Structural Report

Apartment 3.10 St Pancras Chambers, Euston Road, London, NW1 2AR

Document: 2304-R01 Date: 06 December 2023 Revision: 00

Client:

Nathan Roberts

Project Address:

Apartment 3.10 St Pancras Chambers Euston Road London NW1 2AR

Revisions:

Revision	Date	Description
00	06/12/23	First issue

Prepared by:

Lloyd Evans MEng CPEng MIE(Aust)

Notes:

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1.0 Introduction

Spark Structures was appointed by the Client to carry out a structural inspection and evaluation of the existing structure at apartment 3.10 St Pancras Chambers in relation to the proposed alterations to the existing bathroom partition wall and ceiling. The proposed works are shown on the architectural drawings by Pick Our Brains.

An inspection of the property was carried out on 23 November 2023 by Lloyd Evans (Spark Structures).

The purpose of this report is to assess the existing structural arrangement of the apartment and offer recommendations to accommodate the proposed alterations.

This report is based upon a non-intrusive visual inspection of the property. This report describes the findings and draws conclusions of a general nature and is not a detailed structural appraisal of the building. Whilst comments are made to satisfy the requirements of the brief, the report has, of necessity, not been exhaustive and cannot therefore constitute a warranty as to the soundness or otherwise of the property in areas hidden from visual inspection or not part of the brief.

This report has been prepared for the sole use of the Client.

2.0 Overview

The apartment is located within the former Midland Grand Hotel building, with the conversion completed in 2009. The Grade I listed building falls within the Kings Cross St Pancras conservation area.

The third floor apartment comprises three former hotel rooms. The main dividing walls between the former hotel rooms running north west to south east in the apartment appear to be solid masonry, and studwork partitions have been installed within two of the three main spaces to form a separate hallway and bathroom.

All of the rooms have approximately 3.98m floor to ceiling height apart from the bathroom which has a false ceiling installed to form a storage and service void above. This storage and service void above the ceiling can be accessed via a ceiling hatch in front of the bathroom door.

The proposed works are indicated on the architectural drawings by Pick Our Brains and can be summarised as follows:

• Removal of the existing upper portion of the partition wall between the bedroom and bathroom, and installation of a new paddle stair to improve access to a loft storage void located above the bathroom false ceiling.

This report should be read in conjunction with all architectural drawings and reports produced by Pick Our Brains.

3.0 Observations and Discussion

Following the inspection the following items were noted regarding the existing structure and the proposed alterations:

- 1. Both the existing main building structure and the infill partitions and ceilings appear to be in good condition with no noticeable signs of structural distress or deterioration.
- 2. All of the proposed alterations are to non-structural partitions and false ceilings, and will have no impact on the structural stability and integrity of the main building. Refer to figure 1 and 4 below.
- 3. The proposed alterations would improve access to the space above the bathroom however due to the limited clearance to the main building ceiling above, the classification of the space as a ceiling void would remain the same, as would the 0.25kN/m2 design imposed loading for such a space as stipulated by BS6399 and EN1991. Therefore the overall design imposed load on the main building would remain unchanged.
- 4. It is proposed to install a new paddle stair to improve access to the ceiling void. Due to its lightweight nature and the spatial constraints imposed on the main floor space below the paddle stair could be considered akin to a moveable item of furniture. As such, the loading of the stair would account for a minor portion of the residential design imposed floor load allowance of the apartment and would not contribute to an increase in the overall design loading on the existing floor structure.
- 5. Removal of the hot water cylinder, and replacing of the existing partition wall above the false ceiling level with an open balustrade structure would contribute to a net reduction in design dead load on the existing floor structure. Refer to figure 1 and figure 3 below. A detailed breakdown of the loading calculations is provided in appendix B.
- 6. The false ceiling is exposed on top and as such this detailing is not suitable for access as it is unsafe to directly stand on. Replacement of the lightweight steel joists and installation of new boarding would cause a nominal increase in the ceiling dead load, but is still part of a net reduction in loading when considered as part of the whole works. Refer to the detailed breakdown of the loading calculations in appendix B.
- 7. The partition wall appears to be constructed from lightweight steel studs that are continuous from floor to ceiling. Removal of the top part of the partition would require some minor strengthening/modifications to retain its stability for the altered layout.
- 8. It is unclear whether the existing partition wall studs would be suitable for fixing and supporting the proposed timber paddle stair and balustrade. Although designed for the ceiling design imposed load noted in point 3, the robustness of the existing connection between the ceiling and partition wall studs is unknown. Replacing the partition with a lightweight timber framed partition would be preferable to allow a more robust fixing between the ceiling and partition and to accommodate the install of the paddle stair and balustrade.
- 9. Although the proposed works are not likely to exceed the current design loading condition of the main building structure, it should be noted that the building was previously operational as a hotel which has a much higher allowance for design imposed floor load compared with the typical domestic/residential use category. As a result it is likely that the main building structure could accommodate notable increases in design loading if required.

4.0 Conclusion and Recommendations

- 1. No modifications or strengthening works are required to the main building structure.
- The lightweight steel ceiling and partition wall should be replaced with a lightweight timber stud partition and ceiling to allow top boards to be added, improved access, and fixing of the paddle stair/balustrade.

Please refer to the drawings in Appendix B for details of the proposed structural specification and details.



Fig. 1: Photograph of the existing ceiling void viewed from the access hatch facing west with the exposed top surface of the ceiling visible, the top part of the partition wall on the right and the main building masonry party wall on the left.



Fig. 2: Photograph of the existing ceiling void viewed from the access hatch facing south with the services visible, the main building masonry corridor wall on the left and the main building masonry party wall on the right.



Fig. 3: Photograph of the existing hot water cylinder and support bracket within the ceiling void supported on the main building masonry corridor wall.



Fig. 4: Photographs of the existing partition wall viewed from the bedroom looking towards the bathroom/hallway. The proposed works include the removal of the upper portion of the partition.

Appendix A - Loading Calculations

The total relevant loads have been distributed over the plan area of the ceiling void to allow the existing and proposed load conditions on the main building structure to be assessed.

Ceiling void plan area = $6.8m^2$

Existing load Element $[kN/m^2]$ Notes 0.20 Ceiling: Lightweight I joists with no top boarding Includes 109kg hot water cylinder and framing Services: 0.20 Partition: Upper portion of lightweight partition 0.25 Imposed: 0.25 Eurocode standard for ceiling voids TOTAL: 0.90

Proposed load

Element	[kN/m²]	Notes
Ceiling:	0.40	Increased to allow for top boarding/finishes
Services:	0.05	Reduced following removal of hot water cylinder
Balustrade:	0.15	Replacing upper portion of partition
Imposed:	0.25	Eurocode standard for ceiling voids
TOTAL:	0.85	

Appendix B - Structural Drawings





PROPOSED PLAN SCALE 1:50

SDADK	Project:	Stage:	PRELIMIN
	Apartment 3.10 St Pancras Chambers, Euston Road, London, NWI ZAR	Scale: NOTED	@ A3 Date:
21 COURT ROAD SOUTH, CAERPHILLY, CF83 2QW		Drawn: LE	Checked: LE
LLOYD@SPARKSTRUCTURES.COM 07367 079607	PROPOSED PLANS	Drawing No:	2304-S-20



Appendix C - Structural Calculations

SPARK STRUCTURES.	Project Apartment 3.10) St Pancras Ch	Job no. 2304			
21 Court Road South	Calcs for			Start page no./Revision		
Caerphilly	Partition Wall Studs			1		
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LE	06/12/2023	LE	06/12/2023	LE	06/12/2023

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

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex Tedds calculation version 1.0.07

Stud details

Description Restraint in plane of panel Stud spacing Stud height Panel height 47 x 75 C16 timber studs Dwangs (x 1) sstud = **400** mm Istud = **2200** mm IPanel = Istud + 2 × b =**2294** mm



Forces input on Stud

Permanent distributed load on top rail Imposed distributed load on top rail

Stud loading details

Point loads

Total vertical permanent point load (2200 mm) Total vertical imposed point load (2200 mm) Lg_Stud= **0.60** kN/m Lq_Stud= **3.60** kN/m

$$\begin{split} P_{G_1} = L_{G_Stud} \times \texttt{SStud} = \textbf{0.24 kN} \\ P_{Q_1} = L_{Q_Stud} \times \texttt{SStud} = \textbf{1.44 kN} \end{split}$$

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	12.9	0.7	0.055	PASS
Column stability check				0.169	PASS

ANALYSIS

Loading

Self weight included (Permanent x 1)

Tedds calculation version 1.0.37

Project	Job no.					
Apartment 3.10	St Pancras Ch	2304				
Calcs for					Start page no./Revision	
Partition Wall Studs			2			
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
LE	06/12/2023	LE	06/12/2023	LE	06/12/2023	
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Load combination factors

Load combination	Permanent	Imposed	mous	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	Point load	GlobalZ	0.24 kN at 2.2 m
Member	Imposed	Point load	GlobalZ	1.44 kN at 2.2 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection

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	Caerphilly	-	Calcs for	Partition	Wall Studs		Start page no./F	Revision 3
		-	Calcs by	r				-
			LE	Calcs date 06/12/2023	Checked by LE	Checked date 06/12/2023	Approved by LE	Approved date 06/12/2023
			1.00G +	1.00Q (Service) - Total defle	ction		
Node c Load c	leflections ombination:	1.35G + 1.5	0Q (Strength)	1 Y				
Node	Defle	ction	Rotation	Co-ordinate system				
	X (mm)	Z (mm)	(0)					
1	(mm)	(mm)	(*)					
2	0	02	0					
	U	0.2						
Node	Defle X	ction	Rotation	Co-ordinate system				
	(mm)	(mm)	(°)					
1	0	0	0					
2	0	0.1	0					
Total b	ase reaction	s						
Load	case/combin	ation	Force	e E7				
			(kN)	(kN)				
			0	2.5				
1.35G	+ 1.50Q (Stre	enatni						
			(kN)	(kN) 2.5				

SPARK STRUCTURES.	Project Apartment 3.10	Job no. 2304				
21 Court Road South Caerphilly	Calcs for	Start page no./Revision 4				
	Calcs by LE	Calcs date 06/12/2023	Checked by LE	Checked date 06/12/2023	Approved by LE	Approved date 06/12/2023

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.2	1	-2.5	0	0
		2	0	0	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.2	1	-1.7	0	0
		2	0	0	0

Forces

Strength combinations - Moment envelope (kNm)

Member results

Envelope - Strength combinations

Member	Position	Shear	force	Mon	nent
	(m)	(k	N)	(kN	lm)
Member	0	0		0	

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γм = **1.300**

Member details

Load duration - cl.2.3.1.2

Short-term

Tedds calculation version 2.2.20

	Project			D	Job no.	2004
SPARK STRUCTURES.	Apartment 3.1	0 St Pancras Cr	ambers, Eusto	n Road, London,	2	2304
21 Court Road South Caerphilly	Calcs for	Partition	Mall Stude		Start page no./I	Revision 5
	O al a a hui				A	
	LE	06/12/2023	LE	06/12/2023	LE	06/12/2023
Service class - cl 2 3 1 3	•	1	•	•	•	-
Timber section details		I				
Number of timber sections in m	ember	N – 1				
Breadth of sections		h = 47 mm				
Depth of sections		h = 75 mm				
Timber strength class - EN 338:	2016 Table 1	C16				
47						
		Cross-section Section modu Section modu Second mome Radius of gyra Radius of gyra Timber stren Characteristic Characteristic Characteristic Characteristic Characteristic Characteristic Mean modulu Fifth percentili Shear modulu Characteristic Mean density.	al area, A, 3525 mm ² lus, W, 44062.5 mm ³ us, W, 27613 mm ³ ent of area, I, 1652344 ent of area, I, 648894 n titon, I, 21.7 mm titon, I, 13.6 mm gth class C16 bending strength, f _{mb} , 3.2 compression strength (compression strength (compression strength (compression strength (s of elasticity, E _{comm} , 84 e modulus of elasticity, s of elasticity, G _{max} , 310 kg/m ³ p _{max} , 370 kg/m ³	mm ⁴ mm ⁴ 16 N/mm ² 2 N/mm ² parallel to grain, f _{cmx} , 17 N/ perpendicular to grain, f _{cm} el to grain, f _{cm} , 8.5 N/mm ² 000 N/mm ² E _{cor} , 5400 N/mm ² 20 N/mm ²	mm= , 2.2 N/mmi	
Span details Bearing length <u>Consider Combination 1 - 1.35</u>	5G + 1.50Q (Str	L₀ = 100 m rength <u>)</u>	m			
Modification factors						
Duration of load and moisture co	ontent - Table 3	8.1 kmod = 0.9				
Deformation factor - Table 3.2		kdef = 0.6				
System strength factor - cl.6.6		$K_{sys} = 1.1$				
Check compression parallel to	o the grain - cl.	.6.1.4				
Design axial compression		Pd = 2.516	kN	0		
Design compressive stress		$\sigma_{c,0,d} = P_d /$	A = 0.714 N/m	m²		
Design compressive strength		$f_{c,0,d} = K_{mod}$	× Ksys × tc.0.k / γΜ	1 = 12.946 N/mm ²		
DAC		σc,0,d / fc,0,d :	= 0.055	ando docian na	rallal compr	accion atraca
PAS	S - Design para	allel compressio	on strength ex	ceeds design pa	rallel compr	ession stress
Check columns subjected to e	either compres	sion or combin	ed compression	on and bending	- cl.6.3.2	
Effective length for y-axis bendir	ng	$L_{e,y} = 0.9 \times$	2200 mm = 19	980 mm		
Slenderness ratio		$\lambda_y = L_{e,y} / i_y$	= 91.452			
Relative slenderness ratio - exp	. 6.21	$\lambda_{rel,y} = \lambda_y / \pi$	τ × √(fc.0.κ / E0.05) = 1.633		
Effective length for z-axis bendir	ng	L _{e,z} = 1100	mm			
Slenderness ratio		$\lambda_z = L_{e,z} / i_z$	= 81.075			
Relative slenderness ratio - exp	. 6.22	$\lambda_{rel,z} = \lambda_z / z$	τ×√(fc.0.k / E0.05	b) = 1.448		
		Both λ rel	,y > 0.3 and λ re	el,z > 0.3, column	stability che	ck is required
Straightness factor		$\beta c = 0.2$				
Instability factors - exp.6.25, 6.2	6, 6.27 & 6.28	$k_y = 0.5 \times ($	1 + $\beta_c \times (\lambda_{rel,y} -$	0.3) + λrel,y ²) = 1.9	67	
		$k_z = 0.5 \times ($	1 + β c $ imes$ (λ rel,z -	$(0.3) + \lambda_{rel,z^2}) = 1.6$	63	

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$k_{cy} = 1 / (k_y + \sqrt{k_y^2} - \lambda_{rely^2}) = 0.326$ $k_{cz} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rely^2})}) = 0.403$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{cy} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{cy} \times f_{c,0,d}) = 0.137$ PASS - Column stability is acceptable Check columns subjected to either compression or combined compression and bending - cl.6.3.2 Effective length for y-axis bending $L_{ey} = 0.9 \times 2200 \text{ mm} = 1980 \text{ mm}$ Slenderness ratio $\lambda_y = L_{ey} / i_y = 91.452$ Relative slenderness ratio - exp. 6.21 $\lambda_{rely} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0,0,k})} = 1.633$ Effective length for z-axis bending $L_{ez} = 1100 \text{ mm}$ Slenderness ratio $\lambda_z = L_{e,z} / i_z = 81.075$ Relative slenderness ratio - exp. 6.22 $\lambda_{relz} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0,0,k})} = 1.448$ Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is required Straightness factor Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 1.967$ $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 1.663$ $k_{cy} = 1 / (k_x + \sqrt{(k_x^2 - \lambda_{rel,y}^2)}) = 0.326$ $k_{cy} = 1 / (k_x + \sqrt{(k_x^2 - \lambda_{rel,y}^2)}) = 0.403$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{cy} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{cy} \times f_{c,0,d}) = 0.169$					0.000			
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Column stability checks - exp. 6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,x} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,x} \times f_{c,0,d}) = 0.137$ PASS - Column stability is acceptableCheck columns subjected to either compression or combined compression and bending - cl.6.3.2Effective length for y-axis bending $L_{e,y} = 0.9 \times 2200 \text{ mm} = 1980 \text{ mm}$ Slenderness ratio $\lambda_y = L_{e,y} / iy = 91.452$ Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0,05})} = 1.633$ Effective length for z-axis bending $L_{e,z} = 1100 \text{ mm}$ Slenderness ratio $\lambda_z = L_{e,z} / iz = 81.075$ Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} > 0.3 \text{ and } \lambda_{rel,z} > 0.3, \text{ column stability check is required}$ Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is requiredStraightness factor $\beta_c = 0.2$ Instability factors - exp. 6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_0 \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y^2}) = 1.967$ $k_z = 0.5 \times (1 + \beta_0 \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y^2}) = 1.663$ $k_{c,x} = 1 / (k_x + \sqrt{(k_x^2 - \lambda_{rel,x^2})}) = 0.403$ Column stability checks - exp. 6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,x} \times f_{c,0,d}) = 0.137$			$k_{c,z} = 1 / (k_z)$	+ $\sqrt{(kz^2 - \lambda_{rel,z^2})}$	= 0.403			
$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$ PASS - Column stability is acceptable PASS - Column stability is acceptable Check columns subjected to either compression or combined compression and bending - cl.6.3.2 Effective length for y-axis bending Sienderness ratio $\lambda_{y} = L_{e,y} / i_{y} = 0.9 \times 2200 \text{ mm} = 1980 \text{ mm}$ Sienderness ratio $\lambda_{y} = L_{e,y} / i_{y} = 91.452$ Relative sienderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_{y} / \pi \times \sqrt{(f_{c,0,k} / E_{0,05})} = 1.633$ Effective length for z-axis bending Sienderness ratio $\lambda_{z} = L_{e,z} / i_{z} = 81.075$ Relative sienderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_{z} / \pi \times \sqrt{(f_{c,0,k} / E_{0,05})} = 1.448$ Both $\lambda_{rel,z} > 0.3$, column stability check is required Straightness factor Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_{y} = 0.5 \times (1 + \beta_{c} \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^{2}) = 1.967$ $k_{z} = 0.5 \times (1 + \beta_{c} \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^{2}) = 1.663$ $k_{c,z} = 1 / (k_{z} + \sqrt{(k_{z}^{2} - \lambda_{rel,z}^{2})}) = 0.403$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,x} \times f_{c,0,d}) = 0.137$ DASS. Column stability is acceptable	Column stability checks - exp.6	.23 & 6.24	$\sigma_{c,0,d} / (k_{c,y} >$	< f _{c,0,d}) = 0.169				
PASS - Column stability is acceptableCheck columns subjected to either compression or combined compression and bending - cl.6.3.2Effective length for y-axis bendingLe.y = 0.9 × 2200 mm = 1980 mmSlenderness ratio $\lambda_y = L_{e,y} / iy = 91.452$ Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / Eo.05)} = 1.633$ Effective length for z-axis bendingLe.z = 1100 mmSlenderness ratio $\lambda_z = L_{e,z} / iz = 81.075$ Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / Eo.05)} = 1.448$ Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is requiredStraightness factor $\beta_c = 0.2$ Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.967$ $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,y}^2) = 1.663$ $k_{c,z} = 1 / (k_x + \sqrt{(k_{c,2} - \lambda_{rel,x}^2)}) = 0.403$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,x} \times f_{c,0,d}) = 0.137$ DASE			$\sigma_{c,0,d} / (k_{c,z} >$	< f _{c,0,d}) = 0.137				
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Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} = 1.633$ Effective length for z-axis bending $L_{e,z} = 1100 \text{ mm}$ Slenderness ratio $\lambda_z = L_{e,z} / iz = 81.075$ Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} > \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} = 1.448$ Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is requiredStraightness factor $\beta_c = 0.2$ Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y^2}) = 1.967$ $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z^2}) = 1.663$ $k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y^2})}) = 0.326$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$	Slenderness ratio		$\lambda_y = L_{e,y} / i_y$	= 91.452				
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Slenderness ratio $\lambda_z = L_{e,z} / iz = 81.075$ Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} = 1.448$ Both $\lambda_{rel,y} > 0.3$ and $\lambda_{rel,z} > 0.3$, column stability check is requiredStraightness factor $\beta_c = 0.2$ Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.967$ $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 1.663$ $k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.326$ $k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 0.403$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$ DASS	Effective length for z-axis bendi	ng	L _{e,z} = 1100	mm				
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Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_{y} = 0.5 \times (1 + \beta_{c} \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^{2}) = 1.967$ $k_{z} = 0.5 \times (1 + \beta_{c} \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^{2}) = 1.663$ $k_{c,y} = 1 / (k_{y} + \sqrt{(k_{y}^{2} - \lambda_{rel,y}^{2})}) = 0.326$ $k_{c,z} = 1 / (k_{z} + \sqrt{(k_{z}^{2} - \lambda_{rel,z}^{2})}) = 0.403$ Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$	Straightness factor		$\beta c = 0.2$					
$\begin{aligned} k_z &= 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \textbf{1.663} \\ k_{c,y} &= 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \textbf{0.326} \\ k_{c,z} &= 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \textbf{0.403} \\ \hline \sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = \textbf{0.169} \\ \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \textbf{0.137} \end{aligned}$	Instability factors - exp.6.25, 6.2	26, 6.27 & 6.28	$k_y = 0.5 \times (1)^{-1}$	$1 + \beta c \times (\lambda_{rel,y} - 0)$.3) + λ _{rel,y²}) = 1.9	67		
$\begin{aligned} k_{c,y} &= 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \textbf{0.326} \\ k_{c,z} &= 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \textbf{0.403} \\ \\ \sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = \textbf{0.169} \\ \\ \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \textbf{0.137} \end{aligned}$			$k_z = 0.5 \times (2)$	$1 + \beta c \times (\lambda rel, z - 0)$.3) + λ _{rel,z²}) = 1.6	63		
Column stability checks - exp.6.23 & 6.24 $k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z^2})}) = 0.403$ $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$			$k_{c,y} = 1 / (k_y)$	+ $\sqrt{(k_y^2 - \lambda_{rel,y^2})}$	= 0.326			
Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.169$ $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.137$			k _{c,z} = 1 / (k _z	+ $\sqrt{(k_z^2 - \lambda_{rel,z^2})}$	= 0.403			
$\sigma_{c,0,d} / (\mathbf{k}_{c,z} \times \mathbf{f}_{c,0,d}) = 0.137$	Column stability checks - exp.6	.23 & 6.24	σ c,0,d / (kc,y >	< f _{c,0,d}) = 0.169				
DASS Column stability is assertable			$\sigma_{c,0,d} / (k_{c,z})$	< f _{c,0,d}) = 0.137				
PASS - Column stability is acceptable				-	PASS - Colu	mn stability is	s acceptable	

SPARK STRUC	TURES.	Apartment	: 3.10 S	St Pano	cras Ch	namber	s, Eusto	on Road, London.	Job no.	2304
21 Court Road S	South	Calcs for							Start page no./	Revision
Caerphilly				Pa	artition	Wall St	uds			7
		Calcs by	Ca	alcs date	е	Checke	ed by	Checked date	Approved by	Approved
		LE		06/12/	/2023		LE	06/12/2023	LE	06/12/2
In accordance we values Joist details Description Joist spacing Forces input on Vertical permaner Vertical imposed Joist loading det	Joist Ioad on joi oad on joist ails	1-1:2004 + A	2:2014	47 x SJoist —24 Fc_J Fo_J	poratir (100 C = 400 00 00 00 00 00 00 00 00 00	16 timt mm 35 kN/m 25 kN/m	igendu ber joists	m June 2006 and	I the recomm	nended
	it load on joi	st		р <u></u> =	FG_Jois	t × S Joist	= 0.14 k	۸/m		
Vertical imposed	oad on joist	st	Unit	pg = pg =	FG_Jois FQ_Jois	t × SJoist t × SJoist	= 0.14 k = 0.10 k	kN/m kN/m	ilisation	Result
Vertical imposed Member results	oad on joist	st	Unit N/mr	pg = pq = Ca n ² 1.	FG_Jois FQ_Jois Apacity	t × SJoist t × SJoist	= 0.14 k = 0.10 k Max	xN/m xN/m iimum Ut	ilisation	Resul
Vertical imposed Member results s Bearing stress Bending stress	oad on joist	st	Unit N/mm	pg = pq = Ca n ² 1.7 n ² 13	FG_Jois FQ_Jois apacity 7 3.2	t × SJoist t × SJoist	= 0.14 k = 0.10 k Max 0.1 3.3	KN/m KN/m iimum Ut 0. 0.	t ilisation 068 249	Resul PASS PASS
Vertical imposed Member results Bearing stress Bending stress Shear stress	oad on joist	st	Unit N/mm N/mm	pg = pa =	FG_Jois FQ_Jois apacity 7 3.2 4	t × SJoist t × SJoist	= 0.14 k = 0.10 k Max 0.1 3.3 0.2	KN/m KN/m iimum Ut 0. 0. 0.	t ilisation 068 249 084	Resul PASS PASS PASS
Vertical imposed Member results Bearing stress Bending stress Shear stress Deflection	oad on joist	st	Unit N/mm N/mm N/mm	pg = pq = Ca n ² 1. n ² 13 n ² 2.4 6.	= F _{G_Jois} = F _{Q_Jois} = F Q_Jois = P = P P = P P P P P P P P P P	t × SJoist t × SJoist	= 0.14 k = 0.10 k 0.1 3.3 0.2 5.0	<n m<br=""><n m<br=""><a href="https://www.sciencescomescomescomescomescomescomescomesco</th><th>tilisation 068 249 084 750</th><th>Resul PASS PASS PASS PASS</th></n></n>	t ilisation 068 249 084 750	Resul PASS PASS PASS PASS
Vertical imposed I Member results a Bearing stress Bending stress Shear stress Deflection ANALYSIS Loading Self weight includ Load combination	ed (Permano	ent x 1)	Unit N/mm N/mm M/mm	pg = pa = Ca n ² 1.3 n ² 2.4 6.7	FG_Jois FQ_Jois apacity 7 3.2 4 7	t × SJoist t × SJoist	= 0.14 k = 0.10 k 0.1 3.3 0.2 5.0	<n m<br=""><<u>iimum Ut</u> 0. 0. 0. 0. 0.</n>	t illisation 068 249 084 750 Tedds calcula	Result PASS PASS PASS
Vertical imposed I Member results = Bearing stress Bending stress Shear stress Deflection ANALYSIS Loading Self weight includ Load combinatio Load	ed (Permano n factors	ent x 1)	Unit N/mm N/mm N/mm	pg = pa = Ca n ² 1.1 n ² 1.3 n ² 2.4 6.7 4 5 6 1 .35	E FG_Jois E FQ_Jois apacity 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 7 7 7 7 7 7 7 7 7 7 7	t × SJoist t × SJoist	= 0.14 H = 0.10 H 0.1 3.3 0.2 5.0	xN/m xN/m 0. 0. 0. 0. 0.	tillisation 068 249 084 750 Tedds calcula	Resul PASS PASS PASS
Vertical imposed I Member results = Bearing stress Bending stress Deflection ANALYSIS Loading Self weight includ Load combinatio Load Load GG + 1.50Q (Strent 0G + 1.00Q (Servit	ed (Permand n factors	ent x 1)	Unit N/mr N/mr M/mr	pg = pq = Pq = Pq = Pq = Pq = 1.3 Cz 1.3 6.7	E FG_Jois E FQ_Jois Apacity 7 3.2 4 7 7 3.2 4 7 7 1.50 1.00	t × SJoist t × SJoist , , , , , , , , , , , , , , , , , , ,	= 0.14 = 0.10 Max 0.1 3.3 0.2 5.0 5.0	<n <="" m="" td=""> imum Ut 0.1 0.2 0.3 0.4 0.5 0.6</n>	t ilisation 068 249 084 750 Tedds calcula	Resul PASS PASS PASS
Vertical imposed I Member results = Bearing stress Bending stress Shear stress Deflection ANALYSIS Loading Self weight includ Load combinatio Load 5G + 1.50Q (Stren 0G + 1.00Q (Servi 0G + w21.00Q (Qui	ed (Permano n factors combinatio gth) ce) asi)	ent x 1)	Unit N/mr N/mr mm	pg = pq = pq = 1.35 1.00 1.00 1.00	E FG_Jois E FQ_Jois apacity 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 7 7 7 7 7 7 7 7 7 7 7	t × SJoist t × SJoist 7	= 0.14 k = 0.10 k 0.1 3.3 0.2 5.0 5.0	<n <p="" m="">imum imum Ut 0. 0. 0.1 0. 0.2 0. 0.3 0.</n>	t illisation 068 249 084 750 Tedds calcula	Resul PASS PASS PASS
Vertical imposed I Member results : Bearing stress Bending stress Shear stress Deflection ANALYSIS Loading Self weight includ Load combinatio Load 5G + 1.50Q (Stren 0G + 1.00Q (Servi 0G + ψ21.00Q (Qu	ed (Permand oad on joist summary ed (Permand on factors combination gth) ce) asi)	ent x 1)	Unit N/mm N/mm mm	pg = pq = pq = Ca n ² 1.3 n ² 2.4 6.7 6.7 1.35 1.00 1.00	E FG_Jois FQ_Jois Apacity 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 7 3.2 4 7 1.50 1.00 0.30	t × SJoist t × SJoist 7 0.00 0.00 0.00	= 0.14 k = 0.10 k 0.1 3.3 0.2 5.0 5.0	xN/m xN/m 0. 0. 0. 0. 0.	tilisation 068 249 084 750 Tedds calcula	Result PASS PASS PASS
Vertical imposed Member results = Bearing stress Bending stress Shear stress Deflection ANALYSIS Loading Self weight includ Load combinatio Load 5G + 1.50Q (Stren 0G + 1.00Q (Servi 0G + ψ21.00Q (Qu Member Loads Member	ed (Permano n factors combinatio gth) ce) asi)	ent x 1)	Unit N/mr N/mr mm	pg = pq = pq = 1.3 1.35 1.00 1.00	E FG_Jois FQ_Jois 7 3.2 4 7 7 7 3.2 4 7 7 3.2 4 7 7 1.50 1.50 1.00 0.30	t × SJoist t × SJoist , , , , , , , , , , , , , , , , , , ,	= 0.14 k = 0.10 k 0.1 3.3 0.2 5.0 5.0 0.00 0.00 0.00 0.00	<pre>sN/m simum Ut</pre>	tilisation 068 249 084 750 Tedds calcula	Result PASS PASS PASS ation version 1

UDL

GlobalZ

0.1 kN/m at 0 m to 2.4 m

Imposed

Member

SDVE			Project Apartment 3	10 St Pancras	Chambers	Fuston	Road London	Job no.	304
2 [′]	Court Road South	Υ <u>L</u> Ο. h	Calcs for		Chambere	Lacton		Start page no./F	Revision
	Caerphilly			Partiti	on Wall Stu	ds			8
			Calcs by LE	Calcs date 06/12/202	Checked 23 L	by .E	Checked date 06/12/2023	Approved by LE	Approved date 06/12/2023
<u>Result</u> Total o	$\frac{s}{leflection}$		1.35G 1.00G	+ 1.50Q (Stree + 1.00Q (Serv 	ngth) - Tota vice) - Total	I deflect	tion 	2 —Å	
Node	leflections							A	
Load	ombination:	1.35G + 1.	50Q (Strength	ı)					
Node	Defle	ction	Rotation	n Co-ordina	ite				
	x	z		System					
	(mm)	(mm)	(°)						
1	0	0	0.37737	,					
2	0	0	-0.37737	7					
Load o	ombination:	1.00G + 1.	00Q (Service)						
Node	Defle	ction	Rotation	n Co-ordina system	ite				
	x	z							
	(mm)	(mm)	(°)						
1	0	0	0.26783						
2	0	0	-0.26783	3					
Load o	ombination:	1.00G + ψ	21.00Q (Quasi)	1				
Node	Defle	ction	Rotation	n Co-ordina system	ite				
	X	Z							
1	(mm)	(mm)	(°)						
	0	0							
		• •	-0.1341						
	case/combin	ation	For	ce					
Luau			FX	FZ					
			(kN)	(kN)					
1.35G	+ 1.50Q (Stre	ength)	0	0.9					
1.000	6 + 1.00Q (Ser	vice)	0	0.6					

SPARK STRUCTURES Apartment 3.10 St Pancras Chambers, Euston Road, London, 204 Start page no.Results Start page no.Results 9 Calcs for Partition Wall Studs 9 Calcs for Calcs date Off(12)2023 Approved by Approved by Le Oe(12)2023 Calcs date Off(12)2023 Approved by Approved by Load case/combination FX FZ (kN) (kN) (kN) Approved by <		Job no.				Project			
$ \frac{21 \operatorname{Court Read South Casephily} }{2 \operatorname{Calcs Sor}} \frac{\operatorname{Calcs Sor} }{\operatorname{LE} } \frac{\operatorname{Calcs Calca Case } \operatorname{Court Case Case } \operatorname{Court Case } \operatorname{Court Case } \operatorname{Calca Case } $		2304	on Road, London,	ambers, Eust	St Pancras Ch	Apartment 3.10	RES.	K STRUCTU	SPAR
Image: Constraint of the	on	Start page no./Revis		Vall Studs	Partition \	Calcs for	h	Court Road Sout Caerphilly	21
Image: Control table Contro table Control table Co			Chockod data	Chockod by		Calco by			
Load case/combination FX FZ (KN) (KN) (KN) 1.00G + wr1.00Q (Quasi) 0 0.4 Element end forces Load combination: 1.365 et 1.50Q (Strength) Load combination: 1.365 et 1.50Q (Strength) Modes Axial force Shear force Moment 1 2.4 1 0 0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.3 0 <td>6/12/202</td> <td>LE</td> <td>06/12/2023</td> <td>LE</td> <td>06/12/2023</td> <td>LE</td> <td></td> <td></td> <td></td>	6/12/202	LE	06/12/2023	LE	06/12/2023	LE			
FX FZ 1.00 + w21.00Q (Quasi) 0 Determent Determent Determent Determent Determent Construction:						Force	ation	case/combin	Load
(kN) (kN) 1.00G + yz1.00Q (Quasi) 0 0.4 Element end force Edd combination: 1.55 + 1.50Q (Strength) Edd combination: 1.55 + 1.50Q (Strength) Element Imp in Xodes Xaial force Shear force Moment 1 2.4 1 0 0.4 0 1 2.4 1 0 0.4 0 1 2.4 1 0 0.4 0 1 2.4 1 0 0.4 0 Lod combination: 1.00G + Iwz1.00Q (Service) Element Moment (KN) 0.03 0 Lod combination: 1.00G + wz1.00Q (Quasi) Element 1 0.4 1 0 0.2 0 Lod combination: 1.00G + wz1.00Q (Quasi) Element 1 0.4 1 0 0.2 0 0 0 Jos Strength combinations - Moment envelope (kNm) 0.3 0.3 0.3 0.3 0.3 0.3 0.3 0.3 0.3					FZ	FX			
1.00G + ψ21.00Q (Quasi) 0 0.4 Element end forces Lod combination: 1.35G + 1.50Q (Strength) Element Nodes Axial force Moment 1 2.4 1 0 0.4 1 2.4 1 0 0.4 1 2.4 1 0 0.4 Length Nodes Axial force Moment 1 2.4 1 0 0.4 Load combination: 1.00G + 1.00Q (Service) Element Modes Axial force Shear force Moment 1 2.4 1 0 0.3 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Modes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 0 Forces Strength combinations - Moment envelope (kNm) Moment 0.3 0 0 0.3 0.					(kN)	(kN)			
Element forces Indext for the function of the functio					0.4	0	Quasi)	+ ψ21.00Q (C	1.00G
Lear combination: 1.35 + 1.500 (Strength) intermative intermatinte intermative intermative intermative interm								nt end forces	Elemer
Element Length (m) Nodes Axial force (kN) Shear force (kNm) Moment (kNm) 1 2.4 1 0 -0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Moment (kN) (kNm) 1 2.4 1 0 -0.3 0 1 2.4 1 0 -0.3 0 Load combination: 1.00G + w21.00Q (Quasi) Element K(N) K(N) (kN) 1 2.4 1 0 -0.2 0 - 1 2.4 1 0 -0.2 0 - - 1 2.4 1 0 -0.2 0 - - 5 5 3 - - - - 1 0 - <td></td> <td></td> <td></td> <td></td> <td></td> <td>0Q (Strength)</td> <td>1.35G + 1.5</td> <td>ombination:</td> <td>Load c</td>						0Q (Strength)	1.35G + 1.5	ombination:	Load c
(m) Start/End (kN) (kN) (kNm) 1 2.4 1 0 -0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Moment (kN) (kNm) (kNm) 0				Moment	Shear force	Axial force	Nodes	Length	Element
1 2.4 1 0 -0.4 0 Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.3 0 1 2.4 1 0 -0.3 0 1 2.4 1 0 -0.3 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Kint/End (KN) (KN) (KNm) 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 Forces Strength combinations - Moment envelope (kNm) 0.3 0.3 0.3 0.3 0.3 0.3 0.4 0.3 0.3 0.3 <td colspan="</td> <td></td> <td></td> <td></td> <td>(kNm)</td> <td>(kN)</td> <td>(kN)</td> <td>Start/End</td> <td>(m)</td> <td></td>				(kNm)	(kN)	(kN)	Start/End	(m)	
Image: Load combination: 1.00G + 1.00Q (Service) Element Image: Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.3 0 1 2.4 1 0 -0.3 0 1 2.4 1 0 -0.3 0 Lead combination: 1.00G + w21.00Q (Quasi) Image: Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 0 0 1 2.4 1 0 -0.2 0 </td <td></td> <td></td> <td></td> <td>0</td> <td>-0.4</td> <td>0</td> <td>1</td> <td>2.4</td> <td>1</td>				0	-0.4	0	1	2.4	1
Load combination: 1.00G + 1.00Q (Service) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.3 0 2 0 -0.3 0 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Length Nodes Axial force Shear force Moment (m) Start/End (kN) (kN) (kNm) 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 Strength combinations - Moment envelope (kNm) 0.3 Other combinations - Shear envelope (kN)				0	-0.4	0	2		
Element Length (m) Nodes Start/End Axial force (kN) Shear force (kNm) Moment (kNm) 1 2.4 1 0 -0.3 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Length (m) Nodes Axial force Shear force Moment (kNm) 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 Strength combinations - Moment envelope (kNm) 0 0 0.3						0Q (Service)	1.00G + 1.0	ombination:	Load c
(m) Start/End (kN) (kNm) 1 2.4 1 0 -0.3 0 2 0 -0.3 0 0 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 1 2.4 1 0 -0.2 0 0 Forces Strength combinations - Moment envelope (kNm) 0				Moment	Shear force	Axial force	Nodes	Length	Element
1 2.4 1 0 -0.3 0 Load combination: 1.00G + ψ21.00Q (Quasi) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 Forces Forces Strength combinations - Moment envelope (kNm) 03 03 03				(kNm)	(kN)	(kN)	Start/End	(m)	
Load combination: 1.00G + ψ21.00Q (Quasi) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 Forces Strength combinations - Moment envelope (kNm) 0.3 Strength combinations - Shear envelope (kN) 0.3 Strength combinations - Shear envelope (kN)				0	-0.3	0	1	2.4	1
Load combination: 1.00G + ψ21.00Q (Quasi) Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 1 2.4 1 0 -0.2 0 Forces Strength combinations - Moment envelope (kNm) 0.3 Strength combinations - Shear envelope (kN) 0.4				0	-0.3	0	2		
Element Length Nodes Axial force Shear force Moment 1 2.4 1 0 -0.2 0 Torces Forces Strength combinations - Moment envelope (kNm) 0.3 Strength combinations - Shear envelope (kN) 0.3						I.00Q (Quasi)	1.00G + ψ ²	ombination:	Load c
Image:				Moment	Shear force	Axial force	Nodes	Length	Element
1 2.4 1 0 -0.2 0 Forces Strength combinations - Moment envelope (kNm) 0.3 Strength combinations - Shear envelope (kN) 0.3 Strength combinations - Shear envelope (kN) 0.4 0.4				(kNm)	(kN)	(kN)	Start/End	(m)	4
Forces Strength combinations - Moment envelope (kNm) 0.3 Strength combinations - Shear envelope (kN) 0.4				0	-0.2	0	2	2.4	1
Strength combinations - Moment envelope (kNm)				0	-0.2	0	2		
Strength combinations - Moment envelope (kNm)								i	Forces
0.3 Strength combinations - Shear envelope (kN)			ope (kNm)	oment envel	binations - M	Strength con			
0.3 Strength combinations - Shear envelope (kN)		A					_	4	
0.3 Strength combinations - Shear envelope (kN)									
0.3 Strength combinations - Shear envelope (kN)					0.2				
Strength combinations - Shear envelope (kN)					0.5				
Strength combinations - Shear envelope (kN)									
0.4			ope (kN)	Shear envelo	ombinations -	Strength co			
								0.4	
Å									
Å Å									
		æ						Å	
.0.4		-0.4							
-0.4		-0.4							
								-	

Envelope - Strength combinations

Member	Position	Shear	r force	Мо	ment
	(m)	(k	N)	(k	Nm)
Member	0	0.4 (max abs)		0 (min)	
	1.2	0		0.3 (max)	
	2.4	-0.4		0 (min)	
	•				

Tedds calculation version 2.2.20

	1				1	
SPARK STRUCTURES.	Project Apartment 3.10	St Pancras Ch	ambers, Eusto	on Road, London,	Job no. 23	304
21 Court Road South	Calcs for				Start page no./R	evision
Caerphiliy		Partition \	Wall Studs			10
	Calcs by LE	Calcs date 06/12/2023	Checked by LE	Checked date 06/12/2023	Approved by LE	Approved date 06/12/2023
Member - Span 1 Partial factor for material prop Partial factor for material proper	perties and resis ties - Table 2.3	tances γм = 1.300				
Member details Load duration - cl.2.3.1.2 Service class - cl.2.3.1.3		Short-term 1				
Timber section details Number of timber sections in me Breadth of sections Depth of sections Timber strength class - EN 338: 47-07	ember 2016 Table 1 →I	N = 1 b = 47 mm h = 100 mm C16	n ton a, A, 4700 mm ² , 78333.3 mm ³ , 36817 mm ³ area, I ₄ , 3916667 mm ³ area, I ₄ , 365192 mm ⁴ , 28.9 mm , 13.6 mm ass C16 ng strength, f _{1,4} , 3.2 N/m ression strength parallel to (asticity, E _{0, man} , 8000 h ulus of elasticity, E _{0, add} asticity, G _{mean} , 500 N/ ty, p ₄ , 310 kg/m ³ 370 kg/m ³	/mm² m² lei to grain, f _{c.8.4} , 17 N/mm endicular to grain, f _{c.86.4} , 2. grain, f _{c.8.4} , 8.5 N/mm² N/mm² N/mm² 5400 N/mm² mm²	² 2 N/mm²	
Span details Bearing length		L _b = 50 mm	ı			
Consider Combination 1 - 1.3	5G + 1.50Q (Stre	ngth)				
Modification factors Duration of load and moisture c Deformation factor - Table 3.2 Depth factor for bending - Major Bending stress re-distribution fa Crack factor for shear resistanc System strength factor - cl.6.6	ontent - Table 3. axis - exp.3.1 actor - cl.6.1.6(2) e - cl.6.1.7(2)	1 kmod = 0.9 kdef = 0.6 kh,m,y = min km = 0.7 kcr = 0.67 ksys = 1.1	((150 mm / h) ^{0.}	² , 1.3) = 1.084		
Check design at start of span						
Check compression perpendi Design perpendicular compress Effective contact length Design perpendicular compress Design perpendicular compress	cular to the grai ion - major axis ive stress - exp.6 ive strength	n - cl.6.1.5 $F_{c,y,90,d} = 0.4$ $L_{b,ef} = L_{b} + 16$ 5.4 $\sigma_{c,y,90,d} = F_{c}$ $f_{c,y,90,d} = K_{max}$	43 kN min(Lb, 30 mm) ,y,90,d / (b × Lb,ef) od × ksys × fc.90,k /) = 80 mm) = 0.114 N/mm² / γ _M = 1.675 N/mm	1 ²	
PASS - Design pe	rpendicular com	pression stre	$\operatorname{ngth}\operatorname{exceeds}$	design perpendi	cular compre	ession stress

	Project				Job no.			
SPARK STRUCTURES.	Apartment 3	3.10 St Pancras Ch	2304					
21 Court Road South	Calcs for		Start page no./Revision					
Caerphilly		Partition	11					
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
	LE	06/12/2023	LE	06/12/2023	LE	06/12/2023		
Chack chaor force Section	617							
Design shear force	10.1.7	F _{y,d} = 0.43	kN					
Design shear stress - exp.6.6	0	$\tau_{y,d} = 1.5 \times$	$F_{y,d} / (k_{cr} \times b \times b)$	h) = 0.205 N/mm ²				
Design shear strength		$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.437 \text{ N/mm}^2$						
		$\tau_{y,d} / f_{v,y,d} =$	0.084					
		PAS	SS - Design s	near strength exc	eeas aesign	shear stress		
Check design 1200 mm alor	ng span	PA	SS - Design s	near strength exc	eeds design	shear stress		
<u>Check design 1200 mm alor</u> Check bending moment - Se	ng span ection 6.1.6	PA:	55 - Design s	near strength exc	eeds design	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment	ng span ection 6.1.6	PA: M _{y,d} = 0.25	55 - Design s 3 kNm	near strengtn exc	eeas aesign	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress	ng span ection 6.1.6	PA: M _{y,d} = 0.25 σ _{m,y,d} = M _{y,d}	55 - Design s 3 kNm / Wy = 3.293	near strengtn exc N/mm²	eeas aesign	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength	ng span ection 6.1.6	$M_{y,d} = 0.25i$ $\sigma_{m,y,d} = M_{y,d}$ $f_{m,y,d} = k_{h,m,y}$	55 - Design s 8 kNm / Wy = 3.293 / × kmod × ksys >	near strengtn exc N/mm² < fm.k / γM = 13.214	eeas aesign N/mm²	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength	ng span ection 6.1.6	$M_{y,d} = 0.254$ $\sigma_{m,y,d} = M_{y,d}$ $f_{m,y,d} = K_{h,m,y}$ $\sigma_{m,y,d} / f_{m,y,d}$	55 - Design s 8 kNm / Wy = 3.293 - × k _{mod} × k _{sys} > = 0.249	near strengtn exc N/mm² < fm.k / γм = 13.214	eeas aesign N/mm²	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength	ng span ection 6.1.6	PAS $M_{y,d} = 0.25i$ $\sigma_{m,y,d} = M_{y,d}$ $f_{m,y,d} = k_{h,m,y}$ $\sigma_{m,y,d} / f_{m,y,d}$ PASS - E	55 - Design s 8 kNm / Wy = 3.293 / × kmod × ksys > = 0.249 Design bendir	near strength exc N/mm² < fm.k / γM = 13.214 ng strength excee	:eeas aesign N/mm² ds design be	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength	ng span ection 6.1.6	PAS $M_{y,d} = 0.254$ $\sigma_{m,y,d} = M_{y,d}$ $f_{m,y,d} = k_{h,m,y}$ $\sigma_{m,y,d} / f_{m,y,d}$ PASS - E	55 - Design s 8 kNm / Wy = 3.293 / × k _{mod} × k _{sys} × = 0.249 Design bendir	near strength exc N/mm² fm.k / γм = 13.214 h ng strength excee	:eeas aesign N/mm² ds design be	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength Consider Combination 2 - 1.	ng span ection 6.1.6 .00G + 1.00Q (\$	PAS $M_{y,d} = 0.25i$ $\sigma_{m,y,d} = M_{y,d}$ $f_{m,y,d} = k_{h,m,y}$ $\sigma_{m,y,d} / f_{m,y,d}$ PASS - E <u>Service</u>	55 - Design s 3 kNm / Wy = 3.293 / × kmod × ksys > = 0.249 Design bendir	near strength exc N/mm² < fm.k / γM = 13.214 ng strength excee	:eeas aesign N/mm² ds design be	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength Consider Combination 2 - 1. Check design 1200 mm alor	ng span ection 6.1.6 .00G + 1.00Q (\$ ng span	PAS M _{y,d} = 0.25 σ _{m,y,d} = M _{y,d} f _{m,y,d} = k _{h,m,y} σ _{m,y,d} / f _{m,y,d} PASS - E <u>Service)</u>	3 kNm / Wy = 3.293 / × k _{mod} × k _{sys} × = 0.249 Design bendir	near strengtn exc N/mm² fm.k / γм = 13.214 h ng strength excee	:eeas aesign N/mm² ds design be	shear stress		
Check design 1200 mm alor Check bending moment - Se Design bending moment Design bending stress Design bending strength Consider Combination 2 - 1. Check design 1200 mm alor Check y-y axis deflection - S	ng span ection 6.1.6 .00G + 1.00Q (\$ ng span Section 7.2	PA: M _{y,d} = 0.25 σ _{m,y,d} = M _{y,d} f _{m,y,d} = k _{h,m,y} σ _{m,y,d} / f _{m,y,d} PASS - [<u>Service)</u>	55 - Design s 8 kNm / Wy = 3.293 / × kmod × ksys > = 0.249 Design bendir	near strength exc N/mm² < fm.k / γM = 13.214 h ng strength excee	:eeas aesign N/mm² :ds design be	shear stress		

Instantal leous deflection $\delta y = 3.6$ mmQuasi-permanent variable load factor $\psi_2 = 0.3$ Final deflection with creep $\delta_{y,Final} = 0.5 \times \delta_y \times (1 + k_{def}) + 0.5 \times \delta_y \times (1 + \psi_2 \times k_{def}) = 5$ mmAllowable deflection $\delta_{y,Allowable} = L_{m1_s1} / 360 = 6.7$ mm $\delta_{y,Final} / \delta_{y,Allowable} = 0.75$

PASS - Allowable deflection exceeds final deflection