variable Action, Variable Action, Distance(m) Action, G_k Q_{k1} Q_{ki} (kN/m) (kN/m²) (kN/m) (kN/m) Q_{k1} (kN/m²) Q_{ki} (kN/m²) 0.00 Loft Joists 0.35 0.35 c/c's 0.40 0.14 0.14 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 TOTALS: 0.14 0.00 0.14 Clear Span = 100 mm minimum each side 4200 mm. Bearings = 4300 mm (NB - Design span only - not for setting out on site) Design span = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, use equation 6.10 =Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 0.19 +0.21 0 0.40 kN/m += Unfactored UDL = 0.28 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1 =4300 mm Design Point Load 2= 0.00 kN

Distance from A to point load 2 Distance from B to point load 2	0 m 4300 m	ווי ווו					
Design Point Load 3=	0.00 kN						
Distance from A to point load 3	=	0 m	nm				
Distance from B to point load 3 Beam analysed with simple sup	= ports:-	4300 m	าทา				
Design Moment =	0.92	+	0.00	+	0.00	+	0.00
Design Shear Force At $A =$	0.86	+	0.00	+	0.00	+	0.00
Design Shear Force At B =	0.86	+	0.00	+	0.00	+	0.00

Design Moment =	0.92 kNm.
Design. Shear A =	0.86 kN
Design. Shear B =	0.86 kN

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Title of Scheme	Lemsford	l Village Hali	, Lemsford	d, Welwyn Ga	arden C	ity	Job No. 20655
TIMBER BEAM DES	GIGN to B	<u>S EN 1995-</u>	<u>1-1</u>				
Beam Reference =		Ceiling Joi	sts				
From previous calco Max. She	ulation :- ar =	0.86	kN	Maximum N	loment	=	0.92 kNm
Clear Span = Design span =	4200 4.30	mm. m =	Bearings 4300	= mm	100	mm minir	num each side
Exposure condition (Intermediate Floors Load Duration =	= s, Warm R	Service Cla oofs, interna Short Term	ass 1 al and pari 1	ty timber fram (Snow, Main	ne walls itenanc) e on roofs	.)
<u>Design</u> Timber G Bending Strength, f Shear Strength, f _{v,k}	rade = _{m,k} = =	C24 24.0 4.00	N/mm ² N/mm ²	Material = So	olid Tim	ber. Treat	ed or untreated
Try Type of fire exposu Charring rate = Required fire rating Effective size for de Elastic Modulus , W K_h Factor = $k_{mod} = 0.9$ <u>Bending</u> Design bending str	1 re = General I = sign = $I_{yy} =$ 1.00 enoth f	No. Not Expose Rate 30 175 2.6E+05 Type of Be k _{crit} =	175 ed minutes mm deep mm ³ . am= 1	deep x D x Partial Safet Load Sharin (No LTB due	50 = 50 y factor ig e to boa	wide mer 0.60 mm wide $r, \gamma_m =$ $k_{sys} =$ arding)	nber mm/min member 1.3 1.1 18.28 N/mm ²
$M_{ult} = f_{m,d} \cdot W_{yy} =$	4.7	kNm	>	0.92 kN	Jm	O.K.	10.20
<u>Shear</u> Applied shear stres	$k_{cr} =$ s =	0.67 3V _d 2bh k _{cr}	=	0.22 N/	′mm²		O OF N/mmf
Design shear streng	$g tn t_{v,d} =$	N/mm^2	K _{mod} . K _{sys}	s · T _{v,k} / γ _m	/mm ²	=	3.05 W/mm ⁻
As	3.0	IN/ITITI	>	0.22 N/	[[]]]]	U.K.	

STRUC		Sta The Instit e of Struct Engine	2nd Floor, Unit 1, unsted Road, Bishop ution ural cors	Birchanger Indu s Stortford, Herts Telephone:(01 Fax:(01 ail: mail@rcastru	astrial Estate, s, CM23 2TH 279) 506721 279) 506724 actures.co.uk	Page 21 By CS Date Apr-19 Rev A
Title of Scheme	Lemsford V	illage Hall, Le	msford, Welwy	yn Garden (City	Job No. 20655
TIMBER BEAM DE	SIGN to BS I	<u>EN 1995-1-1</u>				
Beam Reference	= C	eiling Joists				
<u>Deflection</u>	A = I _{yy} = 2	8750 mm 2.2E+07 mm	² E _{0,mean} = ⁴ k _{def} =	= 11.00 0.6	kN/mm^2 $\psi_2 = 0$	0.0
Building use =	Roofs	Ass	ume point loa	ds approx	75 9	% imposed
Equivalent unfacto Equivalent unfacto	ored UDL Per ored PL Perm	m. = . =	0.14 kN/m 0.00 kN/m	٢	√l(Perm)=	0.32 kNm
$W_{fin,G}$ =	5wL ⁴ 384EI	+ <u>19</u> E	.2 M x EA	$(1 + k_{def})$	I	
W _{fin,G} =	2.53	+ 0	.06 x	1.6	=	4.15 mm
Equivalent unfacto Equivalent unfacto	ored UDL Var ored PL Varia	iable = ble =	0.14 kN/m 0.00 kN/m	M(V	/ariable)=	0.32 kNm
$W_{fin,Q}$ =	5wL ⁴ 384EI	+ <u>19</u> E	2 M x A	$(1 + \psi_2)$	(_{def})	
$W_{fin,Q} =$	2.54	+ 0	.06 x	1	=	2.60 mm
Final Deflection (ir	ncluding cree	p and shear)	= 6.7	5 mm		
Type of finish:- Allowable deflection NB Deflection not As 16.8	Brittle Finisl on = <i>checked in a</i> 3 mm >	n 16.8 n a fire situation	Limiting nm as serviceab 6.75 mm	value = <i>ility only.</i> O.K.	L/	250
Therefore, use:-						
1 No.	175	x 5	50 Joist/s,	C24	Grade Tim	ber
Centres =	400 m	nm				



Ref = Steel Beam A

Actions

	Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	C	Dist	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Cavity Wall	3.64			height	1.00	3.64	0.00	0.00
Bi Fold Doors	0.45			height	2.50	1.13	0.00	0.00
Pitched Roof	1.07		0.52	on slope	4.00	4.28	0.00	2.08
Flat Roof	0.65	0.60		5.5/2=	2.75	1.79	1.65	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x133x30)			0.59		
					TOTALS:	11.41	1.65	2.08

Clear Span = 3570 mm. Bearings = 100 mm minimum each side Design span =3670 mm (NB - Design span only - not for setting out on site) Span between restraints = 3670 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 3.11769 21.00 kN/m 15.41 +2.48 += Unfactored UDL = 15.14 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1= 3670 mm Design Point Load 2= 0.00 kN Distance from A to point load 2=0 mm Distance from B to point load 2=3670 mm Design Point Load 3= 0.00 kN Distance from A to point load 3= 0 mm Distance from B to point load 3= 3670 mm 0.00 kN Design Point Load 4= Distance from A to point load 4= 0 mm Distance from B to point load 4= 3670 mm Beam analysed with simple supports:-Design Moment = 35.36 kNm ULS Shear Force At A = 38.54 kN ULS Shear Force At B = 38.54 kN

Design Moment =	35.36 kNm
Design. Shear A =	38.54 kN
Design. Shear B =	38.54 kN

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Title of Scheme	Lemsford Village Hall,	Lemsford, Welwy	n Garden City	Job No. 20655						
DESIGN OF STEEL	DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = Steel Beam A									
Design Data (Beam in pure bendi	From previous calculati ing only - no axial forces)	ion:- Moment, M Design Sh	$M_{y,Ed} = 35.36$ hear = 38.54	kNm kN						
Try a section Size = 2 No.	203x133x30	Desiç Between Re	gn Span = 3670 estraints = 3670	mm mm						
Grade of Steel = Web thickness, t_w =	S355 FI 6.40 mm Yi	lange thickness, $t_f =$ field Strength, $f_y =$	9.60 mm 355 N/mm ²	(Table 3.1)						
Classification of Sec	tion									
Web = d/t =	$\frac{172.40}{6.40} =$	26.94 mm	= 3	0.81						
Flange = c =	133.90 -	15.20 -	6.40 =	56.15 mm						
c/t =	<u>56.15</u> = 9.60	2 5.85 mm								
(From table 5.2, for r web Class 1 flange Class 1	$\begin{array}{ll} \text{olled Sections2)} \\ \text{limiting} = & 72 \ \epsilon \\ \text{limiting} = & 9 \ \epsilon \end{array}$	= 58.58 = 7.32	mm Therefore mm Therefore	Class 1 Suitable Class 1 Suitable						
Moment Resistance	$W_{pl,y} = 314000 \text{ m}$	is Class 1 nm ³ W _{el,y} =	use Plastic 280000 mm ³	$\begin{array}{rl} modulus. \\ \gamma_{M0} = & 1 \end{array}$						
$M_{c,Rd} = 314000$	x 355 = 1	111.47	kNm	(Cl 6.2.5)						
As 222.94	> 35.36 kt	Nm	Moment resistance c	f section O.K.						
Shear Resistance	Root radius,	r = 7.60	mm A =	3820 mm ²						
$A_{v,z} = A - 2 b t_f -$	+ $(t_w + 2r) t A_{v,z} =$	1456 mm ²	$\eta \ h_w \ t_w = 1103$	mm ²						
$V_{pl,z,Rd} = \underline{A_{v,z} (f_y)}_{\gamma_{Md}}$	$((3)^{0.5})) =$	298.52 kN	Combined bending/s as max. moment awa	hear not considered y from max. shear.						
As 597.04	> 38.54 kM	N	Shear resistance of s	ection O.K.						



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Title of Scheme	Lemsford	Village Ha	all, Lemsfo	ord, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEEL	MEMBER T	<u>O BS EN</u>	<u>1993-1-1</u>		Ref =	Steel Bea	ım A	
Lateral Torsional Bu	ckling (Con	t)						
<u>h</u> =	206.80 133.90	=	1.54	<	2	use buck. <i>(Table 6.5,</i>	curve)	b
Imperfection factor,	$\alpha_{LT} =$	0.34		$\beta =$	0.75			
$\phi_{LT} = 0.5$	$5 (1 + lpha_{LT})$	$L_{LT} - \lambda_{LTO}) +$	$\beta \lambda_{LT}^{2}$)	=	1.28			
Reduction factor,	$\chi_{LT} =$	$\phi_{LT} + (\phi_{LT}^2)$	1 - βλ _{LT} ²) ^{0.5}	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < 1$	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT} = 0.53$	0.53	<	1		0.53	<	0.60	
Conservatively, facto	or, f =	1		$\chi_{LT,mod} =$	0.53		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} W_{y}$	$_{y}f_{y}^{}/\gamma_{M1}$	=	58.68	kNm				
As 117.35	>	35.36	kNm	Lateral To	orsional B	uckling res	sistance	0.K.
Deflection Checks		Take aver	age partial	safety fact	ors as	1.425		
Equivalent unfactore	ed SLS udl t	o beam =	24.81	x x	8	_=	14.74	kN/m
I _{y=} 2.9E+07	7 mm ⁴	Major sec	ond mome	ent of area	from sectio	n property	tables.	
Deflection = Finishes =	2.86 Brittle	mm	Deflection	n limits =	Span 360	_	10.19	mm
As 2.86	<	10.19	mm	Deflectio	n due to S	SLS loading	g is	0.K.
<i>Frequency Checks</i> By inspection, frequ	ency checks	s not requi	red due to	low deflec	tions.			

2 No.

203x133x30

S355

section

Therefore use



BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam A

Bearing at End "A"	Factored	Reaction	=	38.54	kN ULS		
$f_k = K f_b^{\alpha} f_m^{\beta}$	Masonry	under pac	dstone	Aggregate	Concrete blo	ocks Grou	ıp 1
Unit size below = 2 Act. Comp. Str = 3 .	15 mm h x 50 N/mm ⁻	100 Norm	mm width . Mean Co	n omp. Strer	Shape Fact ngth, f _b =	or, δ = 4.83 №	1.38 \/mm ⁻
From National Annexe, Ta	ble NA4,	K =	0.55	$\alpha =$	0.7	$\beta =$	0.30
Mortar Designation =	(iii)	Mortar Ty	pe M	4	f _m =	4.00	√mm²
f _k = 2.51 N/mm ⁴	$f_d =$	f _k /γ _m		γ_m (Table N	NA1)	=	2.70
Bearing capacity, $N_{Rdc} =$	(1.2 + 0.4	4(a ₁ /h _c))f _d /	A _b but not	more thar	n 1.5f _d A _b .		
Edge distance, a1 from ed	lge of bearing	g =	600	mm			
Height, h _c , of bearing base	e above floor	=	2000	mm			
Bearing capacity, $N_{Rdc} =$	1.32	$f_d A_b$	<	1.5f _d A _b .	use	1.32 f	_d A _b .
Bearing capacity, $N_{Rdc} =$	1.23	N/mm ⁻					
Bearing size = 4	00 mm x	100	mm x	215	mm deep		
38.54 x 10 ³ N							
40000 mm ² =	0.96	N/mm ⁻	<	1.23	N/mm ⁻	C	D.K.
Therefore, use a 4	00 mm x	100	mm x	215	mm dp pac	Istone	

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Title of Scheme	Lemsford Vill	age Hall, Len	nsford, Welw	vyn Garden City	Job No. 20655
DESIGN OF STEEL	MEMBER TO I	<u>BS EN 1993-1</u>	<u>-1</u>	Ref = PFC Colu	ımn A
Combined Axial Co	ompression ar	nd Bending			
Clear Height/lengt Distance between	h = restraints =	30 30	000 mm. 000 mm.		
Consider column t Therefore conside	to be fully brac r nominal mor	ced by surrou nent from eco	Inding struct	ture. lection only.	
Try a section size	260×9	0x35 PFC		Grade of Steel =	S355
Size of section =	depth, h = width, b =	2	260 mm 90 mm	(about y-y directior (about z-z directior	n(Major)) n (Minor))
Ecc about y-y = (Major)	<u>260</u> 2	+ 100) =	230 mm =	0.23 m
Ecc about z-z = (Minor)	<u>90</u> 2	+ 100) =	145 mm =	0.15 m
		Z	R =	0 kN	
R = 38.54	4 kN /			у	
				η = 0	KIN
Load Summany		Z	R =	0 kN	
	0.00 38.54 40 kN	- 0.00 - 0.00 Take i	D) X D) X minimum me	0.23 = 0.15 = oment as	0.00 kNm 5.59 kNm 3 kNm

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Title of Scheme	Lemsford	l Village Ha	ll, Lemsfo	ord, Welwy	ın Garden	City	Job No	. 20655
DESIGN OF STEEL	. MEMBER	TO BS EN 1	1 <u>993-1-1</u>		Ref =	PFC Co	lumn A	
<u>Combined Axial C</u>	ompressio.	<u>n and Bend</u>	<u>ding</u>					
Design Data Moment, $M_{y,Ed} =$ Axial Comp, $N_{b,Ed} =$ Length in y plane =	3.00 = 39.92 = 3000	kNm kN mm	Length in	Moment, z plane =	M _{z,Ed} = 3000	5.59 mm	9 kNm	
$ \begin{array}{llllllllllllllllllllllllllllllllllll$								
Classification of Sec	ction for Be	nding & Co	mpressior	ר		ε =	0.81	
$\alpha = \frac{1}{2}$	(1 +	F _c) =	0.53	3 >	0.5	0	
$\Psi = \frac{2 F_{c}}{A f_{y}}$	1	=	-0.95	>	-1			
Web = c/t =	208.00	_ =	26.00	mm				
Flange = c =	90.00	-	24.00	-	8.00	=	29) mm
c/t =	29.00 14.00	_ =	2 2.07	mm				
(From table 5.2, for i	rolled sectio	ons)	E 4 O E		Thoroforo	Olaca 1	0 C E	Quitable
Flange Class 1	a>0.5 a>0.5	limiting =	54.25 54.25	mm	Therefore	Class 1 Class 1	a>0.5 a>0.5	Suitable
Therefore, section for	or design ta	ke as a	Class 1	Section a	nd use	Plastic	Design	
Therefore use	Wpl,y =	425000	mm ³					

2nd Floor, Stansted Road, I <i>The Institution</i> of Structural Engineers	Unit 1, Birchanger Industrial Estate, Bishops Stortford, Herts, CM23 2THPage 29Telephone:(01279) 506721By CSFax:(01279) 506724Date Apr-19Email: mail@rcastructures.co.ukRev A
Title of SchemeLemsford Village Hall, Lemsford,	Welwyn Garden City Job No. 20655
DESIGN OF STEEL MEMBER TO BS EN 1993-1-1	Ref = PFC Column A
Check Buckling of Member in Compression	
Elastic critical Force, $N_{cr} = \frac{\pi^2 E I}{L_{cr}^2}$ $N_{cr,}$	y = 10877.11 kN z = 812.10 kN
Non-dimensional slenderness, $\begin{pmatrix} A f_y \\ N_{cr} \end{pmatrix}$. ^{0.5} $\lambda_y \lambda_z$ =	= 0.381 = 1.393
Reduction factor, $\chi = \frac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}}$	but $\chi_{LT} < 1.0$ 6.3.1.2
$\frac{h}{b} = \frac{260.00}{90.00} = 2.89$	> 1.2 t_f = 14.00 mm
about y-y, use buck. curvea(Table 6.2)about z-z, use buck. curveb(Table 6.2)	$\begin{array}{ll} \text{Imperfection factor, } \alpha_y = & 0.21 \\ \text{Imperfection factor, } \alpha_z = & 0.34 \end{array}$
$ \phi_y = 0.5 (1 + \alpha (\lambda - 0.2) + \lambda^2) = 0.59 \phi_z = 0.5 (1 + \alpha (\lambda - 0.2) + \lambda^2) = 1.67 $	9 $\chi_y = 0.958$ 7 $\chi_z = 0.385$
$\chi_{min} = 0.385$ (Max 1.0)	
Design buckling Resistance, $N_{b,Rd}=~~\chi$ A $f_y/\gamma_{M1}=$	606.21 kN 6.3.1.1
Therefore 606.21 > 39.92 kN O.k	Κ
Moment Resistance, M _{z,Rd}	
$W_{pl,z} = \begin{array}{cc} As \text{ member is} & Cla \\ W_{el,z} = 102000 \text{ mm}^3 & W_{el,z} \end{array}$	ss 1 use Plastic modulus. _z = 56300 mm ³ $\gamma_{M0} = 1$
$M_{z,Rd} = 102000 ext{ x} ext{ 355} = 1$	36.21 kNm (Cl 6.2.5)
As 36.21 > 5.59 kNm	Moment resistance of section O.K.

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Title of Scheme	Lemsford Villag	ge Hall, Lemsfe	ord, Welwy	n Garden	City	Job No.	20655
DESIGN OF STEEL MEMBER TO BS EN 1993-1-1 Ref = PFC Column A							
Lateral Torsional Bu	ickling						
$\lambda_{LT} =$ (Conserv	ative for Column	design) =	$0.9 \ \lambda_z$		$i_z =$	28.2	mm
$\lambda_z = \underline{\qquad L_{cr} \ / \ i_z} \\ \underline{\qquad \lambda_1}$	$\lambda_1 = 76.3$	7 $\lambda_{LT} =$	1.25	For rolled	profiles,	$\lambda_{\text{LTO}} =$	0.4
<u>h</u> =	260.00 90.00	= 2.89	>	2	use buck. <i>(Table 6.5,</i>	curve)	d
Imperfection factor,	$\alpha_{LT} = 0.$	76	$\beta =$	0.75			
$\phi_{LT} = 0.8$	5 (1 + α_{LT} (λ_{LT} - λ	$_{LTO}) + \beta \lambda_{LT}^{2})$	=	1.41			
Reduction factor,	$\chi_{LT} = \qquad $	$\frac{1}{\left(\phi_{LT}\right)^2 - \beta \lambda_{LT}^2}^{0.5}$	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} <$	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT}=~0.43$	0.43	< 1		0.43	<	0.64	O.K.
Conservatively, take	e factor, $f = 1$		$\chi_{\text{LT,mod}} =$	0.43		$\gamma_{M1} =$	1.00
$M_{b,Rd} = -\chi_{LT,mod} W$	$_{y} f_{y} / \gamma_{M1} =$	65.05	i kNm				
As 65.05	> 3.	00 kNm	Lateral To	orsional B	uckling res	sistance	O.K.
Combined bending and compression buckling							
Simplified version o	f equations 6.61 &	& 6.62:-					
N _{ed} +	M _{y,Ed} -	+ 1.5	M _{z,Ed}	<	1		

39.92	+	3.00	+	1.5	5.59	=	0.34	0.K.
606.21		65.05			36.21			

|--|

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Title of Scheme Lemsford V	Job No.	20655	

BASEPLATE DESIGN TO BS EN 1993



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Title of Scheme Lemsford Village Hall, Lemsford, Welwyn Garden City	Job No. 20655
BASEPLATE DESIGN TO BS EN 1993	
Reference:- Baseplate for Column A	
Minimum thickness, $t_b = c \left(\underbrace{3 f_{jd}}{f_y} \right)^{0.5} = 1.14$ but not less, than $tf = 14.0 \text{ mm}$	mm
Minimum thickness, $t_b = 14.0 \text{ mm}$	
Therefore 15 mm plate is <u>O.K.</u>	
$\frac{\text{Check to Welds}}{\text{Shear Load}} = 10.00 \text{ kN}$	
Grade of Steel = S275 EN10025-2 Thickness of material =	t<40mm
Simplified design from BS EN 1993:-	
$f_u =$ 430 N/mm ² $\beta =$ 0.85 $\gamma_{M2} =$ 1.25	
Design shear Strength, $f_{vw,d} = \frac{f_u / (3)^{0.5}}{\beta_w \gamma_{M2}}$ $f_{vw,d} = 233.66 \text{ N/mm}^2$	
Try a fillet weld, leg size = 6 mm a = 4.2 mm Design resistance of weld = 0.981 kN/mm	
Effective length of welds =520 mmWeld Resistance =510.31As 510.31 > 10.00 kN Weld is $O.K.$	kN
Summary:- Baseplate for Column A	
Therefore use 400 mm x 200 mm x 15 mm thick	(baseplate
Grade = S275Assume concrete below =C16Use a6 mm thick fillet weldMinimum length =520Holding down bolts , by inspection use 4 No. Hilti HY-200 HIT RODS M16	/ 20 mm

STRUC Title of Scheme	CTURES	Star The Institu of Structu Enginee age Hall, Lem	2nd Floor, Unit 1, f sted Road, Bishops fion ral rs ISO 9001 REGISTERED TREM Email Sford, Welwyn	Birchanger Indus Stortford, Herts, Telephone:(012 Fax:(012 il: mail@rcastruc Garden Cia	trial Estate, CM23 2TH 79) 506721 79) 506724 tures.co.uk	Page 33 By CS Date Apr-19 Rev A Job No. 20655	
PAD FOOTING DESIGN (CENTRALLY LOADED) Bef:- Pad under Column A							
Assume	100 kN/m ² allowa	able bearing p	ressure (to be	e checked d	on site)		
Pressure =	<u>Р</u> А	Worst Worst Additio	Case Axial loa Case Unfacto onal Load =	ad = ored Axial Lo	oad =	39.92 kN 26.62 kN 0.00 kN	
Pad S	Size =	1.00 m dp >	c 0.60 r	m wd x	0.60	m wide.	
Take	self weight reliev	ed by overbu	den pressure	=	6.00	kN/m ²	
P =	= 2.16	+ 26.6	2 +	0.00	=	28.78 kN	
A =	= 0.36 m ²						
There	efore:-						
	<u>28.78</u> = 0.36	79	.93 kN/m ²	<	100	kN/m ²	
Therefore Pa	d is O.K.						
Use a 1	.00 m dp x	0.60 m wd	x 0.60 r	n wide pad	l footing		



Ref = Steel Beam B

Actions

	Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	I	Dist	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Cavity Wall	3.64			height	1.00	3.64	0.00	0.00
Bi Fold Doors	0.45			height	2.50	1.13	0.00	0.00
Pitched Roof	1.07		0.52	on slope	4.00	4.28	0.00	2.08
Flat Roof	0.65	0.60		3.6/2=	1.80	1.17	1.08	0.00
Loft	0.35	0.35		7/2=	3.50	1.22	1.23	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x133x30)			0.29		
					TOTALS:	11.73	2.31	2.08

Clear Span = 4047 mm. Bearings = 100 mm minimum each side Design span = 4147 mm (NB - Design span only - not for setting out on site) Span between restraints = 4147 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 3.11769 22.41 kN/m 15.83 +3.46 += Unfactored UDL = 16.11 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1 =4147 mm Design Point Load 2= 0.00 kN Distance from A to point load 2=0 mm Distance from B to point load 2=4147 mm Design Point Load 3= 0.00 kN Distance from A to point load 3=0 mm Distance from B to point load 3=4147 mm Design Point Load 4= 0.00 kN Distance from A to point load 4= 0 mm Distance from B to point load 4=4147 mm Beam analysed with simple supports:-Design Moment = 48.16 kNm ULS Shear Force At A = 46.46 kN ULS Shear Force At B = 46.46 kN

Design Moment =	48.16 kNm
Design. Shear A =	46.46 kN
Design. Shear B =	46.46 kN

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Title of Scheme	Lemsford Village Ha	all, Lemsford, Welwy	/n Garden City	Job No. 20655
DESIGN OF STEEL	MEMBER TO BS EN	<u>1993-1-1</u>	Ref = Steel Bea	am B
Design Data (Beam in pure bending Try a section Size =	From previous calcu ing only - no axial force	lation:- Moment, es) Design Sl	M _{y,Ed} = 48.16 hear = 46.46	kNm kN
1 No.	203x133x3	0 Desi Between R	gn Span = 4147 estraints = 4147	mm mm
Grade of Steel = Web thickness, t_w =	S355 6.40 mm	Flange thickness, t_f = Yield Strength, f_y =	= 9.60 mm 355 N/mm ²	(Table 3.1)
Classification of Sec	xtion			
Web = $d/t =$	172.40 =	26.94 mm	= 3	0.81
Flange = c =	133.90 -	15.20 -	6.40 =	56.15 mm
c/t = (From table 5.2, for r	$\frac{56.15}{9.60} =$ colled Sections2)	2 5.85 mm		
web Class 1 flange Class 1	limiting = 72 limiting = 9	$\epsilon = 58.58$ $\epsilon = 7.32$	3 mmTherefore2 mmTherefore	Class 1 Suitable Class 1 Suitable
Moment Resistance	As memb W _{pl,y} = 314000	er is Class 1 mm ³ W _{el,y} =	use Plastic 280000 mm ³	modulus. $\gamma_{M0} = 1$
$M_{c,Rd} = 314000$	x 355 1	_ 111.47	′ kNm	(Cl 6.2.5)
As 111.47	> 48.16	kNm	Moment resistance c	of section O.K.
Shear Resistance	Root radiu	us, r = 7.60	0 mm A =	3820 mm [∠]
$A_{v,z} = A - 2 b t_f -$	+ $(t_w + 2r) t$ $A_{v,z} =$	1456 mm ⁻	$\eta h_w t_w = 1103$	mm ²
$V_{pl,z,Rd} = A_{v,z} (f_y)$	$((3)^{0.5})) =$	298.52 kN	Combined bending/s as max. moment awa	shear not considered ay from max. shear.
As 298.52	> 46.46	kN	Shear resistance of s	section O.K.



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Title of Scheme	Lemsford Village	Hall, Lemsfo	ord, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEEL	MEMBER TO BS E ckling (Cont)	<u>N 1993-1-1</u>		Ref =	Steel Bea	ım B	
<u>h</u> =	<u>206.80</u> = 133.90	1.54	<	2	use buck. <i>(Table 6.5)</i>	curve)	b
Imperfection factor,	$\alpha_{LT} = 0.34$		$\beta =$	0.75			
$\phi_{LT} = 0.5$	$5 (1 + lpha_{LT} (\lambda_{LT} - \lambda_{LTO}))$	$)+\beta\lambda_{LT}^{2})$	=	1.40			
Reduction factor,	$\chi_{LT} = {\varphi_{LT} + (\varphi_{LT})^2}$	1 _{PLT} ² - βλ _{LT} ²) ^{0.5}	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < 1$	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT}=0.47$	0.47 <	1		0.47	<	0.51	
Conservatively, facto	or, f = 1		$\chi_{\text{LT,mod}} =$	0.47		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} W_{y}$	$_{/} f_{y} / \gamma_{M1} =$	52.93	kNm				
As 52.93	> 48.16	6 kNm	Lateral To	orsional B	uckling res	sistance	O.K.
Deflection Checks	Take av	verage partial	safety fact	ors as	1.425		
Equivalent unfactore	ed SLS udl to beam	= <u>33.80</u> 4.15	x x	8 4.15	_=	15.72	kN/m
I _{y=} 2.9E+07	⁷ mm ⁻ Major s	econd mome	ent of area	from sectio	n property	tables.	
Deflection = Finishes =	9.94 mm Brittle	Deflection	n limits =	Span 360	-	11.52	mm
As 9.94	< 11.52	2 mm	Deflectio	n due to S	SLS loading	g is	О.К.
Frequency Checks By inspection, frequ	ency checks not red	quired due to	low deflec	tions.			

Therefore use1 No.203x133x30S355section					
	Therefore use	1 No.	203x133x30	S355	section



BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam B

Bearing at End "A"	Factored	Reaction	=	46.46	kN ULS		
$f_k = K f_b^{\ \alpha} f_m^{\ p}$	Masonry	under pa	dstone	Aggregate	Concrete bl	ocks Grou	ıp 1
Unit size below = 21 Act. Comp. Str = 3.5	5 mm h x 50 N/mm ⁻	100 Norn) mm widtl n. Mean C	n omp. Strer	Shape Fact ngth, f _b =	tor, δ = 4.83 N	1.38 \/mm ⁻
From National Annexe, Tab	ole NA4,	K =	0.55	$\alpha =$	0.7	$\beta =$	0.30
Mortar Designation =	(iii)	Mortar T	ype M	4	$f_m =$	4.00	V/mm ²
$f_k = 2.51 \text{ N/mm}^2$	$f_d =$	f _k /γ _m		γ_m (Table I	NA1)	=	2.70
Bearing capacity, $N_{Rdc} =$	(1.2 + 0.4	$4(a_1/h_c))f_c$	_a A _b but not	: more thar	n 1.5f _d A _b .		
Edge distance, a_1 from edg	ge of bearing	g =	1300	mm			
Height, h _c , of bearing base	above floor	´ =	2000	mm			
Bearing capacity, $N_{Rdc} =$	1.46	f_dA_b	<	1.5f _d A _b .	use	1.46 f	_d A _b .
Bearing capacity, $N_{Rdc} =$	1.36	N/mm ⁻					
Bearing size = 40	00 mm x	100) mm x	215	mm deep		
46.46 x 10 ³ N							
40000 mm ² =	1.16	N/mm ⁻	<	1.36	N/mm ⁻	(D.K.
Therefore, use a 40	0 mm x	100)mm x	215	mm dp pad	dstone	

Bearing at End "B" Factored Reaction = 46.46 kN ULS $K f_{b}^{\alpha} f_{m}^{\beta}$ $f_k =$ Masonry under padstone Aggregate Concrete blocks Group 1 Shape Factor, $\delta =$ Unit size below = 215 mm h x 100 mm width 1.38 4.83 N/mm⁻ 3.50 N/mm⁻ Act. Comp. Str = Norm. Mean Comp. Strength, $f_{b} =$ From National Annexe, Table NA4, K = 0.55 $\alpha =$ 0.7 $\beta =$ 0.30 Mortar Designation = Mortar Type M 4 $f_m =$ 4.00 N/mm⁻ (iii) 2.51 N/mm⁻ $f_k =$ $f_d = f_k / \gamma_m$ γ_m (Table NA1) 2.70 = $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$. Bearing capacity, $N_{Bdc} =$ Edge distance, a_1 from edge of bearing = 1300 mm Height, h_c , of bearing base above floor = 2000 mm Bearing capacity, $N_{Bdc} =$ 1.46 f_dA_b. $1.46 f_{d}A_{b}$ $1.5f_dA_b$. use < 1.36 N/mm⁻ Bearing capacity, $N_{Bdc} =$ Bearing size = 200 mm x 100 mm x 215 mm deep 46.46 x 10° N 40000 mm² 1.16 N/mm² < 1.36 N/mm² O.K. = Therefore, use a 200 mm x 100 mm x 215 mm dp padstone







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	BS EN 1005			,		-		
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Consider studs hel Take Studs as simp	d at top and bly supporte	d bottom. ed.	Centres =	=	400	mm c/c		
Wind Loading =	0.00 k	N/m ²	(Estimate	ed)	$\gamma_{f} =$	1.5		
Horizontal Load pe	r post =	0.00	kN/m	Vertical S	span =	0.8	m	
Moment =	0.00 k	Nm	per post.					
Axial Load =	Perm Load	Var. Load		Distance(m)		Dead	Live	
_	(kN/m ²)	(kN/m ²)				(kN/m)	(kN/m)	
Pitched Roof	1.07	0.52		on slope	3.05	3.26	1.58	
						0.00	0.00	
						0.00	0.00	
L	ļ		<u>ļ</u>		TOTALS:	3.26	1.58	
$\gamma_f = 1.35$	Perm	1.5	Variable		ULS load	=	6.78	kN/m
Axial Load =	6.78 k	N/m	х	0.4	=	2.71	kN ULS	
Therefore, design post with following loads:-								
Major moment, My Minor moment, Mz Axial Compresion	/ = . = =	0.00 0 2.71	knm ULS knm ULS knn ULS					

See design over

Title of Scheme Lemsford	2n Stansted of <u>Structural</u> Engineers	d Floor, Unit 1, Birchanger Inde B Road, Bishops Stortford, Hert I So 9001 Email: mail@rcastru J, Welwyn Garden City	ustrial Estate, s, CM23 2TH 279) 506721 279) 506724 uctures.co.uk	Page 43 By CS Date Apr-19 Rev -
TIMBER POST TO BS EN 199	5			
Post Reference =	Ashlar Stud.			
From previous calculation :-	Major moment, My Minor moment, Mz Axial Compresion =	= 0.00 = C = 2.71) kNm ULS) kNm ULS kNN ULS	
Design span = 0.80	0 m = 800) mm		
Exposure condition = (Ground Floors, Cold Roofs, ex Load Duration =	Service Class 2 kternal timber frame w Medium Term	valls) (Imposed Floor Load	ding)	
$\begin{array}{ll} \underline{Design} & \text{Timber Grade} = \\ \text{Bending Strength, } f_{m,k} = \\ \text{Compressive Strength, } f_{c,0,k} = \\ & \text{Try} & 1 \\ \end{array}$	C24 24.0 N/mm ² 21.0 N/mm ² No. 100	Material = Solid Tim deep x 50	nber. Treate wide post	ed or untreated
Charring rate = General R Required fire rating = Effective size for design =	ate 30 minutes 100 mm deep	= o x 50	0.60) mm wide	mm/min post
Elastic Modulus , $W_{yy} =$ Elastic Modulus , $W_{zz} =$	8.3E+04 mm ³ . 4.2E+04 mm ³ .	K_h Factor = K_h Factor =	1.08 1.25	
Partial Safety factor, $\gamma_m = k_{mod} = 0.8$	1.3Type of E $k_{crit} =$ 1	Beam= Load Sha (No LTB due to boa	aring Irding)	$k_{sys} = 1.1$
<u>Design Strengths</u> Design bending strength f _{my,d} Design bending strength f _{mz,d} Design compressive strength <u>Design Stresses</u> Design Compressive Stress, c	$= k_{mod} \cdot k_{h}$ $= k_{mod} \cdot k_{h}$ of timber, f _{c,0,d} =	. k _{crit} . k _{sys} . f _{m,k} / γ _m . k _{crit} . k _{sys} . f _{m,k} / γ _m k _{mod} f _{c,0,} k / γ _m F/A =	= = = 0.54	17.62 N/mm ² 20.24 N/mm ² 12.92 N/mm ² N/mm ²
Design Bending Stress in y-y,	$\sigma_{m,y,d} =$	$M_y/W_{yy} =$	0.00	N/mm ²
Design Bending Stress in z-z,	$\sigma_{m,z,d} =$	$M_z/W_{zz} =$	0.00	N/mm ²

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Title of Scheme	Lemsford \	/illage Ha	all, Lemsfo	ord, Welwy	n Garden	City	Job No	20655
<u>TIMBER POST TO</u>) BS EN 1995	5						
Post Reference =	· /	Ashlar St	ud.					
Buckling about th Radius of gyratior	<u>e z-z axis</u> 1. iz =	14.43	Consider mm	⁻ 0.5L as f	ully restrai	ined by sh	eathing bo	bard
Eff. Length =	0.5 l	_ =	400	mm	$\lambda_z =$	27.71	Factor=	0.01696
$\lambda_{rel,z} = 0.47$ Therefore, use	$> \ \sigma_{c,o,d} \ k_{c,z} f_{c,o,d}$	0.30 +	(Slender) k _m	f _{m,y,d}	- +	$\frac{\sigma_{m,z,d}}{f_{m,z,d}}$	_ <	1
$\begin{array}{ll} \beta_{c} = & 0.20 \\ k_{z} = & 0.5 \; (1 + \\ k_{c,z} = & 1 \; / \; k_{z} \; + \end{array}$	Solid Timb $\beta_c (\lambda_{rel,z} - 0.3)$ $(k_z^2 - \lambda_{rel,z}^2)^{0.2}$	er. Treat 3) + λ _{rel,z} 5	ed or untr 2)	eated = =	0.63 0.96	k _m =	0.7	
<u>0.54</u> + 12.39	0.70 _	0.00 17.62	+	0.00	_ =	0.04	<	1 O.K.
Buckling about the y-y axisRadius of gyration, $i_y =$ 28.87 mmEff. Length =0.5 L =400 mm $\lambda_y =$ 13.86Factor = 0.01696								
$\lambda_{rel,y} = 0.23$ Therefore, use $\beta_c = 0.20$	 k _{c,y} f _{c,0,d} Solid Timb	er. Treat	$\sigma_{m,y,d}$ $f_{m,y,d}$ ed or untr	- + eated	k _n	a:) n	_ <	1
$k_y = 0.5 (1 + k_{c,y}) = 1 / k_y + 1 / k_y$	$\beta_{c} (\lambda_{rel,y} - 0.3) (k_{y}^{2} - \lambda_{rel,y}^{2})^{0.2}$	$\beta) + \lambda_{rel,y}$	2)	=	0.52 1.0 ⁻	k _m = 1	0.7	
<u>0.54</u> + 13.10	0.00	+	0.70	0.00	_ =	0.04	<	1 O.K.
The post is therefore. use	ore adequate	e at ULS	for direct (compress	ion and b	uckling ab	out both a	xes.
1 No.	100	х	50	Joist/s,	C24	Grade Ti	mber	

Centres = 400 mm

Actions

Ref = Steel Beam C Large Span

	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	D	ist	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x102x23	3			0.23		
					TOTALS:	6.03	1.19	1.97

Clear Span = 3930 mm. Bearings = 100 mm minimum each side Design span =4030 mm (NB - Design span only - not for setting out on site) Span between restraints = 4030 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 8.13 1.79 2.96181 12.88 kN/m ++= Unfactored UDL = 9.19 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1= 4030 mm Design Point Load 2= 0.00 kN Distance from A to point load 2= 0 mm Distance from B to point load 2=4030 mm Design Point Load 3= 0.00 kN Distance from A to point load 3= 0 mm Distance from B to point load 3= 4030 mm 0.00 kN Design Point Load 4= Distance from A to point load 4= 0 mm Distance from B to point load 4= 4030 mm Beam analysed with simple supports:-Design Moment = 26.15 kNm ULS Shear Force At A = 25.96 kN ULS Shear Force At B = 25.96 kN

Design Moment =	26.15 kNm
Design. Shear A =	25.96 kN
Design. Shear B =	25.96 kN

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Title of Scheme	Lemsford	Village Ha	all, Lemsfo	ord, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEE	EL MEMBER TO	<u>) BS EN '</u>	<u>1993-1-1</u>		Ref =	Steel Bea Large Sp	am C an	
Design Data	From provid		ation	Momont	М —	06 15	kNm	
<i>(Beam in pure ber</i> Try a section Size	nding only - no =	axial force	ation es)	Design St	near =	25.96	kN	
1 No.	20	03x102x23	3	Desi Between Re	gn Span = estraints =	4030 4030	mm mm	
Grade of Steel = Web thickness, t_w	S35 = 5.40 r	55 mm	Flange th Yield Stre	ickness, t _f = ngth, f _y =	= 9.30 355	mm N/mm ²	(Table 3.1)
Classification of S	ection							
Web = d/t =	169.40	=	31.37	' mm		= 3	0.81	
Flange = c =	101.80	-	15.20	-	5.40		40.6	mm
c/t =	40.60	=	2 4.37	'mm				
(From table 5.2, fo	r rolled Sectior	is2)						
web Class 1 flange Class 1	limiting = limiting =	72 9	= 3 = 3	58.58 7.32	mm mm	Therefore Therefore	Class 1 Class 1	Suitable Suitable
Moment Resistan	W _{pl,y} =	As membe 234000	er is mm³	Class 1 W _{el,y} =	use 207000	Plastic mm ^³	$\begin{array}{l} \text{modulus.} \\ \gamma_{\text{M0}} = \end{array}$	1
$M_{c,Rd} = 23400$	0 x 1	355	=	83.07	kNm		(Cl 6.2.5)	
As 83.07	~ >	26.15	kNm		Moment re	esistance c	of section	0.K.
Shear Resistance	F	Root radiu	s, r =	7.60	mm	Α =	2940	mm ²
$A_{v,z} = A - 2 b$	$t_f + (t_w + 2r) t$	$A_{v,z} =$	1238	3 mm ²	$\eta \; h_w t_w =$	915	mm ²	
$V_{pl,z,Rd} = A_{v,z}$	(f _y / ((3) ^{0.5})) = /Mo	=	253.76	6 kN	Combinec as max. m	l bending/s oment awa	shear not c ay from max	onsidered «. shear.
As 253.7	6 >	25.96	kN		Shear resi	stance of s	section	О.К.

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Title of Scheme	emsford Villag	e Hall, Lemsfo	ord, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEEL M Lateral Torsional Buck	IEMBER TO BS	<u>EN 1993-1-1</u>		Ref =	Steel Bea Large Spa	m C an	
<u>h</u> =	203.20 = 101.80	2.00	<	2	use buck. <i>(Table 6.5)</i>	curve	b
Imperfection factor, α_L	_T = 0.3	4	$\beta =$	0.75			
$\phi_{LT} = 0.5$ ($1 + \alpha_{LT} (\lambda_{LT} - \lambda_{LT})$	$_{TO})+\beta\lambda_{LT}^{2})$	=	1.73			
Reduction factor, χ	$\chi_{LT} = \frac{1}{\Phi_{LT} + 1}$	$\frac{1}{(\phi_{LT}^2 - \beta \lambda_{LT}^2)^{0.5}}$	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < 1$	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT}=0.37$	0.37 <	1		0.37	>	0.37	
Conservatively, factor,	f = 1		$\chi_{\text{LT,mod}} =$	0.37		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} W_y f_y$	$_{,}/\gamma_{M1}$ =	30.60	kNm				
As 30.60	> 26.	15 kNm	Lateral To	orsional B	uckling res	sistance	0.K.
Deflection Checks	Take	average partial	safety fact	ors as	1.425		
Equivalent unfactored $I_{y=}$ 2.1E+07 r	SLS udl to bear mm⁴ Major	$n = \underline{18.35}$ 4.03 second mome	x x ent of area t	8 4.03 from sectio	n property t	9.04 tables.	kN/m
Deflection = Finishes = E	7.01 mm Brittle	Deflectior	n limits =	Span 360	=	11.19	mm
As 7.01	< 11.7	19 mm	Deflectio	n due to S	LS loading	g is	0.K.
<i>Frequency Checks</i> By inspection, frequen	icy checks not r	equired due to	low deflec	tions.			

Therefore use	1 No.	203x102x23	S355	section

Actions

Ref = Steel Beam C Short Span

	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	Di	st	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	203x102x2	3			0.23		
					TOTALS:	6.03	1.19	1.97

Clear Span = 1670 mm. Bearings = 100 mm minimum each side Design span = 1770 mm (NB - Design span only - not for setting out on site) Span between restraints = 1770 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 8.13 1.79 2.96181 12.88 kN/m ++= Unfactored UDL = 9.19 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1 =1770 mm Design Point Load 2= 0.00 kN Distance from A to point load 2= 0 mm Distance from B to point load 2=1770 mm Design Point Load 3= 0.00 kN Distance from A to point load 3=0 mm Distance from B to point load 3=1770 mm Design Point Load 4= 0.00 kN Distance from A to point load 4= 0 mm Distance from B to point load 4=1770 mm Beam analysed with simple supports:-Design Moment = 5.04 kNm ULS Shear Force At A =11.40 kN ULS Shear Force At B = 11.40 kN

Desian Moment =	5.04 kNm
Design Shear A -	11 10 kN
Design, Shear A –	
Design. Snear B =	TT.40 KIN

STRU	CTURES	2nd Floor, Unit - Stansted Road, Bisho Institution Structural Ingineers	I, Birchanger Industrial Estate, os Stortford, Herts, CM23 2TH Telephone:(01279) 506721 Fax:(01279) 506724 nail: mail@rcastructures.co.uk	Page 50 By CS Date Apr-19 Rev A
Title of Scheme	Lemsford Village H	Hall, Lemsford, Wel	wyn Garden City	Job No. 20655
DESIGN OF STEEL	_ MEMBER TO BS EN	<u>l 1993-1-1</u>	Ref = Steel Bea Short Sp	am C an
Design Data	From previous calo	ulation:- Momen	t M — 5.04	kNm
(Beam in pure bend Try a section Size =	ling only - no axial fore	ces) Design	Shear = 11.40	×NN
1 No.	203x102x	23 De Between	esign Span = 1770 Restraints = 1770	mm mm
Grade of Steel = Web thickness, t _w =	S355 5.40 mm	Flange thickness, Yield Strength, f _y =	t _f = 9.30 mm = 355 N/mm ²	(Table 3.1)
Classification of Se	ction			
Web = $d/t =$	<u>169.40</u> = <u>5.40</u>	31.37 mm	= 3	0.81
Flange = c =	101.80 -	15.20 -	5.40 =	40.6 mm
c/t =	40.60 =	2 4.37 mm		
(From table 5.2, for	rolled Sections2)			
web Class 1 flange Class 1	limiting = 7 limiting =	$2 \varepsilon = 58.$ $9 \varepsilon = 7.$	58 mmTherefore32 mmTherefore	Class 1 Suitable Class 1 Suitable
Moment Resistance	e As mem W _{pl,y} = 23400	ber is Class 1 0 mm ³ W _{el,y} =	use Plastic 207000 mm ³	modulus. $\gamma_{M0} = 1$
$M_{c,Rd} = 234000$	x 355 1	_= 83.	07 kNm	(Cl 6.2.5)
As 83.07	> 5.04	kNm	Moment resistance of	of section O.K.
Shear Resistance	Root rad	ius, r = 7.	60 mm A =	2940 mm ²
$A_{v,z} = A - 2 b t_f$	+ $(t_w + 2r) t$ A _{v,z} =	= 1238 mm ²	$\eta h_w t_w =$ 915	, mm ⁻
$V_{pl,z,Rd} = A_{v,z} (f_{y})$, / ((3) ^{0.5})) =	253.76 kN	Combined bending/s as max. moment awa	shear not considered ay from max. shear.
As 253.76	> 11.40	kN	Shear resistance of	section O.K.

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Title of Scheme	Lemsford Village	Hall, Lemsfo	ord, Welwy	n Garden	City	Job No.	20655
DESIGN OF STEEL	MEMBER TO BS El	<u>N 1993-1-1</u>		Ref =	Steel Bea Short Spa	m C an	
<u>h</u> =	203.20 =	2.00	<	2	use buck. <i>(Table 6.5)</i>	curve	b
Imperfection factor,	$\alpha_{LT} = 0.34$		$\beta =$	0.75			
$\phi_{LT} = 0.5$	$5~(1+lpha_{ ext{LT}}~(\lambda_{ ext{LT}}$ - $\lambda_{ ext{LT0}})$	$+ \beta \lambda_{LT}^{2}$)	=	0.95			
Reduction factor,	$\chi_{LT} = $$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$	1 _ ² - βλ _{LT} ²) ^{υ.ອ}	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < $	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT} = 0.72$	0.72 <	1		0.72	<	1.07	
Conservatively, facto	or, f = 1		$\chi_{\text{LT,mod}} =$	0.72		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} W_{y}$	$_{,}f_{y}^{}/\gamma_{M1} = $	59.85	kNm				
As 59.85	> 5.04	kNm	Lateral To	orsional B	uckling res	sistance	0.K.
Deflection Checks	Take av	erage partial	safety fact	ors as	1.425		
Equivalent unfactore $I_{y=}$ 2.1E+07	ed SLS udl to beam 7 mm [∓] Major se	= <u>3.54</u> 1.77 econd mome	x x ent of area	8 1.77 from sectio	 n property t	9.04 tables.	kN/m
Deflection = Finishes =	0.26 mm Brittle	Deflection	i limits =	Span 360	=	4.92	mm
As 0.26	< 4.92	mm	Deflectio	n due to S	LS loading	g is	O.K.
<i>Frequency Checks</i> By inspection, freque	ency checks not req	uired due to	low deflec	tions.			

Therefore use	1 No.	203x102x23	S355	section
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BEARING DESIGN TO BS EN 1996-1-1

Reference =	Steel Beam C		Short Span	
$\frac{\text{Bearing at End "A"}}{f_{k} = K f_{b}^{\alpha} f_{m}^{\ p}}$	Factored Masonry	Reaction = under padstone	11.40 kN ULS Aggregate Concrete b	locks Group 1
Unit size below = Act. Comp. Str =	215 mm h x 3.50 N/mm ⁻	100 mm widt Norm. Mean C	h Shape Fac omp. Strength, f _b =	tor, δ = 1.38 4.83 N/mm ⁻
From National Ann	exe, Table NA4,	K = 0.55	$\alpha = 0.7$	$\beta = 0.30$
Mortar Designation	= (iii)	Mortar Type M	4 f _m =	4.00 N/mm ²
f _k = 2.51	N/mm ² $f_d =$	f _k /γ _m	γ_m (Table NA1)	= 2.70
Bearing capacity, N	$N_{\rm Rdc} = (1.2 + 0.4)$	4(a ₁ /h _c))f _d A _b but not	t more than 1.5f _d A _b .	
Edge distance, a1 f	rom edge of bearin	g = 600	mm	
Height, h _c , of beari	ng base above flooi	r = 2000	mm	
Bearing capacity, N	N _{Rdc} = 1.32	f _d A _b <	1.5f _d A _b . use	1.32 f _d A _b .
Bearing capacity, N	N _{Rdc} = 1.23	N/mm ⁻		
Bearing size =	300 mm x	100 mm x	150 mm deep	
11.40 x 10° N				
30000 mm ⁻	= 0.38	N/mm ⁻ <	1.23 N/mm ⁻	O.K.
Therefore, use a	300 mm x	100 mm x	150 mm dp pa	dstone

Reference = Steel Beams C

Bearing at End "B"	Factored Rea	action =	37.35 kN U	LS
$f_k = K f_b^{\ \alpha} f_m^{\ p}$	Masonry und	ler padstone	Aggregate Cond	rete blocks Group 1
Unit size below = 215	mm h x	100 mm widt	h Shap	e Factor, $\delta = 1.38$
Act. Comp. Str = 3.50	N/mm ⁻	Norm. Mean C	omp. Strength,	f _b = 4.83 N/mm ⁻
From National Annexe, Table Mortar Designation =	NA4, K = (iii) Mo	= 0.55 ortar Type M	$\begin{array}{c} \alpha = & 0.\\ 4 & f_m \end{array}$	$\begin{array}{rcl} 7 & \beta = & 0.30 \\ = & 4.00 \text{ N/mm}^2 \end{array}$
f _k = 2.51 N/mm ⁻	$f_d = 1$	k/γm	γ_m (Table NA1)	= 2.70
Bearing capacity, $N_{Rdc} =$	$(1.2 + 0.4(a_1))$	/h _c))f _d A _b but no	t more than 1.5f	dAb.
Edge distance, a_1 from edge	of bearing =	600) mm	
Height, $h_{\rm c},$ of bearing base a	bove floor =	2000) mm	
Bearing capacity, $N_{Rdc} =$	1.32 f _d A	-b <	1.5f _d A _b . us	e 1.32 f _d A _b .
Bearing capacity, $N_{Rdc} =$	1.23 N/r	nm ⁻		
Bearing size = 400	mm x	100 mm x	215 mm (deep
37.35 x 10 ³ N				
$40000 \text{ mm}^2 =$	0.93 N/r	mm ² <	1.23 N/mr	m ² O.K.
Therefore, use a 400	mm x	100 mm x	215 mm (dp padstone

Ref = Steel Beam D

Actions

	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	Di	st	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Pitched roof	1.07		0.52	on slope	3.80	4.06	0.00	1.97
Ashlar Wall	0.55			height	1.00	0.55	0.00	0.00
Ceiling Joists	0.35	0.35		6.8/2=	3.40	1.19	1.19	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	178x102x19	9			0.19		
					TOTALS:	5.99	1.19	1.97

Clear Span = 2900 mm. Bearings = 100 mm minimum each side Design span = 3000 mm (NB - Design span only - not for setting out on site) Span between restraints = 3000 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 8.08 1.79 2.96181 12.83 kN/m ++= Unfactored UDL = 9.15 kN/m Design Point Load 1= 0.00 kN Distance from A to point load 1 =0 mm Distance from B to point load 1 =3000 mm Design Point Load 2= 0.00 kN Distance from A to point load 2= 0 mm Distance from B to point load 2=3000 mm Design Point Load 3= 0.00 kN Distance from A to point load 3=0 mm Distance from B to point load 3=3000 mm Design Point Load 4= 0.00 kN Distance from A to point load 4= 0 mm Distance from B to point load 4=3000 mm Beam analysed with simple supports:-Design Moment = 14.43 kNm ULS Shear Force At A =19.24 kN ULS Shear Force At B = 19.24 kN

Design Moment =	14.43 kNm
Design. Shear A =	19.24 kN
Design. Shear B =	19.24 kN

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Title of Scheme	Lemsford Village Hall, Lems	ford, Welwyn Garden City Job No. 20655
DESIGN OF STEEL	<u>MEMBER TO BS EN 1993-1-1</u>	Ref = Steel Beam D
Design Data (Beam in pure bendi	From previous calculation:- ng only - no axial forces)	Moment, $M_{y,Ed} =$ 14.43 kNm Design Shear = 19.24 kN
1 No.	178x102x19	Design Span = 3000 mm Between Restraints = 3000 mm
Grade of Steel = Web thickness, t_w =	S355 Flange th 4.80 mm Yield Stre	hickness, $t_f =$ 7.90 mm ength, $f_y =$ 355 N/mm ² (Table 3.1)
Classification of Sec	tion	
Web = d/t =	$\frac{146.80}{4.80} = 30.5$	$\epsilon = 0.81$
Flange = c =	101.20 - 15.20	- 4.80 = 40.6 mm
c/t =	$\frac{40.60}{7.90} = 5.1$	4 mm
web Class 1 flange Class 1	$\begin{array}{llllllllllllllllllllllllllllllllllll$	58.58 mmTherefore Class 1Suitable7.32 mmTherefore Class 1Suitable
Moment Resistance	As member is W _{pl,y} = 171000 mm ³	Class 1 use Plastic modulus. $W_{el,y} = 153000 \text{ mm}^3 \gamma_{M0} = 1$
$M_{c,Rd} = 171000$	<u>x 355</u> =	60.71 kNm (Cl 6.2.5)
As 60.71	> 14.43 kNm	Moment resistance of section O.K.
Shear Resistance	Root radius, r =	7.60 mm A = 2430 mm ²
$A_{v,z} = A - 2 b t_f +$	$+ (t_w + 2r) t A_{v,z} = 98$	$_{39} \text{ mm}^2$ $\eta h_w t_w = 705 \text{ mm}^2$
$V_{pl,z,Rd} = A_{v,z} (f_y)$	$\frac{1}{2} \left((3)^{0.5} \right) = 202.7$	71 kN Combined bending/shear not considered as max. moment away from max. shear.
As 202.71	> 19.24 kN	Shear resistance of section O.K.

STRUCT	CA	2n Stansted The Institution of Structural Engineers	d Floor, Unit 1, E Road, Bishops	Birchanger Indu Stortford, Herts Telephone:(01: Fax:(01: I: mail@rcastru	ustrial Estate, s, CM23 2TH 279) 506721 279) 506724 uctures.co.uk	Page By Date Rev	57 CS Apr-19 A
Title of Scheme	Lemsford Vill	lage Hall, Lems	ford, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEE	L MEMBER TO	<u>BS EN 1993-1-1</u>		Ref =	Steel Bea	m D	
<u>h</u> =	177.80	= 1.76	<	2	use buck. <i>(Table 6.5)</i>	curve	b
Imperfection factor	, $\alpha_{LT} =$	0.34	$\beta =$	0.75			
$\phi_{LT} = 0$.5 (1 + α_{LT} (λ_{LT} -	λ_{LTO}) + $\beta \lambda_{LT}^{2}$)	=	1.39			
Reduction factor,	$\chi_{LT} = {\varphi_{LT}}$	$\frac{1}{1} + (\phi_{LT}^2 - \beta \lambda_{LT}^2)^{0.}$	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < $	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT}=0.48$	0.48	< 1		0.48	<	0.52	
Conservatively, fac	tor, f = 1		$\chi_{\text{LT,mod}} =$	0.48		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} V$	$V_y f_y / \gamma_{M1} =$	29.1	3 kNm				
As 29.13	> 1	4.43 kNm	Lateral T	orsional B	uckling res	sistance	O.K.
Deflection Checks	Tal	ke average partia	al safety fact	tors as	1.425		
Equivalent unfactor	red SLS udl to b	eam = 10.13	X	8		9.00	kN/m
I _{y=} 1.4E+C	o7 mm [∓] Ma	ajor second morr	x nent of area	from sectio	on property t	tables.	
Deflection = Finishes =	3.33 mr Brittle	n Deflectic	on limits =	Span 360	_	8.33	mm
As 3.33	<	8.33 mm	Deflectio	on due to S	SLS loading	g is	0.K.
Frequency Checks By inspection, freq	uency checks no	ot required due t	o low deflec	tions.			

Therefore use	1 No.	178x102x19	S355	section

BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam D

Bearing at End "A"	Factored	Reaction	=	19.24	kN ULS		
$f_k = K f_b^{\alpha} f_m^{\beta}$	Masonry	under pa	dstone	Aggregate	Concrete blo	ocks Gro	up 1
Unit size below = Act. Comp. Str =	215 mm h x 3.50 N/mm ⁻	100 Norn) mm widtl n. Mean C	n omp. Strer	Shape Fact ngth, f _b =	or, δ = 4.83	1.38 N/mm ⁻
From National Annexe, T	able NA4,	K =	0.55	$\alpha =$	0.7	$\beta =$	0.30
Mortar Designation =	(iii)	Mortar T	ype M	4	$f_m =$	4.00	N/mm ²
$f_k = 2.51 \text{ N/mr}$	m^{-} $f_{d} =$	f _k /γ _m		γ_m (Table I	NA1)	=	2.70
Bearing capacity, N_{Rdc} =	= (1.2 + 0.4	$4(a_1/h_c))f_c$	A _b but not	: more thar	ח 1.5f _d A _b .		
Edge distance, a1 from e	edge of bearing	g =	600	mm			
Height, h _c , of bearing ba	ise above floor	· =	2000	mm			
Bearing capacity, N_{Rdc} =	= 1.32	f_dA_b	<	1.5f _d A _b .	use	1.32	f _d A _b .
Bearing capacity, N_{Rdc} =	= 1.23	N/mm ⁻					
Bearing size =	300 mm x	100) mm x	150	mm deep		
19.24 x 10 [°] N							
30000 mm ² =	= 0.64	N/mm ⁻	<	1.23	N/mm ⁻		0.K.
Therefore, use a	300 mm x	100) mm x	150	mm dp pac	dstone	

Reference = Steel Beam C (large Span) & D

Bearing at End "B"	Factored Re	eaction =	45.19 kN UL	S
$f_k = K f_b^{\ \alpha} f_m^{\ \mu}$	Masonry une	der padstone	Aggregate Concr	ete blocks Group 1
Unit size below = 215	mm h x	100 mm widt	h Shape	e Factor, $\delta = 1.38$
Act. Comp. Str = 3.50	N/mm ⁻	Norm. Mean C	omp. Strength, f	_o = 4.83 N/mm ²
From National Annexe, Table Mortar Designation =	NA4, K (iii) Mo	= 0.55 ortar Type M	$\begin{array}{c} \alpha = & 0.7 \\ 4 & f_{m} = \end{array}$	$\beta = 0.30$ = 4.00 N/mm ²
f _k = 2.51 N/mm ⁻	$f_d =$	f _k /γ _m	γ_m (Table NA1)	= 2.70
Bearing capacity, $N_{Rdc} =$	(1.2 + 0.4)	₁ /h _c))f _d A _b but no	t more than 1.5f _d	A _b .
Edge distance, a_1 from edge	of bearing =	= 2000) mm	
Height, h_c , of bearing base a	bove floor =	2000) mm	
Bearing capacity, $N_{Rdc} =$	1.60 f _d /	A _b >	1.5f _d A _b . use	e 1.50 f _d A _b .
Bearing capacity, $N_{Rdc} =$	1.39 N/	mmf		
Bearing size = 450	mm x	100 mm x	215 mm d	еер
45.19 x 10 ³ N				
$45000 \text{ mm}^2 =$	1.00 N/	mm ² <	1.39 N/mm	0 ² O.K.
Therefore, use a 450	mm x	100 mm x	215 mm d	p padstone

Ref = Steel Beam E

Actions

	Permanent Action, G _k (kN/m ²)	Leading Variable Action, Q _{k 1} (kN/m ²)	Variable Action, Q _{k i} (kN/m ²)	Di	st	Permanent Action, G _k (kN/m)	Variable Action, Q _{k 1} (kN/m)	Variable Action, Q _{k i} (kN/m)
Pitched roof	1.07		0.52	nominal	0.40	0.43	0.00	0.21
Cavity Wall	3.64			height	3.00	10.92	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
						0.00	0.00	0.00
Self weight of beam	Using a	152x89x16				0.16		
					TOTALS:	11.50	0.00	0.21

Clear Span = 1000 mm. Bearings = 100 mm minimum each side Design span = 1100 mm (NB - Design span only - not for setting out on site) Span between restraints = 1100 mm Conservatively, use equation 6.10 = $1.35 \text{ G}_{\text{k}} + 1.5 \text{ Q}_{\text{k}1} + \Sigma \psi_{0i} 1.5 \text{ Q}_{\text{k}i}$ Conservatively, take $\psi_{0i} =$ 1.00 Design UDL= 0.00 0.31177 15.84 kN/m 15.53 ++= Unfactored UDL = 11.71 kN/m Design Point Load 1= 11.40 kN Steel Beam C (short span) Distance from A to point load 1 =90 mm Distance from B to point load 1 =1010 mm Design Point Load 2= 0.00 kN Distance from A to point load 2=0 mm Distance from B to point load 2=1100 mm Design Point Load 3= 0.00 kN Distance from A to point load 3=0 mm Distance from B to point load 3=1100 mm Design Point Load 4= 0.00 kN Distance from A to point load 4= 0 mm Distance from B to point load 4=1100 mm Beam analysed with simple supports:-Design Moment = 3.34 kNm ULS Shear Force At A =19.18 kN ULS Shear Force At B = 9.64 kN

Design Moment =	3.34 kNm
Design. Shear A =	19.18 kN
Design. Shear B =	9.64 kN

STRUC	CA Stanster The Institution of Structural Engineers	nd Floor, Unit 1, Birchanger Industrial Estate, d Road, Bishops Stortford, Herts, CM23 2TH Page 60 Telephone:(01279) 506721 By CS Date Apr-19 Email: mail@rcastructures.co.uk Rev A
Title of Scheme	Lemsford Village Hall, Lems	sford, Welwyn Garden City Job No. 20655
DESIGN OF STEEL	MEMBER TO BS EN 1993-1-1	<u>1</u> Ref = Steel Beam E
Design Data (Beam in pure bendi Try a section Size =	From previous calculation:- ing only - no axial forces)	Moment, $M_{y,Ed} =$ 3.34 kNm Design Shear = 19.18 kN
1 No.	152x89x16	Design Span = 1100 mm Between Restraints = 1100 mm
Grade of Steel = Web thickness, t _w =	S355 Flange 4.50 mm Yield St	thickness, $t_f = 7.70 \text{ mm}$ rength, $f_y = 355 \text{ N/mm}^2$ (Table 3.1)
Classification of Sec	tion	
Web = d/t =	$\frac{121.80}{4.50} = 27.0$	07 mm ε = 0.81
Flange = c =	88.70 - 15.20	0 - 4.50 = 34.5 mm
c/t = (From table 5.2. for r	$\frac{34.50}{7.70} = 4.4$	48 mm
web Class 1 flange Class 1	limiting = $72 \epsilon =$ limiting = $9 \epsilon =$	58.58 mmTherefore Class 1Suitable7.32 mmTherefore Class 1Suitable
Moment Resistance	$W_{pl,y} = 123000 \text{ mm}^3$	Class 1 use Plastic modulus. $W_{el,y} = 109000 \text{ mm}^3 \gamma_{M0} = 1$
$M_{c,Rd} = 123000$	<u>x 355</u> =	43.67 kNm (Cl 6.2.5)
As 43.67	> 3.34 kNm	Moment resistance of section O.K.
Shear Resistance	Root radius, r =	7.60 mm A = 2030 mm ²
$A_{v,z} = A - 2 b t_{f} - $	+ $(t_w + 2r) t$ A _{v,z} = 8	16 mm ² $\eta h_w t_w = 548 \text{ mm}^2$
$V_{pl,z,Rd} = A_{v,z} (f_y)$	$\frac{1}{((3)^{0.5})} = 167.5$	19 kN Combined bending/shear not considered as max. moment away from max. shear.
As 167.19	> 19.18 kN	Shear resistance of section O.K.

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Title of Scheme	Lemsford V	illage Hal	l, Lemsfo	ord, Welwy	ın Garden	City	Job No.	20655
DESIGN OF STEE	L MEMBER TC	BS EN 1	<u>993-1-1</u>		Ref =	Steel Bea	m E	
<u>h</u> =	<u>152.40</u> 88.70	=	1.72	<	2	use buck. (Table 6.5)	curve	b
Imperfection factor	, $\alpha_{LT} =$	0.34		$\beta =$	0.75			
$\phi_{LT} = 0$.5 (1 + α_{LT} (λ_{LT}		βλ _{LT} ²)	=	0.73			
Reduction factor,	$\chi_{LT} = \overline{\varphi_{I}}$	1 + (\psi_t -	βλ _{LT} ²) ^{0.5}	but	$\chi_{LT} < 1.0$	and	$\chi_{LT} < 1$	$\frac{1}{\lambda_{LT}^2}$
$\chi_{LT}=0.87$	0.87	< 1	l		0.87	<	2.05	
Conservatively, fac	tor, $f = 1$			$\chi_{\text{LT,mod}} =$	0.87		$\gamma_{M1} =$	1.00
$M_{b,Rd} = \chi_{LT,mod} W$	$V_y f_y / \gamma_{M1} =$		38.02	! kNm				
As 38.02	>	3.34 k	Nm	Lateral To	orsional B	uckling res	sistance	O.K.
Deflection Checks	Ta	ake avera	ge partial	safety fact	ors as	1.425		
Equivalent unfactor	red SLS udl to	beam = _	2.34	x	8	=	15.49	kN/m
I _{y=} 8.3E+0	06 mm [∓] N	lajor seco	nd mome	x ent of area f	from sectio	n property t	tables.	
Deflection = Finishes =	0.17 m Brittle	im [Deflectior	n limits =	Span 360	_	3.06	mm
As 0.17	<	3.06 r	nm	Deflection	n due to S	LS loading	g is	O.K.
Frequency Checks	: uency checks i	not require	ed due to	low deflect	tions.			

Therefore	1 N.a	150,00,10	0055	aadian
i nereiore use	I NO.	152x89x16	5355	section

BEARING DESIGN TO BS EN 1996-1-1

Reference = Steel Beam E

Bearing at End "A"	Factored	Reactior	า =	19.18	kN ULS		
$f_k = K f_b^{\ \alpha} f_m^{\ \beta}$	Masonry	under pa	adstone	Aggregate	e Concrete bl	ocks Grou	ıp 1
Unit size below = 215 Act. Comp. Str = 3.50	5 mm h x) N/mm ⁻	100 Norr	0 mm widt m. Mean C	h omp. Strer	Shape Fac ^a ngth, f _b =	tor, δ = 4.83 Γ	1.38 \/mm ⁻
From National Annexe, Table	e NA4,	K =	0.55	$\alpha =$	0.7	$\beta =$	0.30
Mortar Designation =	(iii)	Mortar T	уре М	4	$f_m =$	4.00	√/mm²
f _k = 2.51 N/mm ²	$f_d =$	f _k /γ _m		γ_m (Table I	NA1)	=	2.70
Bearing capacity, $N_{Rdc} =$	(1.2 + 0.4)	4(a ₁ /h _c))f	_d A _b but no	t more thai	n 1.5f _d A _b .		
Edge distance, a1 from edge	e of bearing	g =	C	mm			
Height, h _c , of bearing base a	above floor	=	2000	mm			
Bearing capacity, $N_{Rdc} =$	1.20	f_dA_b	<	1.5f _d A _b .	use	1.20 f	_d A _b .
Bearing capacity, $N_{Rdc} =$	1.12	N/mm ⁻					
Bearing size = 400) mm x	10	0 mm x	215	mm deep		
19.18 x 10° N							
40000 mm ⁻ =	0.48	N/mm ⁻	<	1.12	N/mm ⁻	(D.K.
Therefore, use a 400) mm x	10	0 mm x	215	mm dp pao	dstone	

Bearing at End "B" Factored Reaction = 9.64 kN ULS $f_k = K f_b^{\alpha} f_m^{\beta}$ Masonry under padstone Aggregate Concrete blocks Group 1 Unit size below = 215 mm h x 100 mm width Shape Factor, $\delta =$ 1.38 3.50 N/mm⁻ Act. Comp. Str = Norm. Mean Comp. Strength, $f_{b} =$ 4.83 N/mm⁻ $\beta =$ From National Annexe, Table NA4, K = 0.55 $\alpha =$ 0.7 0.30 4.00 N/mm⁻ Mortar Designation = Mortar Type M 4 (iii) $f_m =$ 2.51 N/mm⁻ 2.70 $f_k =$ $f_d =$ f_k/γ_m γ_m (Table NA1) = Bearing capacity, $N_{Bdc} =$ $(1.2 + 0.4(a_1/h_c))f_dA_b$ but not more than $1.5f_dA_b$. Edge distance, a_1 from edge of bearing = 0 mm Height, h_c , of bearing base above floor = 2000 mm Bearing capacity, $N_{Rdc} =$ $1.20 f_{d}A_{b}$ $1.5f_dA_b$. 1.20 f_dA_b. < use 1.12 N/mm⁻ Bearing capacity, $N_{Bdc} =$ Bearing size = 300 mm x 150 mm deep 100 mm x x 10³ N 9.64 1.12 N/mm² 30000 mm² 0.32 N/mm² O.K. <= 300 mm 100 mm x 150 mm dp padstone Therefore, use a Х

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Title of Scheme	Job No.	20655			

WALL LOADINGS FOR STRIP FOUNDATION CHECK

	Perm.G _k (kN/m²)	Var. Q _k (kN/m²)	Length/Height		Perm. Gk (kN/m)	Var. Q _k (kN/m)
Pitched Roof	1.07	0.52	on slope	4.00	4.28	2.08
Loft	0.35	0.35	6.7/2=	3.35	1.17	1.17
Flat Roof	0.65	0.60	5.4/2=	2.70	1.75	1.62
Cavity Wall	3.64		height	2.50	9.10	0.00
Wall below DPC	6.58		height	0.50	3.29	0.00
					0.00	0.00
					0.00	0.00
					0.00	0.00

Total

= 19.59 4.87

MINIMUM FOUNDATION WIDTH

Take Ground bearing Pressure as:-

100 kN/m²

TBC on site.

Wall Ref.	Perm. Load G _k (kN/m)	Variable Load Q _k (kN/m)	Total Load (kN/m)	Width Required (m)	Width Used (m)	Check
Rear Wall	19.59	4.87	24.46	0.24	0.45	<u>O.K.</u>