32 The Green, Richmond, London Mr Shaun Brown

224260

Structural Calculations

 $224260 - \mathsf{MNP} - \mathsf{XX} - \mathsf{XX} - \mathsf{CA} - \mathsf{S} - 0001$

Status S2

Rev P01

November 2024



mason navarro pledge	Project 32 The Green, Richmond,	Job No. 22420	60	Sheet / Rev.	i / P01
	London	Design Element N/A			
	ion	Calc. by	Date		Chk'd by
		СМ	18/1	1/2024	

Revision History

Revision	Date	Description
P01	18/11/2024	Initial issue.

Document Verification

	Name / Qualifications	Signature
Prepared by	Caspar More	Carper More
Checked by	James Lennon	
Approved by	James Lennon	

These calculations have been prepared for our client in accordance with the terms and conditions of our appointment for 32 The Green, Richmond, London. Mason Navarro Pledge Ltd cannot accept any responsibility for any use of or reliance on the contents of these calculations by any third party.

Appendix A – Drawing



32 The Green, Richmond 224260 - MNP - XX - XX - RP - S - 0001



General

Key

 \rightarrow

Note: Allow for fixing all joists to main beams & walls with 90 x 90 x 60 proprietary

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
- Do not scale from any of the structural drawings.
 All dimensions to be verified on site and any discrepancies should be highlighted.
- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

 Beam G1 to be strengthened with steel flitch plate (size TBC). See typical detail on drawing (224260-MNP-XX-XX-DR-S-2105).

 Cintec anchors installed at high level to restrain external elevation.

steel cleats

 Note:

 Allow for internal cracking to be repaired with helical ties with 500mm embedment length either side of the cracks.

 Note:

 Once timber paneling is removed in accordance with conservation architects requirement. MNP to inspect, an allowance for internal cracking to be repaired with Helical ties, with a 500mm embedment length either side of cracks. Furthermore allow for any larger cracks to be repaired with cintec anchors.

Existing joists

P01PRELIMINARY ISSUE15.11.24CMREVCOMMENTSDATECHKSTATUSCOMMENTSCOMMENTSCHK

PRELIMINARY



SHAUN BROWN

32 THE GREEN, RICHMOND LONDON

PROJECT

DRAWING TITLE PROPOSED GROUND FLOOR GA

SCALE @ A1	DRAWN BY	DATE
1.25	JSE	31 07 24
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224260	50	P 01
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224260-M	NP-XX-00-DR-\$	S-1100



General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
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- 1.5. All materials to comply with the relevant British Standard.

strapping to joists - see pattress (224260-MNP-XX-XX-DR-S-2105).

- Beam F1 to be strengthened with steel flitch plate (size TBC). see typical detail on drawing (224260-MNP-XX-XX-DR-S-2105).

Installing pattress plate and (224260-MNP-XX-XX-DR-S-2105).

 Installing pattress plate and strapping to joists - see pattress (224260-MNP-XX-XX-DR-S-2105).

strapping to joists - see pattress (224260-MNP-XX-XX-DR-S-2105).

(224260-MNP-XX-XX-DR-S-2105).

strapping to joists - see pattress (224260-MNP-XX-XX-DR-S-2105).



ties with 500mm embedment length either side of the cracks.

Note:

Once timber paneling is removed in accordance with conservation architects requirement. MNP to inspect, an allowance for internal cracking to be repaired with Helical ties, with a 500mm embedment length either side of cracks. Furthermore allow for any larger cracks to be repaired with cintec anchors.

P01 PRELIMINARY ISSUE 15.11.24 CM DATE CHK REV COMMENTS STATUS

PRELIMINARY



RICHMOND LONDON

DRAWING TITLE PROPOSED FIRST FLOOR GA

SCALE @ A1	DRAWN BY	DATE
1:25	JSE	31.07.24
MNP No.	STATUS CODE	REV
224260	S0	P 01
Ref No. 224260 - M	NP-XX-01-DR-	S-1101



Beam S3 to be strengthened with steel flitch plate (size TBC). See typical detail on drawing (224260-MNP-XX-XX-DR-S-2105). General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
- 1.3. Do not scale from any of the structural drawings. All dimensions to be verified on site and any discrepancies should be highlighted.
- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

Beam S1 to be strengthened with steel flitch plate (size TBC).
 See typical detail on drawing (224260-MNP-XX-XX-DR-S-2105).

Key	
	Existing joists
0_>	Existing engineered timber JJ joists
	JJ JOISTS

Note: Allow for fixing all joists to main beams & walls with 90x90x60 proprietary steel cleats

Note: Allow for the plate to be fixed down to masonry wall head with 12mm threaded rod & Rawl KEM - II resin. Fixing @ 500mm and embedment 300mm into masonry.

P01 PRELIMINARY ISSUE 15.11.24 CM DATE CHK REV COMMENTS STATUS

PRELIMINARY



RICHMOND LONDON

DRAWING TITLE PROPOSED SECOND FLOOR GA

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SCALE @ A1	DRAWN BY	DATE

 Beam S2 to be strengthened with steel flitch plate (size TBC). see typical detail on drawing (224260-MNP-XX-XX-DR-S-2105).



and install new steel plate (80mm (H) X 8mm (T) X 1.5m (L)) fixed to the side of hip member with M12 coach screws @ 200mm staggered centre

Ceiling joists to be

raised at	pove purlins	r	raised above purlins
al ties es m	Allow for strengthening damaged valley beam with steel PFC sections bolted with M12 bolts @ 250mm staggered centres. Exact repair TBC	Install steel plates (size TBC) on both sides of 4no. split rafters. Plates to be fixed with M12 coach screws @ 150mm staggered centres.	Proprietary steel strap bent to slope and installed to hold purlin back to masonry
		• • • • • • • • • • • • • • • • • • •	
· · · · · · · · · · · · · · · · · · ·			••••••••••••••••••••••••••••••••••••••
i			

Ceiling joists to be

Ceiling joists to be raised above purlins

Ceiling joists to be raised above purlins

Install new rafter alongside existing 'cut' rafter. General

1.1. Refer to MNP drawing S-000 for full notes.

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- 1.5. All materials to comply with the relevant British Standard.

- Chimney stack above to be dismantled & rebuilt - see planning reference 24/1992/ HOT & 24/1993/ LBC

2101 D

B 2100

> Strengthen purlin with 1No. steel 150 x 90
> PFC 24 channel sections fixed to existing purlin with M12 coach screws @ 250mm staggered centres- see detail (2106.4) on drawing (224260-MNP-XX-XX-DR-S-2106).

Allow for installing helical ties into joints every 3 courses embedment to be 500mm either side of cracking.

 Proprietary steel strap bent to slope and installed to hold purlin back to masonry

 Allow for installing helical ties into joints every 3 courses embedment to be 500mm either side of cracking.

Chimney stack above to be dismantled & rebuilt - see planning reference 24/1992/ HOT & 24/1993/ LBC

 Hip and corner strengthened with 120 x 120 SHS 10 bolted to underside of hip member. Strengthening steelwork to legs fixed to the wall head & strapped to the internal face of the wall - see detail on drawing (224260-MNP-XX-XX-DR-S-2106).

▲ B 2100



<u>Note</u>: Install additional timbers 100 x 75 C24 around all roof dormers/ roof lights



DRAWING TITLE PROPOSED SECOND FLOOR CEILING GA

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224260	S0	P 01
NP No.	STATUS CODE	REV
:25	JSE	31.07.24
CALE @ A1	DRAWN BY	DATE

224260-MNP-XX-03-DR-S-1103



General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
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- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

Note: Allow for internal cracking to be repaired with helical ties with 500mm embedment length either side of the cracks.



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PROJECT 32 THE GREEN, RICHMOND LONDON

DRAWING TITLE PROPOSED BASEMENT GA

SCALE @ A1	DRAWN BY	DATE
1:25	JSE	31.07.24
MNP No.	STATUS CODE	REV
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Ref No. 224260 - M	NP-XX-B1-DR-	5-1000

224260-MNP-XX-B1-DR-S-1000



Elevation A-A

Elevation B-B

General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
- 1.3. Do not scale from any of the structural drawings. All dimensions to be verified on site and any discrepancies should be highlighted.
- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

Provisional items

- Allow for • Cutting out and replacing 2m² of damaged brickwork. replace with bricks
- that match the existing. (area taken over total area of wall).
- Allow for resettling 3No. displaced lintels.

P01 PRELIMINARY ISSUE 15.11.24 CM DATE CHK REV COMMENTS STATUS PRELIMINARY mason navarro pledge Consulting Civil and Structural Engineers LONDON · MANCHESTER · HITCHIN 0203 9265613 0161 8701197 01462 632012

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JSE

STATUS CODE

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Ref No. 224260 - MNP- XX - 02 - DR-S-2100

DATE

REV

P 01

31.07.24

CLIENT

PROJECT

DRAWING TITLE

SCALE @ A1

1:50 MNP No.

224260

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32 THE GREEN,

RICHMOND LONDON

PROPOSED ELEVATIONS

A-A & B-B

Elevation C-C

General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
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- 1.5. All materials to comply with the relevant British Standard.

224260

S0

Ref No. 224260 - MNP-XX - 02 - DR-S-2101

P 01

Typical Flitch Plate Detail Scale 1:5

Typical pattress plate detail 2 - Joist parallel to wall

General

1.1. Refer to MNP drawing S-000 for full notes.

- 1.2. This drawing is to be read in conjunction with all Architect's, Engineer's and Services Engineer's drawings and specifications.
- 1.3. Do not scale from any of the structural drawings. All dimensions to be verified on site and any discrepancies should be highlighted.
- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

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RICHMOND

REV COMMENTS

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15.11.24 CM DATE CHK

DATE

PRELIMINARY

P01 First issue

SCALE @ A1 DRAWN BY AS MNP

FLOOR STRUCTURE

REPAIR DETAILS

AS NOTED	JSE	13.11.24	
MNP No.	STATUS CODE	REV	
224260	S0	P 01	
Ref No. 224260 - MNP- XX - XX - DR - S - 2105			

Steelwork junction - See detail 2106.1 & 2106.2 -----

Plan of hip corner steelwork Scale 1:10

Detail 2106.6 - 120x120SHS Leg Support Scale 1:5

Scale 1:5

Scale 1:5

General

1.1. Refer to MNP drawing S-000 for full notes.

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- 1.3. Do not scale from any of the structural drawings. All dimensions to be verified on site and any discrepancies should be highlighted.
- 1.4. All waterproofing to the Architect's details.
- 1.5. All materials to comply with the relevant British Standard.

DRAWING TITLE ROOF STRUCTURE **REPAIR DETAILS**

SHAUN BROWN

32 THE GREEN,

RICHMOND

P01 First issue REV COMMENTS

STATUS

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PROJECT

224260	S0	P 01
MNP No.	STATUS CODE	REV
AS NOTED	JSE	13.11.24
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224260-MNP-XX-XX-DR-S-2106

Appendix B – Calculations

32 The Green, Richmond 224260 - MNP - XX - XX - RP - S - 0001

Project 32 The Green Sheet / Rev. Job No. 24260 1 Richwood Design Element tructure mason navarro pledge punlips &)estan of Rafters. Calc. Title Date Chk'd by Calc. by UN JL 8 2 Design of punchin & Rafters, 0.98 lin wor 1.82 BH 2.21 BH 2.18 2.00 of Pu 0.86 3.92m Up SH 2.21 High level wind mm 2.00 Typical purlin BH 2.26 BH 2.16 he 1.86 0.98 Purtin Z.07m - (0.98 2.00 2.00 0.98 0.98 2 an (NTS) MAN max load of members. guerdeflect alot on site. practically Pholins www.mnp.co.uk © Mason Navarro Pledge

mason navarro pledge	^{Project} 32 The Green, Richmond, London	Job No. 2242 Design Element Ro	260 pof Structure	Sheet / 2	
^{Calc.} Title Loading		Calc. by	Date 18	/11/24	Chk'd by JL
Loadin	σ				
Roof	ь				
Dead					
Existin	g Loads			Units	
Tiles			0.70	kN/m ²	
Softwo	ood Batterns		0.02	kN/m ²	
Rafters	5		0.08	kN/m²	
		SUM	0.80	kN/m ²	
Additio	onal Historic Loads				
Lath +	Plaster		0.31	kN/m ²	
		SUM	0.31	kN/m ²	_
Additio	onal Future Loads				
Kingsp	an Membrane		0.00	kN/m ²	
2x50m	m Insulation		0.03	kN/m²	
Allow f	for Max of 15mm				
Fire Bo	oard or Lath +		0.04	2	
Plaster	·	<u></u>	0.31	KN/m	
		SOM	0.34	KIN/m	
live					
Snow			0 30	kN/m ²	
		SUM	0 30	kN/m ²	

	mnd	Project 32 The Green,	طمم	Job No. 224	4260	Sheet / Rev. 3	
mas	on navarro pledge	Richmond, Lond	aon	Design Element	Roof Structure	Э	
Calc. Title	Loading			Calc. by	Date 18	8/11/24	Chk'd by JL
	Ceiling						
	Dead						
	Existing L	oads					
	Ceiling Joi	ists			0.06	kN/m ²	
				SUM	0.06	kN/m ²	
	A .] .]!+!						
	Additiona	II HISTORIC LOADS			0.21	LINI /	
	Lath + Pla	ster		CLINA	0.31	kiv/m	
				SUIVI	0.31	KN/M	
	Additiona	l Future Loads					
	200mm R	oll of Insulation			0.02	kN/m ²	
	Allow for Fire Board	Max of 15mm d or Lath +					
	Plaster				0.31	kN/m ²	
				SUM	0.33	kN/m ²	
	Live						
	Access				0.25	kN/m ²	
				SUM	0.25	kN/m ²	

mnd	^{Project} 32 The Green,	Job No. 224260		Sheet / Rev. 4	
mason navarro pledge	Richmond, London	Design Element Roof S	Structure		
Calc. Title Loading		Calc. by CM	Date 18/1	1/24	Chk'd by JL

	Existing	Historical Loads	Future Loads	Units
Purlin Span	2.80	2.80	2.80	m
Purlin Midspan	1.40	1.40	1.40	m
Loading width Upper <mark>(</mark> On Slope)	2.59	2.59	2.59	m
Loading width Lower (On Slope)	1.00	1.00	1.00	m
Loading width Upper <mark>(</mark> On Plan)	1.83	1.83	1.83	m
Loading width Lower (On Plan)	0.70	0.70	0.70	m
Loading width going onto Purlin (On Slope)	3.59	3.59	3.59	m
Loading width going onto Purlin (On Plan)	2.53	2.53	2.53	m
Density of Purlin, Softwood	4.20	4.20	4.20	kN/m ³
Purlin Depth	115	115	115	mm
Purlin Width	200	200	200	mm
Self weight	0.10	0.10	0.10	kN/m
Loading onto Purlin				
Dead Load				
	2.07	2.00	4.10	L.N. /
	2.87	3.98	4.10	KIN/ M
Roof - Min	0.80	1.11	1.14	kN/m
Celling	0.12	0.08	0.72	KIN/ M
Live Load				
Roof - Max	0.76	0.76	0.76	kN/m
Roof - Min	0.21	0.21	0.21	kN/m
Ceiling	0.46	0.46	0.46	kN/m
Left Hand Support Reaction				
Dead				
Full UDL	1.28	2.51	2.60	kN
Partial UDL	0.73	1.01	1.03	kN
Triangluar Load	1.93	2.68	2.76	kN
Self Weight	0.14	0.14	0.14	kN
Live				
Full UDL	0.93	0.93	0.93	kN
Partial UDL	0.19	0.19	0.19	kN
Triangular Load	0.51	0.51	0.51	kN

mnp Project 32 The Gr	een,	Job No.	24260	Rev. 5		
ason navarro pledge	, London	Design Element	Roof Structu	re		
Loading		Calc. by	M Date 1	8/11/24	Chk'd by	
Right Hand Support Reaction						
Dead						
Full UDL	1.2	8	2.51	2.60	kN	
Partial UDL	2.1	8	3.02	3.10	kN	
Triangular Load	0.4	8	0.67	0.69	kN	
Self Weight	0.1	4	0.14	0.14	kN	
Live						
Full UDL	0.9	3	0.93	0.93	kN	
Partial UDL	0.5	8	0.58	0.58	kN	
Triangular Load	0.1	3	0.13	0.13	kN	
Totals						
LHS Reaction (Dead)	4.0	8	6.34	6.53	kN	
LHS Reaction (Live)	1.6	4	1.64	1.64	kN	
RHS Reaction (Dead)	4.0	8	6.34	6.53	kN	
RHS Reaction (Live)	1.6	4	1.64	1.64	kN	
RHS Reaction (Both)	5.7	2	7.98	8.17	kN	
LHS Reaction (Both)	5.7	2	7.98	8.17	kN	
RHS Reaction (Factored)	7.9	6	11.01	11.27	kN	
LHS Reaction (Factored)	7.9	6	11.01	11.27	kN	
LHS Reaction PL onto Hip						
2X PL onto Hip (Dead)	8.1	6	12.67	13.06	kN	
2X PL onto Hip (Live)	3.2	8	3.28	3.28	kN	
2X PL onto Hip (Both)	11.4	43	15.95	16.34	kN	
2X PL onto Hip (Factored)	15.9	93	22.03	22.55	kN	
Percentage Difference	Historio Existing	cal to Loads	Existing to Future Loads	Historical to Future Loads	3	
2X PL onto Hip (Dead)	-36	5	60	3	%	
2X PL onto Hip (Live)	0		0	0	%	
2X PL onto Hip (Both)	-28	3	43	2	%	
2X PL onto Hip (Factored)	-28	3	42	2	%	
Note:						
The point loads from the purlins onto steelwork in the hipped corners. Wh the historical and future loads, there	o the hip m en compar is only a v	ember ing the ery sm	are to be use percentage (all increase.	ed to design th difference bety	ne ween	

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Sheet / Rev. Job No. Project 7 32 The Green 724260 Richmond Design Element mason nava pledge Nr.C. rear Chk'd by Calc. Title Calc. by Date to hip 0 prove 2 W JL de pport to hip meni Corne New Stec P

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	mnp	Project 32 The Green, Richmond	Job No. 2242	60	Sheet / Rev.	8	/ P01
T	eson navarro piedge	,	Design Element Roof \$	Structure			
Calc. Title Su	pport to Hip		Calc. By CM	Date 16/09)/2024	Chk'c	з By JL

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

S2 The Green, Richmond Design Element Roof Structure Image: Support to Hip Calc. By CM Date CM Date 16/09/2024 Analysis results Maximum moment Mamx = 0 kN Mamme = -46 kNm Maximum shear Vmmx = -22.6 kN Vmm = -23.4 kN Deflection Xmmx = 0 kN RA_min = 0 kN Maximum reaction at support A RA_max = 0 kN RA_min = 0 kN Maximum reaction at support B RB_max = 23.4 kN RB_max = 23.4 kN Unfactored permanent load reaction at support B RB_max = 13.7 kN Unfactored variable load reaction at support B Section details Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Steel grade Image: Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance VcRe = 439.5 PASS - Design shear resistance VcRe = 439.5 Check bending moment Mcs = 46 kMm Des.bending resistance Mcse = 62.2 PASS - Design bending resistance moment exceeds design ber PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Sec	Chk'd By JL N	Date 16/09/2024	Design Element Roof S Calc. By CM	, Richmond	32 The Green,	meson navairo pindge
Support to Hip Calc. By Date CM 16/09/2024 Analysis results Maximum moment Mmm = 0 kNm Mmm = -46 kNm Maximum shear Vmm = -22.6 kN Vmm = -23.4 kN Deflection Ømm = 0 kNm Ra.min = 0 kN Maximum reaction at support A RA.min = 0 kN Ra.min = 0 kN Maximum reaction at support B RB.max = 23.4 kN RB.min = 23.4 kN Unfactored permanent load reaction at support B RB.max = 23.4 kN RB.min = 23.4 kN Unfactored permanent load reaction at support B RB.max = 23.4 kN RB.min = 23.4 kN Unfactored permanent load reaction at support B RB.max = 23.4 kN RB.min = 23.4 kN Unfactored permanent load reaction at support B RB.min = 3.3 kN Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: standard control Image: standard control Steel grade Image: standard control Class 1 Section classification Class 1 Check shear - Section 62.5 Design shear resistance VcArd = 439.5 Design bending moment Mez = 46 kNm Des.bending resistance exceeds design	Chk'd By JL N	Date 16/09/2024	Calc. By			
Analysis results Maximum moment Mmax = 0 kNm Mmin = -46 kNm Maximum moment Mmax = 0 kNm Mmin = -46 kNm Maximum shear Vmax = -22.6 kN Vmin = -23.4 kN Deflection Simar = 4.9 mm Sim = 0 kN Maximum reaction at support B Ra.max = 0 kN Ra.min = 0 kN Maximum reaction at support B Ra.max = 0 kN Ra.min = 23.4 kN Unfactored variable load reaction at support B Ra.max = 0 kN Ra.min = 23.4 kN Unfactored variable load reaction at support B Ra.min = 23.4 kN Ra.min = 23.4 kN Unfactored variable load reaction at support B Ra.min = 23.4 kN Ra.min = 23.4 kN Section details Section tetails Section tetails Section tetails Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section classification Class 1 Check shear - Section 62.6 Design shear resistance Vera = 23 kN PASS - Design shear resistance Vera = 439.5 PASS - Design shear resistance exceeds design PASS - Design shear resistance exceeds design PASS - Design shear resistance exceeds design berding resistance moment exceeds design berding resistance moment exceeds design berding resistance moment exceeds design berding resista	n	16/09/2024	Саіс. Ву			
Analysis results Maximum moment Mmax = 0 kNm Mmin = -46 kNm Maximum shear Vmax = -22.6 kN Vma = -23.4 kN Deflection $\delta_{max} = 4.9 \text{ mm}$ $\delta_{mn} = 0 \text{ mm}$ Maximum reaction at support A Ra.max = 0 kN Ra.min = 0 kN Maximum reaction at support B Re.max = 2.3.4 kN Re.min = 2.3.4 kN Unfactored permanent load reaction at support B Re.Permanent = 13.7 kN Unfactored variable load reaction at support B Section details Section details Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance V.EM = 433.5 Design shear force VEW = 23 kN Design shear resistance V.EM = 439.5 PASS - Design shear resistance exceeds design Check bending moment - Section 6.2.5 Design shear resistance exceeds design PASS - Design bending resist.moment M.EM = 62.2 PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads <td>n N</td> <td></td> <td></td> <td></td> <td></td> <td>Support to Hip</td>	n N					Support to Hip
Maximum moment Maximum moment Maximum shear $V_{max} = 0 \text{ kNm}$ $M_{max} = 0 \text{ kN}$ $M_{max} = -23.6 \text{ kN}$ $V_{max} = -23.4 \text{ kN}$ $M_{max} = 0 \text{ kM}$ $M_{max} = 0 \text{ kM}$ $M_{max} = 0 \text{ kM}$ $M_{max} = 0 \text{ kN}$ $M_{max} = 0 \text{ kN}$ $R_{max} = 3.3 \text{ kN}$ Section dassification Class 1 Check shear - Section 6.2.6 $Design shear resistance moment M_{max} = 43 \text{ kN}$ $Des.bending resist.moment M_{max} = 42.2 \text{ pASS - Design bending resist.moment M_{max} = 62.2 \text{ pASS - Design bending resist.moment M_{max} = 62.2 \text{ pASS - Design bending resist.moment M_{max} = 62.2 \text{ pASS - Design bending resist.moment M_{max} = 62.2 \text{ pASS - Design bending resist.moment M_{max} = 62.2 \text{ pasin bending resist.moment M_{max} = 62.2 pasin bend$	n N					Analysis results
Maximum shear Vmax = -22.6 kN Vmm = -23.4 kN Deflection $\delta_{max} = 4.9 \text{ mm}$ $\delta_{min} = 0 \text{ mm}$ Maximum reaction at support A R.4.max = 0 kN R.4.min = 0 kN Maximum reaction at support B R.9.max = 23.4 kN R.9.min = 23.4 kN Unfactored permanent load reaction at support B R.9.max = 23.4 kN R.9.min = 23.4 kN Unfactored variable load reaction at support B R.9.Minate = 3.3 kN Section details Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance V.G.ed = 439.5 Design shear force VExt = 23 kN Design shear resistance exceeds design PASS - Design shear resistance exceeds design Check bending moment - Section 6.2.5 Design bending resist.moment Mc.ed = 62.2 PASS - Design bending resist.moment Mc.ed = 62.2 PASS - Design bending resist.moment Mc.ed = 62.2 Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads Steel grade Steel grade	N	M _{min} = -46 kNm	Im	$M_{max} = 0 kN$		Aaximum moment
Deflection Smax = 4.9 mm Smin = 0 mm Maximum reaction at support A RA_max = 0 kN RA_min = 0 kN Maximum reaction at support B RB_max = 23.4 kN RB_min = 23.4 kN Unfactored permanent load reaction at support B RB_max = 23.4 kN RB_min = 23.4 kN Unfactored variable load reaction at support B RB_max = 23.4 kN RB_min = 23.4 kN Section details Section details Section details Section type SHS 120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: SHS 120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance VGRet = 439.9 PASS - Design shear resistance VGRet = 439.9 Check bending moment MEd = 46 kNm Des.bending resist.moment Med = 62.2 PASS - Design bending resist.moment Med = 62.2 PASS - Design bending resist.moment Med = 62.2 PASS - Design bending resist.moment Med = 62.2 PASS - Design bending resist.moment Med = 62.2		Vmin = -23.4 kN	6 kN	Vmax = -22 .0		Aaximum shear
Maximum reaction at support A RA_max = 0 kN RA_min = 0 kN Maximum reaction at support B RB_max = 23.4 kN RB_min = 23.4 kN Unfactored permanent load reaction at support B RB_Permanent = 13.7 kN RB_min = 23.4 kN Section details Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Image: Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance V_GRd = 439.5 Design shear resistance V_GRd = 439.5 Check bending moment MEd = 46 kNm Des.bending resist.moment Med = 62.2 PASS - Design bending resist.moment Med = 46.2 PASS - Design bending resist.moment Med = 62.2 Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads Section for the to variable loads		$\delta_{min} = 0 mm$	nm	δ _{max} = 4.9 r		Deflection
Maximum reaction at support B RB_max = 23.4 kN RB_min = 23.4 kN Unfactored permanent load reaction at support B RB_Permanent = 13.7 kN Unfactored variable load reaction at support B RB_Vermable = 3.3 kN Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Steel grade Steel grade Section classification Class 1		$R_{A_{min}} = 0 \ kN$	٨N	$R_{A_{max}} = 0$	ort A	Aaximum reaction at suppo
Unfactored permanent load reaction at support B Re_Permanent = 13.7 kN Unfactored variable load reaction at support B Re_Variable = 3.3 kN Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Steel grade	Ň	R _{B_min} = 23.4 kN	.4 kN	RB_max = 23	ort B	Aaximum reaction at suppo
Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade Section type Section classification Class 1 Check shear - Section 6.2.6 Design shear force VEd = 23 kN Design shear resistance VCRd = 439.5 PASS - Design shear resistance exceeds design Check bending moment MEd = 46 kNm Des.bending resist.moment McRd = 62.2 PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads			= 13.7 kN	B RB_Permanent	reaction at support B	Infactored permanent load
Section details Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade			3.3 kN	$R_{B_Variable} =$	action at support B	Infactored variable load rea
Section type SHS 120x120x10.0 (Tata Steel Celsius (Gr355 Gr420)) Steel grade $\int_{2}^{27} \int_{2}^{27} \int_{10}^{10} $						Section details
Section classification Class 1 Check shear - Section 6.2.5 Design shear force $V_{Ed} = 23 \text{ kN}$ Design shear resistance $V_{e,Rd} = 439.9$ PASS - Design shear resistance exceeds design Check bending moment - Section 6.2.5 Design bending moment $M_{Ed} = 46 \text{ kNm}$ Des.bending resist.moment $M_{e,Rd} = 62.2$ PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads	grade S355F	Gr420)) Steel grade	Celsius (Gr355 0	10.0 (Tata Stee	SHS 120x120x10	Section type
Section classification Class 1 Check shear - Section 6.2.6 Design shear resistance Vc,Rd = 439.9 Design shear force VEd = 23 kN Design shear resistance Vc,Rd = 439.9 PASS - Design shear resistance Vc,Rd = 439.9 PASS - Design shear resistance exceeds design Check bending moment - Section 6.2.5 Design bending resist.moment Mc,Rd = 62.2 PASS - Design bending resist.moment Mc,Rd = 62.2 PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads Vertical deflection due to variable loads				120		
Check shear - Section 6.2.6 VEd = 23 kN Design shear resistance Vc,Rd = 439.9 PASS - Design shear resistance Vc,Rd = 439.9 PASS - Design shear resistance Vc,Rd = 439.9 Check bending moment - Section 6.2.5 PASS - Design shear resistance vc,Rd = 62.2 PASS - Design bending resist.moment Mc,Rd = 62.2 PASS - Design bending resistance moment exceeds design bending resistance Check vertical deflection - Section 7.2.1 Kert exceeds design bending resistance Check vertical deflection due to variable loads Kert exceeds design bending resistance					Class 1	Section classification
Design shear force VEd = 23 kN Design shear resistance Vc,Rd = 439.9 PASS - Design shear resistance PASS - Design shear resistance Vc,Rd = 439.9 Check bending moment - Section 6.2.5 Design bending resist.moment Mc,Rd = 62.2 Design bending moment MEd = 46 kNm Des.bending resist.moment Mc,Rd = 62.2 PASS - Design bending resistance moment exceeds design bending resistance mome					.6	Check shear - Section 6.2
Check bending moment - Section 6.2.5 Design bending moment MEd = 46 kNm PASS - Design bending resistance moment exceeds design b	= 439.9 kN s design shear fo	stance V _{c,Rd} = 439 . resistance exceeds des	Design shear resi S - Design shear	PAS	V _{Ed} = 23 kN	Design shear force
Design bending moment MEd = 46 kNm Des.bending resist.moment Mc,Rd = 62.2 PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads					Section 6.2.5	Check bending moment -
PASS - Design bending resistance moment exceeds design ber Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads	= 62.2 kNm	st.moment Mc,Rd = 62.2	Des.bending resis		M _{Ed} = 46 kNm	Design bending moment
Check vertical deflection - Section 7.2.1 Consider deflection due to variable loads	gn bending mom	ment exceeds design be	ng resistance mo	- Design bendir	PASS -	
					- Section 7.2.1	Check vertical deflection
Limiting deflection $\delta_{iim} = 5.6 \text{ mm}$ Maximum deflection $\delta = 4.888 \text{ mm}$	388 mm	on δ = 4.888 m	Maximum deflecti		δlim = 5.6 mm	imiting deflection
PASS - Maximum deflection does not exceed of	ceed deflection I	lection does not exceed	S - Maximum defl	PAS		

Project 32 The Green	Job No. 224260 Sheet/ 10
mason navarro pledge Rich mond	Design Element Roof Structure
Calc. Title Support to hip	Calc. by Date 0/9/24 Chk'd by JL
Desian of Support to 1 building	rip member in Conner of
Monneuts @ fixed	end.
Dead = 13.66 KN x	2.0m = 26.11KNM
Line - 3.28KN x	2.0m = 6.6KNM
	SUM= 32.7KNm
Factored = 1.35x 26.1kN+1.5	× 6.6KN = 45.1 KNm

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Sheet / Rev. Job No. Project The Green 27 18 224260 Richmond Design Element Chrevet mason navarro pledge Roc Calc. by Chk'd by Calc. Title Date Support 110 3 hip 0 JL calculate nods and channel in masonry Applied force down rods from SHS = 2.89KN 2No. 16 mm diancter use inspection are ok bes channel section use 100x 50 PEC Check Uphath = soonin 500mm × 50mm = 25000 mm Avea = (2.89 KW × 1000) /25000 mm Stuces = 0.12N By inspection of Say 4No. Miz resin fixings @ 135 mon de Use 2No.M16 rods 50×100 20 4No. M12 Lowin wide 1EV o Marine 6 135 135 135 PFC Set into Elevation on channe Masonry action through hanne

	mnp	^{Project} 32 The Green,	J	ob No. 2242	260	Sheet / Rev. 1	9		
ma	son navarro pledge	Richmond	Ē	Design Element RC	of Structur	е			
Calc. Title	Over Deflecting Pu	rlin	C	CM	Date 09/0)9/24	Chk'd J	by L	
	Design of strength	ening works to over defl	lecting	g purlin					
		Loading							
		Roof							
		Dead							
		Existing Loads			Units				
		Tiles		0.70	kN/m2				
		Softwood Batterns		0.02	kN/m2				
		Rafters		0.08	kN/m2			_	
			SUN	0.80	kN/m2				
		Additional Historic Loads							
		Lath + Plaster		0.31	kN/m2				
			SUN	0.31	kN/m2				
		Additional Future Loads							
		Kingspan Membrane		0.00	kN/m2				
		2x50mm Insulation		0.03	kN/m2				
		Allow for Max of 15mm							
		Fire Board or Lath +							
		Plaster		0.31	kN/m2				
			SUN	1 0.34	kN/m2				
		Live							
		Snow		0.30	kN/m2				
			SUN	0.30	kN/m2				
		Ceiling							
								_	
		Dead							
		Existing Loads						_	
		Ceiling Joists		0.06	kN/m2				
			SUN	0.06	kN/m2				
		Additional Historic Loads							
		Lath + Plaster		0.31	kN/m2				
			SUN	0.31	kN/m2				
		Additional Future Loads							
		200mm Roll of Insulation		0.02	kN/m2				
		Allow for Max of 15mm Fire Board or Lath +							
		Plaster		0.31	kN/m2				
			SUN	0.33	kN/m2				
		Live							
		Access		0.25	kN/m2				
			SUN	0.25	kN/m2				

ma	ison na	M avarra			D	Proj	ject	32 Rie	Tr chr	ne no	Gro nd	ee	n,			J D E	ob No esigr leme	^{2.} 2	224 Ro	26 201	0 • St	tru	ctu	re	Sh Re	eet / v.	20)				
Calc. Title	Wors	st Ca	se	Pur	lin											С	alc. t CN	ру /			D	ate (09/	09	/24	Ļ			Chk'	^{d by}		
								_	_	_		_	_				_						-		-						_	_
																										<u> </u>						
		W	ors	t Ca	se P	Purli	in										_									_						
		D		C													An	noi 4 a	unt		Ur	nits				_						
			iriin Irlin	- Spa	an den:	20												4.2 21	0		m					_						
			adi	ng w	vidtl	an h Ur	nne	er (C)n S	lon	e)							2.1	9		m					_						
			adi	ng w	vidtl	h Lo	ppe	er (O)n S	lop	e)							2.5 1.0	0		m					_						
		Lc	adi	ng w	/idtl	h Ur	ppe	er (C)n P	lan))							1.8	3		m					_					_	
		Lc	adi	ng w	vidt	h Lo	we	er (O	n P	lan	,)						(0.7	0		m					_					_	
		Lc	adi	ng w	/idtl	h go	oing	g on	to P	url	, lin (On	Slo	ope)		:	3.5	9		m					_					_	
		Lc	adi	ng w	/idt	h go	oing	g on	to P	url	lin (On	Pla	an)				2.5	3		m					_					_	
		D	ensi	ty o	f Pu	rlin,	, So	ftw	000	ł								4.2	0		kN	/m	3			_					-	
		Ρι	ırlin	De	pth													120	D		mı	m										
		Ρι	ırlin	Wie	dth													190	D		mı	m				_					-	
		Se	lf w	eigh	nt												(0.1	0		kN	/m				_						
				_	_					_																_					-	
		Ca	pac	city (of E	xisti	ing	Pur	lin f	fro	m T	ed	ds				_									_					-	
																	Ca	pa	city		01	iits				_						
		Co	omp	ress	sive	Stre	ess										1	.53	88		n/	mn	n2									
		Be	endi	ng S	tres	SS											1(0.7	07		n/	mn	n2									
		Sh	ear	Stre	ess												2	.46	52		n/	mn	n2			_						
		D	efleo	ctior	n												14	4.0	00		mı	m										
																										_						
		Ca	ipad	city (of E	xisti	ing	Pur	lin f	fro	m T	ed	ds													_					_	_
		D	efleo	ctior	n Lir	nite	d											1.0	0		kN	l/m									_	
		St	ren	gth l	limi	ted												3.0	0		kN	/m				_					-	
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Project 32 The Green Richmond					Job No	°. 2	2426	60		Sheet / Rev. 21				
ason navarro pledge	Richmor	nd			Desigr Eleme	n ent	Root	f Struc	ture					
Worst Case Purlin				Calc. by			Date 09			/24		Chk'd by	-	
Loading onto Purlin														
Dead Load			Exis	sting	Loads	Hi	istoric	al Loads	Fut	ure Loads	Uni	its		
Roof - UDL				2.8	7		3.	98		4.10	kN/	/m		
Ceiling				0.1	2		0.	68		0.72	kN/	/m		
Boof UD				0.7	6		0	76		0.76	LN	/m	-	
Ceiling				0.7	5 5		0.	70 46		0.76	kin/	/m /m		
Centing				0.4	0		0.	40		0.40	KIN/		-	
Total Line Loads													-	
Dead Load				2.9	9		4.	67		4.81	kN/	/m		
Live Load	Live Load					1.22				1.22	kN/	/m		
Unfactored	Unfactored				1	5.89				6.03	kN/	/m	_	
Factored				5.8	6		8.	13		8.32	kN/	/m		
Design of Strengthen	ing to Purlin												-	
Split loads into result	ant forces, the thir	nner d	depth	will a	attach	mo	ove loa	ad.						
Downward Slope Mo	vement												_	
UDL down slope (Dea	ad)			3.40	D	kΝ	l/m						_	
UDL down slope (Live	e)			0.8	6	k٨	l/m						_	
UDL down slope (Tot	al)			4.2	6	kN	l/m						_	
Inward Movement														
UDL Perpendicular to	Slope (Dead)			3.4	0	kΝ	l/m							
UDL Perpendicular to	Slope (Live)			0.8	6	kΝ	l/m							
UDL down slope (Tot	al)			4.2	6	kΝ	l/m							
The sector sector sector sector														
to support the result	w steer channel see	LUONS												
directions and fix ney	w steel channel to													
existing purlin.													-	
From Tedds calculati	ons, new steel cha	nnel												
sections to be: 1No. 1	150x90x24 PFC. Th	е												
steel channel will hav	e to be split into												-	
sections and then we	elded together to s	uit :											-	
the existing deflected	i snape of the purl	ın.												
											_			

mnp	Project 32 The Green, Richmond	Job No. 2242	60	Sheet / Rev.	22 _{/ P01}
meson navarno pledge	,	Design Element Roof \$			
Calc. Title Worst Case Purlin	- Strengthening y-y	Calc. By CM	Date 20/09	9/2024	Chk'd By

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14

	Project		Job No.		Sheet /	22
mnp	32 The Green, Ri	ichmond	224	4260	Rev.	23 / P01
			Design Element Roc	of Structure		
worst Case Purlin	- Strengthening v-v		Calc. By	Date		Chk'd By
			CM	20/09	/2024	JL
Analysis results Maximum moment Maximum shear Deflection Maximum reaction at suppo Unfactored permanent load Unfactored variable load rea Maximum reaction at suppo Unfactored permanent load Unfactored variable load rea Section details Section type	ort A reaction at support A action at support A ort B reaction at support B action at support B PFC 150x90x24 (B	Mmax = 13.7 Vmax = 13 k δmax = 1.4 m RA_max = 13 RA_Permanent RA_Variable = RB_max = 13 RB_Permanent RB_Variable =	CM KNm kN = 7.6 kN 1.8 kN kN = 7.6 kN 1.8 kN Section Rang	 Mmin = Vmin = δmin = RA_min RB_min	0 kNm -13 kN 0 mm = 13 kN = 13 kN	9 S355
		90				
Section classification	Class 1					
Check shear - Section 6.2	.6					
Design shear force	V _{Ed} = 13 kN	PAS	Design shear S - Design sh	resistance ear resistance (V _{c,Rd} = 226 exceeds des	kN ign shear force
Check bending moment -	Section 6.2.5					
Design bending moment	MEd = 13.7 kNm		Des.bending r	esist.moment	Mc,Rd = 63.4	l kNm
Slenderness ratio for later	ral torsional buckling					
LTB slenderness ratio	$\overline{\lambda}_{LT} = 1.072$		Limiting slende	erness ratio	$\overline{\lambda}$ LT,0 = 0.4	00
		Ā	, μτ > λ ιτ,ο - La	iteral torsional	buckling car	not be ignored
Design resistance for buc Des.buckling resist.moment	kling - Section 6.3.2.1 Mb,Rd = 33.8 kNm PASS - De	sign bucklir	ng resistance	moment excee	ds design be	ending moment
Check vertical deflection	- Section 7.2.1					
Limiting deflection	λim – 11 7 mm		Maximum defl	ection	δ – 1 428 m	m
		PAS	S - Maximum	deflection does	0 = 1.420 II not exceed	deflection limit

	mesor navaro piede	Project 32 The Green, Richmond	Job No. 2242	60	Sheet / Rev.	24 _{/ P01}
	meson navarro piedge		Design Element Roof \$	Structure		
Calc. Title	Worst Case Purlin	- Strengthening z-z	Calc. By CM	Date 20/09)/2024	Chk'd By JL

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Load Envelope - Combination 1 6.196 0.0 4200 mm L 1 Bending Moment Envelope kNm 0.0 13.662 -13.7 4200 mm Shear Force Envelope kN 13.0 13.012 -0.0 -13.012 -13.0 mm 4200

Support conditions Support A Vertically restrained Rotationally free Support B Vertically restrained Rotationally free **Applied loading** Beam loads Permanent self weight of beam $\times 1$ Permanent full UDL 3.4 kN/m Variable full UDL 0.86 kN/m Load combinations Support A Permanent × 1.35 Load combination 1 Variable \times 1.50 $Permanent \times 1.35$ Variable \times 1.50 Permanent × 1.35 Support B Variable \times 1.50

TEDDS calculation version 3.0.14

	Project) i o b m o m o l	Job No.	24260	S R	heet / ev.	25 _{/ P01}
mason nevarro piedge	32 The Green, R	achmona	Design	24200			1.01
			Element R		;		
Worst Case Purli	n - Strengthening z-z		Calc. By CM	Date 20	0/09/2	024	Chk'd By
Analysis results							
Maximum moment		Mmax = 13.7	' kNm	М	min = 0	kNm	
Maximum shear		V _{max} = 13 k	N	Vr	min = -1	3 kN	
Deflection		δ _{max} = 6.6 n	nm	δn	nin = 0 r	nm	
Maximum reaction at sup	port A	RA_max = 13	kN	R	A_min = '	13 kN	
Unfactored permanent lo	ad reaction at support A	RA_Permanent	= 7.6 kN				
Unfactored variable load	reaction at support A	RA_Variable =	1.8 kN	_			
Maximum reaction at sup	port B	RB_max = 13	kN Z C LN	R	B_min = '	13 kN	
Unfactored permanent lo	ad reaction at support B	RB_Permanent	= 7.6 KN				
Unlactored variable load	reaction at support B	RB_Variable =	1.8 KIN				
Section details						.	0055
Section classification	→ 12 ← ↓ Class 1	ي ن ن ن ن ن ن ن ن ن ن ن ن ن ن ن ن ن ن ن		→ 12 ←	-		
Conton blabbilloation	326						
Check shear - Section 6	.2.0						
Check shear - Section 6 Design shear force	V _{Ed} = 13 kN	PAS	Design shea S - Design s	r resistance hear resistar	nce exe	√ _{c,Rd} = 488. ceeds desi	2 kN ign shear force
Check shear - Section 6 Design shear force Check bending momen	V _{Ed} = 13 kN t - Section 6.2.5	PAS	Design shea S - Design s	r resistance hear resistar	nce exe	V _{c,Rd} = 488. ceeds desi	2 kN ign shear force
Check shear - Section 6 Design shear force Check bending moment Design bending moment	V _{Ed} = 13 kN t - Section 6.2.5 M _{Ed} = 13.7 kNm PASS - D	PAS esign bendir	Design shea S - Design s Des.bending ng resistanc	r resistance hear resistar resist.mome e moment ex	nce exi nt I ceeds	V _{c,Rd} = 488. ceeds desi M _{c,Rd} = 26.9 design be	2 kN ign shear force kNm ending moment
Check shear - Section 6 Design shear force Check bending moment Design bending moment Check vertical deflection Consider deflection due t	V _{Ed} = 13 kN t - Section 6.2.5 M _{Ed} = 13.7 kNm PASS - D on - Section 7.2.1 o variable loads	PAS esign bendir	Design shea S - Design s Des.bending ng resistanc	r resistance hear resistar resist.mome e moment ex	nce exe nt l	V _{c,Rd}	2 kN ign shear force) kNm ending moment
Check shear - Section & Design shear force Check bending moment Design bending moment Check vertical deflection Consider deflection due t Limiting deflection	V_{Ed} = 13 kN t - Section 6.2.5 M_{Ed} = 13.7 kNm PASS - D on - Section 7.2.1 o variable loads δ_{lim} = 11.7 mm	PAS esign bendir	Design shea S - Design s Des.bending ng resistanc Maximum de	r resistance hear resistar resist.mome e moment ex	nce exi nt I ceeds	$V_{c,Rd} = 488.$ ceeds desi $M_{c,Rd} = 26.9$ design be $\delta = 6.554 \text{ m}$	2 kN ign shear force) kNm inding moment

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mnd	Project 32 The Green,	Job No. 224260		Sheet / 29 Rev. 29	
mason navarro pledge	Richmond, London	Design Element Floor \$	Structure		
Calc. Title Loading		Calc. by CM	Date 18/1	1/24	Chk'd by JL

calculatio	ла				anu streng	juii.
		Joists				
Room		Depth (mm)	Wide (mm)	Centres (mm)	Span (m)	Timber Grade
4	F	175	75	400	3.34	C24
1	F	170	65	390	2.72	C24
2	F	175	70	385	2.5	C24
3	F	175	75	450	1.91	C24
1	G	150	80	410	2.6	C24
2	G	175	80	450	2.8	C24
3	G	165	80	370	2	C24
3	G	175	80	400	2.94	C24
4	G	No data				C24
		loists - Max	oading			
		Deflection lin	nited to 14m	m		
		or 0.004xSp	an	Strength limite	ed	
Room		Capacity	Capacity	Capacity	Capacity	
		(kN/m)	(kN/m^2)	(kN/m)	(kN/m^2)	
4	F	1.7	4.25	3.8	9.50	
1	F	2.5	6.41	4.7	12.05	
2	F	3.8	9.87	6.4	16.62	
3	F	8.9	19.78	11.5	25.56	
1	G	2.5	6.10	5	12.20	
2	G	3.1	6.89	5.8	12.89	
3	G	7.1	19.19	10.2	27.57	
3	G	2.7	6.75	5.3	13.25	
4	G	No data				
	_					
	_					

Project 32 The Green,	Job No. 224260	Sheet / Rev.	30
mason navarro pledge Richmond, London	Design Element Floor	Structure	
Calc. Title Loading	Calc. by	Date 18/11/24	Chk'd by JL

Loading	Dead			
	Existing Loa	ds	(kN/m^2)	
	18mm Plywo	bod		
	(Exclude jois	sts)	0.01	
			2	
	Additional H	listoric Loads	(kN/m²)	
	Lath + Plaste	er	0.31	
	Additional F	uture	(kN/m^2)	
	Services		0.05	
	Insulation		0.05	
	Lath + Plaste	er	0.31	
		Sum	0.41	
			- 2	
	Live		(kN/m²)	
	Domestic Lo	ading	1.5	
				_
				_

mason navarro pledge	^{Project} 32 The Green, Richmond, London	Job No. 224260 Design Element Floor	0 r Structure	Sheet / 31 Rev.	
Calc. Title Loading		Calc. by	Date 18/1	1/24	Chk'd by

		Joists - Max	Loading			
_		Deflection li	mited to 14m	m		
		or 0.004xSp	an			
		Capacity	Capacity	Capacity	Capacity	
		Existing	Historically	Future Dead	Future D+L	Check
Room		(kN/m²)	(kN/m²)	(kN/m²)	(kN/m ²)	
4	F	4.24	3.93	3.83	2.33	PASS
1	F	6.40	6.09	5.99	4.49	PASS
2	F	9.86	9.55	9.45	7.95	PASS
3	F	19.77	19.46	19.36	17.86	PASS
1	G	6.09	5.78	5.68	4.18	PASS
2	G	6.88	6.57	6.47	4.97	PASS
3	G	19.18	18.87	18.77	17.27	PASS
3	G	6.74	6.43	6.33	4.83	PASS
4	G	No data				
_		Strength lim	ited			
		Capacity	Capacity	Capacity	Capacity	
_		Existing	Historically	Future	Future D+L	Check
Room		(kN/m ²)	(kN/m ²)	(kN/m²)	(kN/m ²)	
4	F	9.49	9.18	9.08	7.58	PASS
1	F	12.04	11.73	11.63	10.13	PASS
2	F	16.61	16.30	16.20	14.70	PASS
3	F	25.55	25.24	25.14	23.64	PASS
1	G	12.19	11.88	11.78	10.28	PASS
2	G	12.88	12.57	12.47	10.97	PASS
3	G	27.56	27.25	27.15	25.65	PASS
Room Room 4 1 2 3 4 1 2 3 4 1 2 3 4 1 2 3 3 4 1 2 3 3 4 1 2 3 3 4 1 2 3 3 4 1 2 3 3 4 1 2 3 3 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 4 1 2 3 3 4 5 5 1 5 5 5 5 5 5 5	G	13.24	12.93	12.83	11.33	PASS
	G	No data				
<u>Sum</u>	ma	ary				
All jo	oist	s pass in def	lection and b	ending.		

mason navarro pledge	Project 32 The Green,	Job No. 224260		Sheet / 32 Rev. 32		
	Richmond, London	Design Element Floor \$	Structure			
Calc. Title Loading		Calc. by CM	Date 18/1	1/24	Chk'd by JL	

	Bea Car calc	ry c cula	<u>s</u> but design cho ition allowable	eck on existi e line load fo	ing beams. Limit on both deflection	deflection to and strengt	14mm and h.
-			Beam Size				
+-	Room		Depth (mm)	Wide (mm)	Loading Width (m)	Span (m)	Timber Grade
	4	F	-	-	-	-	C24
	1	F	180	290	2.205	5	C24
	2	F	180	305	2.46	5.18	C24
	3	F	185	290	1.765	4.57	C24
	1	G	200	310	2.1	4.7	C24
	2	G	250	275	2.4	4.7	C24
	3	G	210	370	1.6	4.33	C24
_	3	G	-	-	-	-	C24
	4	G	220	290	2.3	4.35	C24
_			Note: Beam in	4G has splice	e joint in vault that is	s supported o	off brick vault
-							
+							
			Beams - Max	Loading			
			Deflection lim	ited to 14mm	Strength limited		
	Room		Capacity	Capacity	Capacity	Capacity	
			(kN/m)	(kN/m ²)	(kN/m)	(kN/m ²)	
	4	F	-	-	-	-	
	1	F	1.4	0.63	6.2	2.81	
	2	F	1.3	0.53	6.1	2.48	
	3	F	2.3	1.30	8	4.53	
_	1	G	2.8	1.33	9.4	4.48	
-	2	G	4.9	2.04	13.1	5.46	
-	3	G	5.5	3.44	14.7	9.19	
-	3	G	-	-	-	-	
-	4	G	4.8	2.09	12.5	5.43	
+							
+							
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-							
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masor		nn ro ple	D	Project	32 T Rich	The nmo	Gree nd, L	n, ondon	Jo De El	b No. 2 esign ement	2426 Floc	60 or Sti	ructu	ıre	Sheet / Rev.	33			
alc. ïtle L									Ca	alc. by	M	Da	^{ite} 1	8/1 <i>*</i>	1/24		Chk'	JL	
																			Γ
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		Load	ling	_												-			
				0	ead	_								2.		_			_
				Ex	istir	ng L	oads	-				(kN/r	m²)		_			+
				18	ßmm	l Ply	woo	d					0.0	1		_			
				Jo	Dead ixisting Loads ixisting Loads 8mm Plywood oists 8mm Plywood oists Sun Additional Historio ath + Plaster Additional Future ervices nsulation ath + Plaster Sun ive Domestic Loading								0.2	9					_
					ad isting Loads mm Plywood ists Iditional Histor Iditional Futur rvices sulation th + Plaster Sulation th + Plaster Sulation th + Plaster Sulation th + Plaster			Sum			0.3					_			-
		-												2		_			
				A	ead xisting Loads 8mm Plywood oists Additional Histor Additional Futu ervices nsulation ath + Plaster Additional Futu ervices nsulation ath + Plaster			toric l	oad	ls		(kN/r	m²)					+
				La	th +	Pla	ster						0.3	1					
				_										2,					-
		-		Ac	dditi	ona	al Fut	ure				()	kN/r	m⁻) -		_			F
				Se	ervice	es							0.0	5		_			
				In	sula	tior	1						0.0	5					-
				La	th +	Pla	ster	_					0.3	1					
				_				Sum					0.4	1		_			_
														2.					-
		-		Liv	ve							(kN/r	m~)		_			
				Do	Existing Loads 18mm Plywood Joists Additional Histor Lath + Plaster Services Insulation Lath + Plaster Services Insulation Lath + Plaster Services Insulation Lath + Plaster Substration Lath + Plaster Substration Lath + Plaster Substration Lath + Object Substration Live Domestic Loading Substration Substratin Substration			ding					1.5	5					-
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mnd	^{Project} 32 The Green, Richmond, London	Job No. 224260		Sheet / Rev. 34	
mason navarro pledge		Design Element Floor \$	Structure		
^{Calc.} Title Loading		Calc. by CM	Date 18/1	1/24	Chk'd by JL

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Room Existing Historically Future Dead Future D+L (kN/m^2) 4 F - <	Check - FAIL FAIL FAIL FAIL PASS - FAIL Check
Room (kN/m^2) (kN/m^2) (kN/m^2) (kN/m^2) 4 F - - - - 1 F 0.33 0.02 -0.08 -1.58 2 F 0.23 -0.08 -0.18 -1.68 3 F 1.00 0.69 0.59 -0.91 1 G 1.03 0.72 0.62 -0.88 2 G 1.74 1.43 1.33 -0.17 3 G 3.14 2.83 2.73 1.23 3 G - - - - 4 G 1.79 1.48 1.38 -0.12 - - - - - - 4 G 1.79 1.48 1.38 -0.12 - - - - - - 4 G 1.79 1.48 1.38 -0.12 - Ca	- FAIL FAIL FAIL FAIL PASS - FAIL
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3 F 1.00 0.69 0.59 -0.91 $1 G$ 1.03 0.72 0.62 -0.88 $2 G$ 1.74 1.43 1.33 -0.17 $3 G$ 3.14 2.83 2.73 1.23 $3 G$ - - - - $4 G$ 1.79 1.48 1.38 -0.12 $4 G$ 1.79 1.48 1.38 -0.12 $4 G$ 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 Strength limited Capacity Capacity Capacity 6 Existing Historically Future Dead Future D+L 7 7 7 7 7 7 6 7 7 7 7 7 7 7	FAIL FAIL PASS - FAIL Check
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3 G - - - - $4 G$ 1.79 1.48 1.38 -0.12 $4 G$ 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6 1.79 1.48 1.38 -0.12 6	- FAIL Check
4 G 1.79 1.48 1.38 -0.12 A A A A A A A A A A A A A A A A A A A A A A A A A B A A A A A A A A Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity A <td>FAIL Check</td>	FAIL Check
Image: strength limited Image: strength limited Image: strength limited Image: strength limited Capacity Capacity Capacity Image: strength limited Capacity Capacity Capacity Image: strength limited Capacity Capacity Capacity Image: strength limited Existing Historically Future Dead Room (kN/m²) (kN/m²) (kN/m²) (kN/m²) Image: strength limited Image: strength limited Image: strength limited Image: strength limited Room (kN/m²) (kN/m²) (kN/m²) (kN/m²) (kN/m²) Image: strength limited Room (kN/m²) (kN/m²) (kN/m²) (kN/m²) (kN/m²) Image: strength limited Image: strength limited Image: strength limited Image: strength limited Image: strength limited Image: strength limited Image: strength limited Image: strengt limited Image: strengt limi	Check
Strength limited Strength limited Capacity Capacity Capacity Capacity Capacity Capacity Existing Historically Future Dead Future D+L Room (kN/m²) (kN/m²) (kN/m²) 4 F - - 1 F 2.51 2.20 2.10 0.60	Check
Capacity Capacity Capacity Capacity Capacity Capacity Existing Historically Future Dead Future D+L Capacity Room (kN/m²) (kN/m²) (kN/m²) (kN/m²) (kN/m²) 4 F - - - - 1 F 2.51 2.20 2.10 0.60	Check
ExistingHistoricallyFuture DeadFuture D+LRoom (kN/m^2) (kN/m^2) (kN/m^2) (kN/m^2) 4F1F2.512.202.100.60	Check
Room (kN/m^2) (kN/m^2) (kN/m^2) (kN/m^2) 4 F - - - 1 F 2.51 2.20 2.10 0.60	
4 F	
1 F 2 51 2 20 2 10 0 60	-
	PASS
2 F 2.18 1.87 1.77 0.27	PASS
3 F 4.23 3.92 3.82 2.32	PASS
1 G 4.18 3.87 3.77 2.27	PASS
2 G 5.16 4.85 4.75 3.25	PASS
3 G 8.89 8.58 8.48 6.98	PASS
	-
4 G 5.13 4.82 4.72 3.22	PASS
Summory	
All beams pass in bending but many fail in deflection and will	
The peak as in bending but many fail in denection and with	
require strengthening. See typical flitch plate design over page	
require strengthening. See typical flitch plate design over page	ill age.

meson neverte piedge		Project 32 The Green, Richmond	Job No. 22420	60	Sheet / Rev.	35 _{/ P01}
			Design Element Design Check on Ground Floor Structure			or Structure
Calc. Title	Main Beam in Room 1G		Calc. By CM	Date 11/11/2024		Chk'd By

FLITCH BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating **National Amendment No.1**

mnp	Project 32 The Green Rich	Job No. 2242	Job No. 224260		36 _{/ P01}		
meson neverso piedgo		Design Element Desig	Design Element Design Check on Ground Floor Stru				
Calc. Title Main Beam in Room	m 1G	Calc. By CM	Date 11/11	/2024			
Reactions at support B $R_{B_max} = 12.310 \text{ kN}$ $R_{B_min} = 12.310 \text{ kN}$ Unfactored permanent load reaction at support B $R_{B_Permanent} = 12.310 \text{ kN}$ $\int_{0}^{\infty} \int_{0}^{\infty} \int_{$							
Timber section details Breadth of section Number of sections Timber strength class	b = 141 mm N = 2 C24	Depth of sectior	1	h = 200 mn	n		
Steel section details Breadth of steel plate Number of steel plates in be = 355 N/mm ² Bolt diameter	bs = 28 mm eam φ _b = 12 mm	Depth of steel p N₅ = 1	late	h₅ = 160 mi nominal yie	m Id stress f _y		
Member details Service class of timber Length of span Length of bearing	1 L₅1 = 4700 mm Lь = 100 mm	Load duration		Long-term			
Compression perpendicu Design compressive stress	lar to grain - cl.6.1.4 σ _{c.90.d} = 0.437 N/mm² PASS - Design comj	Design compres	ssive strength ds design cor	fc.90.d = 1.34 mpressive st	6 N/mm ² ress at bearing		
Bending - cl 6.1.6 Design timber bending stres N/mm ²	$\sigma_{m.t.d} = 3.905 \text{ N/mm}^2$	Design timber b	ending strengt	th f _{m.d}	= 12.923		
PASS - Design timber bending strength exceeds design timber bending stDesign steel bending stress $\sigma_{m.s.d} = 92.927 \text{ N/mm}^2$ Design steel bending strength fy.d = 355.000 N/mm^2PASS - Design steel bending strength exceeds des					bending stress 00 N/mm² bending stress		
Shear - cl.6.1.7							
Applied shear stress	τd = 0.248 N/mm ²	Permissible she PASS - Design she	ar stress ar strength e	f _{v.d} = 2.154 xceeds desig	N/mm² gn shear stress		
Deflection - cl.7.2							
Deflection limit	δ_{lim} = 14.000 mm	Total final defled PASS - Total final d	ction deflection is le	δ _{fin} = 13.93 ess than the	7 mm deflection limit		
Steel-to-timber connectio	ns - cl.8.2.3						
Characteristic yield momen	Characteristic yield moment - exp.8.30 Mv.R.k =			0.3 mm ^{0.4} × f _{u.k} × φ _b ^{2.6} = 76745 Nmm			
Char.embed.strength par.to	grain - exp.8.32 fh.		0.082e9 m/sec ² × (1 mm - 0.01 × φ _b) × ρ _k = 25.256 N/mm ² .35 + 0.015 × φ _b / 1 mm = 1.530				
Char.embed.strength perp.t	Char.embed.strength perp.to grain - exp.8.31			= fh.o.k / k90 = 16.507 N/mm ²			
Thickness limit for thin stee	Thickness limit for thin steel plates bs.thn =			$\phi_b / 2 = 6 \text{ mm}$			
Thickness limit for thick stee	el plates ba	$b_{s.thk} = \phi_b = 12 \text{ mm}$					

Project Job No. Sheet Rev. 37 /	P01						
S2 The Green, Richmond Design Element Design Check on Ground Floor Stru	Ground Floor Structure						
Calc. Title Main Beam in Room 1G Calc. By Date Chk'd CM 11/11/2024 J	^{By}						
Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11 $F_{v.Rk.f} = f_{h.k} \times b \times \phi_b = 27.930 \text{ kN}$							
$F_{v.Rk.g} = f_{h.k} \times b \times \phi_b \times (\sqrt{[2 + 4 \times M_{y.Rk} / (f_{h.k} \times \phi_b \times b^2)]} - 1) = 12.$	331 kN						
F _{v.Rk.h} = 2.3 × √[M _{y.Rk} × f _{h.k} × φ _b] = 8.968 kN							
F _{v.Rk} = Min(F _{v.Rk.f} , F _{v.Rk.g} , F _{v.Rk.h}) = 8.968 kN							
Flitch plate bolting requirements							
Total load on member $W_{tot} = 24.621 \text{ kN}$ Total load taken by steel $W_s = 18.896 \text{ kN}$							
Characteristic bolt capacity $F_{v.Rk} = 8.968 \text{ kN}$ Number of interfaces $N_{int} = 2$							
Bolts required at supports $N_{be} = 2$ Bolts required to beam length $N_{bl} = 1.054$	Nbl = 1.054						
Limiting bolt spacingSlimit = 500 mmMaximum bolt spacingSmax = 500 mm	S _{max} = 500 mm						
Provide a minimum of 2 No.12 mm diameter bolts at each support							
Provide 12 mm diameter bolts at maximum 500 mm centres staggered 50 mm alternately above and below the cer	ntre line						
Minimum bolt spacings - cl.8.5							
Minimum end spacingSend = 84 mmMinimum edge spacingSedge = 48 mm							
Minimum bolt spacing $S_{bolt} = 48 \text{ mm}$	4 0.0 mm						
Minimum washer diameter $\phi_w = 36 \text{ mm}$ Minimum washer thickness $t_w = 3.6 \text{ mm}$							

32 The Green Sheet / Rev. Job No. 224260 38 Design Element Richmond Structure mason navarro pledge FLOOF Chk'd by JL Calc. Title Calc. by Date Design of beam in G4 M 18/11 Design of Strengthening to beau in GG. The beam has a splice connection at midspan that cannot canny beading forces in the beau has been propped off the valition before. The beam is to be strengthered and the prop removed. Design 2No, steel channel sections to Strengthen the bear New steelionk is to · Span of beam = 435m - loading with = 2.3 m · Size of beam = 220mm (d) x 290mm (w) Loading - Full UDL . Dead = 071 KN/12× 23m = 1.63 KN/m -line = 1.5KN/m2 × 2:3m = 3:45 KN/m - S.W. beau= 4.2KN/13 × 0.22m× 0.29u = 0.27KN/m Results Use 2No. 150 x90 PFCs fixed See Tedds calculations over

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mation navarro pilidge	Project 32 The Green.	Job No. 2242	60	Sheet / Rev.	39 _{/ P01}
	Richmond, London	Design Element Floor Structures			
Salc. ittle Strengthening to Beam in G4		Calc. By CM	Date 18/11	/2024	Chk'd By

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

	Project		Job No.			Sheet /	40	
mnp	32 The Green,		224260 40			40 P01		
material control of proge	Richmond, Lond	on	Design Element F	loor St	r Structures			
tle Strengthening to B	eam in G4		Calc. By	C	Date	/2024	Chk'd By	
			CIVI		10/11	/2024	JL	
					Variab	le × 1.50		
Analysis results								
Maximum moment		Mmax = 19.8	8 kNm		Mmin =	0 kNm		
Maximum shear		Vmax = 18.2	kN		Vmin =	-18.2 kN		
Deflection		δ _{max} = 5.6 n	nm		$\delta \min$ =	0 mm		
Maximum reaction at suppo	ort A	RA_max = 18	. 2 kN		$R_{A_{min}}$	= 18.2 kN		
Unfactored permanent load	reaction at support A	RA_Permanent	= 5.2 kN					
Unfactored variable load re	action at support A	$R_{A_Variable} =$	7.5 kN					
Maximum reaction at suppo	ort B	RB_max = 18	.2 kN		$R_{B_{min}}$	= 18.2 kN		
Unfactored permanent load	reaction at support B	RB_Permanent	= 5.2 kN					
Unfactored variable load re	action at support B	RB_Variable =	7.5 kN					
Section details								
Section type	2 x PFC 150x90x2	24 (British St	eel Section	n Range	e 2022 (BS4	I-1)) Ste	el grade	
	S355							
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	4 90_	▶						
Section classification	Class 1							
Check shear - Section 6.2	6							
Design shear force			Docian cho	or rooid	tanaa	\/ <u> 45</u> 2	1 LN	
Design snear force			Design snea	ar resisi		Vc,Rd = 432 .	T KIN	
		PAS	S - Design :	snear r	esistance e	exceeas aes	ign snear force	
Check bending moment -	Section 6.2.5							
Design bending moment	MEd = 19.8 kNm		Des.bendin	ig resist.	.moment	Mc,Rd = 126	.8 kNm	
Slenderness ratio for late	ral torsional buckling							
LTB slenderness ratio	λιτ = 1.091		Limitina sle	ndernes	ss ratio	$\overline{\lambda}$ LT.0 = 0.4	00	
		ā	$\bar{h}_{T} > \bar{\lambda}_{T}$	Lateral	torsional	buckling car	not be ignored	
				Laterai	101 3101101		inot be ignored	
Design resistance for buc	ckling - Section 6.3.2.1							
Des.buckling resist.momen	t M _{b,Rd} = 66.2 kNm							
	PASS - De	esign bucklir	ng resistand	ce mor	nent excee	ds design be	ending moment	
Check vertical deflection	- Section 7.2.1							
Consider deflection due to	permanent and variable	loads						
Limiting deflection	δlim = 12.1 mm		Maximum d	deflection	n	δ = 5.559 m	nm	
-		PAS	S - Maximui	m defle	ction does	not exceed	deflection limit	
				2. 5.1.0				

Hitchin

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