WEARE Symmetrys Structural calculation Package

43A REDINGTON ROAD London NW3 7RA

REF: 21141 REV B





Revision History

Revision	Description Date		Ву	Checked
А	First Issue	31.08.21	SB	DS
В	Second Issue	14.01.22	SB	DS

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1. Introduction

1.1 The structural works for the BIA consisted of the following:

• The sub-structure designs associated with the new basement. This will include the design of the new ground bearing slab, design of underpins, new retaining walls and new suspended ground floors.

• The reconfiguration of the lower ground and ground floors with the removal of structure as can be seen from overlaying the existing plans with the proposed plans.

2.0 Design Codes

- 2.1 The following design codes/guidance were used to carry out the design:
 - BS EN 1991-1-1:2002 Actions on Structures
 - BS EN 1992-1-1:2004 Design of Concrete Structures
 - BS EN 1993-1-1:2005 Design of Steel Structures
 - BS EN 1996-1-1:2005 Design of Masonry Structures
 - CP 111 Masonry

3.0 Ground Conditions

3.1 Allowable bearing pressures of 60KN/m2 and 80KN/m2 have been adopted for the design of foundations at formation level of respectively +9.00 and +8.00. Refer to the site investigation report by Geofirma for details of the ground conditions.

4.0 Substructure Design

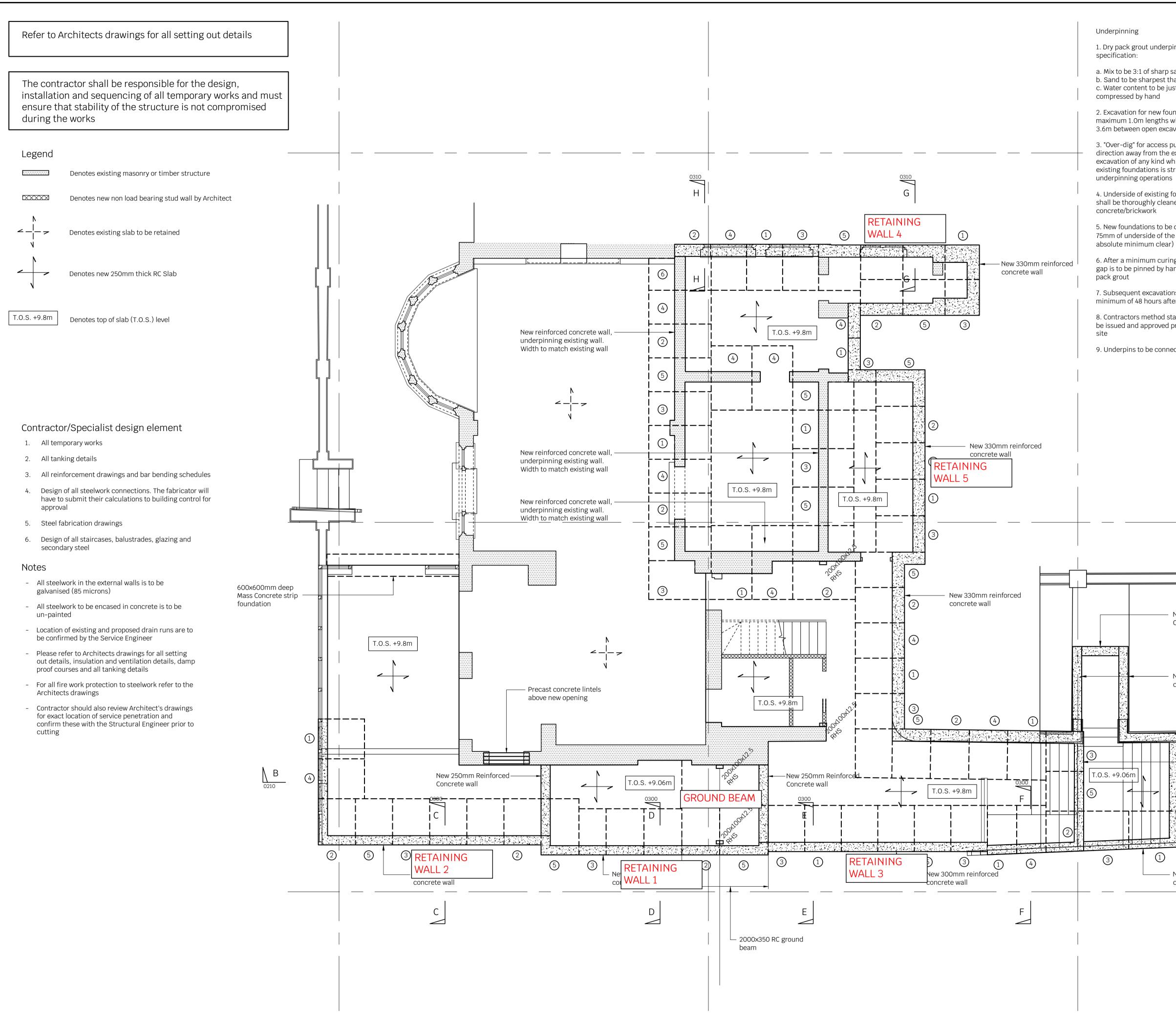
4.1 Existing footings determined from trial pits. Refer to the site investigation report by Geofirma.

5.0 Loading

5.1 The loadings used throughout the design are shown in the table below:



Item	DL (kPa)	LL (kPa)
Timber Roof		
Asphalt waterproofing	0.45	
Timber joists and insulation	0.20	
Ceiling and services	0.15	
Flat	0.90	0.60
Pitched	1.30	0.60
Timber Floor		
Ceiling and services	0.15	
Timber joists and insulation	0.20	
Plyboard and finishes	0.25	
Total	0.60	1.5
Existing Masonry (19kN/m ³)		
225mm thick	4.30	
Finishes	0.20	
Total	4.50	
Existing Masonry (19kN/m³)		
330mm thick	6.30	
Finishes	0.20	
Total	6.50	
Cavity Wall (19kN/m³)		
100mm thick blockwork	1.50	
102.5mm thick brickwork	2.00	
Finishes	0.50	
Total	4.00	



1. This drawing is to be read in conjunction with all relevant Architects & Engineers 1. Dry pack grout underpinning to be to the following drawings and specifications 2. Do not scale from this drawing a. Mix to be 3:1 of sharp sand o.p.c. General Propping Notes b. Sand to be sharpest that can be handled c. Water content to be just enough to allow balling when 1. The contractor will be responsible for all temporary works and will be responsible for the stability of entire structure during the works 2. Excavation for new foundations to be carried out in maximum 1.0m lengths with a minimum clear space of 2. Our plans and section show a possible solution for 3.6m between open excavations at any one time the temporary works 3. "Over-dig" for access purpose is only permitted in the 3. All party wall notices are to be agreed direction away from the existing building. additional excavation of any kind which may undermine the 4. Install needles and props above the locations of the existing foundations is strictly forbidden during proposed beams Ensure all props are laterally restrained and install 4. Underside of existing foundations to be underpinned all necessary bracing shall be thoroughly cleaned to expose sound 6. Install any channel required across the head of the needles 5. New foundations to be concreted/bricked up to within 75mm of underside of the existing foundation (50mm If required ensure all temporary works are dry packed into place 6. After a minimum curing period of 24 hours, the 75mm 8. Ensure no heavy materials are stored on the floors gap is to be pinned by hand or machine rammed of dry that are being propped 9. Break out the load bearing walls in question 7. Subsequent excavations cannot commence for a minimum of 48 hours after pinning 10. Install new substructures 8. Contractors method statement and programme is to 11. Install new steel frames and ensure that the are be issued and approved prior to works commencing on fully dry-packed into place in order to take the permanent loads 9. Underpins to be connected with 4No. H16 bars 12. Once dry pack is set and approved by the building control officer strike the temporary props Make good all areas of brickwork where needles where inserted New 250mm Reinforced Concrete wall New 250mm reinforced concrete wall P1 00.08.21 JNS XX Preliminary Issue Rev Date Drwn Chkd Amendments Drawing Status **PRELIMINARY** New 250mm reinforced concrete wall В ____ STRUCTURAL / CIVIL ENGINEERS T 020 8340 4041 / E INFO@SYMMETRYS.COM / SYMMETRYS.COM / LONDON N8 8SL Job Title 43A REDINGTON ROAD New 330mm reinforced LONDON, NW3 concrete wall Drawing Title TEMPORARY WORKS AND UNDERPINNING LAYOUT Project Company Zones Level Type Role Number 21141 - SYM - XX - XX - DR - S - 0080 Revision Scale: 1:50 AT A1 Drawn By: JNS

Notes

ompany No. 5873122 VAT Registration No. 894 2993 61 Registered In England And Wales

P1

Checked By: SB

Date: AUG 2021

Refer to Architects drawings for all setting out details

The contractor shall be responsible for the design, installation and sequencing of all temporary works and must ensure that stability of the structure is not compromised during the works

Legend

	Denotes existing masonry or timber structure
	Denotes new masonry walls built in 15N/mm² compressive strength brickwork and grade iii mortar
<i><!--/////</i--></i>	Denotes new masonry walls built in 7.3N/mm² compressive strength blockwork and grade iii mortar
	Denotes new non load bearing stud wall by Architect, unless noted otherwise
<i></i>	Denotes new floor joists 200x50mm C24 timber at 300mm centres
2 ⁵³	Denotes 660 long x 330 wide x 215mm high mass concrete C30 padstone unless noted otherwise
25° mm	Denotes 330 long x 215 wide x 215mm high mass concrete C30 padstone unless noted otherwise
:	Denotes joists doubled up and bolted together using M12 bolts at 400mm

Contractor/Specialist design element

1. All temporary works

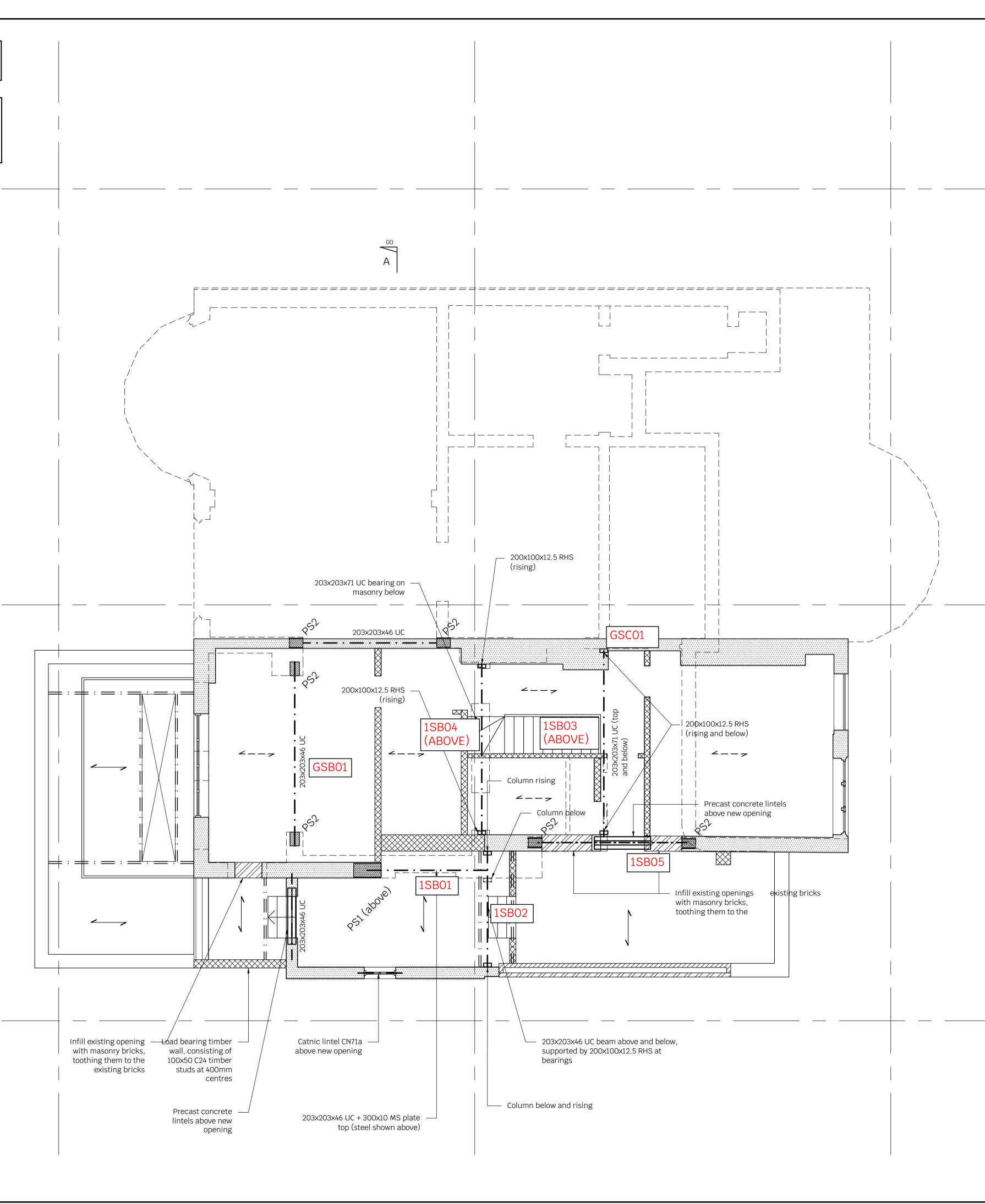
centres

∠ — — → Denotes existing timber floor joists

- 2. All tanking details
- 3. All reinforcement drawings and bar bending schedules
- 4. Design of all steelwork connections. The fabricator will have to submit their calculations to building control for approval
- 5. Steel fabrication drawings
- 6. Design of all staircases, balustrades, glazing and secondary steel

Notes

- All steelwork in the external walls is to be galvanised (125 microns)
- All steelwork to be encased in concrete is to be un-painted
- Location of existing and proposed drain runs are to be confirmed by the Service Engineer
- Please refer to Architects drawings for all setting out details, insulation and ventilation details, damp proof courses and all tanking details
- For all fire work protection to steelwork refer to the Architects drawings
- Contractor should also review Architect's drawings for exact location of service penetration and confirm these with the Structural Engineer prior to cutting



	Notes
	1. This drawing is to be read in conjunction with all relevant Architects & Engineers
	drawings and specifications2. Do not scale from this drawing
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ET WALL D: Carity Wall, b= 135 Carity Wall, b= 1.35 Carity Wall, b= 1.35 Carity Wall, b= 1.35	Job Ne. ZI 14,1 Date C DMAN 20.8.2 04 33,3 1,8 1,8 1,4 × 2 33,9 DL 5,5 4,4	1/2 Made By 58 L L C C S S C C C C C C C C C C C C C C	Revision Checked By BS	DETRICAL LOA RET. WALL BAD THE AS A REAL SAND STRIPT VERTICAL LOA RET. WALL BAD MALES RET. WALL BAD WALL GROUNDA RET. WALL GROUNDA RET. WALL STRIP FOUNDA 225. WASPARS	TRYS ILBRINKER WALLS LOAD THAF DOWN WALLS LOAD THAF THAF DOWN WALLS LOAD THAF THAF THAF THAF THAF THAF THAF THAF	Job No. 2(141 Date 20.8 21 0 0 0 0 0 0 0 1 1 1 0 1 1 1 0 1 1 0 1 1 0 1 1 1 1 0 1 1 0 1 1 1 1 1 1 1 1 1 1 1 1 1	2/2 Made By S &	Revis Checki DS CLC
STREE AS REDINGTON ROAD LONG AND STREP FOUNDATIONS RET WALL A: 2DS MM WASENY, h= 7.4 m Reg (spen 1.4 m) read floar (Job Ne. 2/14/ 2/14/ Date 70000 20.8.21 0L 33,3 1/8 1/4 × 2 33,9 33,9 0L 5,5 4,4	1/2 Made By 58 4 4 4 7 8 8 8 9 8 9 8 9 8 9 9 8 9 9 9 9 9 9 9	Revision Checked By BS	DETRICAL LOA RET. WALL BAD THE AS A REAL SAND STRIPT VERTICAL LOA RET. WALL BAD MALES RET. WALL BAD WALL GROUNDA RET. WALL GROUNDA RET. WALL STRIP FOUNDA 225. WASPARS	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Jeb No. 2(14) Date 20.8 2) 04 04,2 1,6 x 2,1 1/,7 04,2 1,2 4,2 04 04 04 04 04 04 04 04 04 04	2/2 Made By S &	Revis Check DS CLC - - - - - - - - - - - - - - - - - -
STREE BY BRUTTERS BRUTTERAL (OWNER ENDERERS) STREE & S. REDINGTON ROAD LONG CONCRETENTING WALLS LOAD TAKE AND STREP FOUNDATIONS I FATLAS (DADS RET WALL A: 2DS MM WASENY, h: 7.4 M Reg (seen 1.4 m) FEAST floar (span 1.4 m) FEAST floar (span 1.4 m) Castly well, h: 1.85 m Resent (seen 1.4 m) FEAST floar (span 1.4 m) Castly well, h: 1.85 m Resent (seen 1.4 m) Excess Resent (seen 1.4 m)	Job Ne. Zi (4 i Date) Zi (4 i Date) Zo - B · Zi GL 33, 3 I, 8 I, 8 I, 8 I, 8 X, 4 X, 4 IQ, 9	1/2 Made By 58 4 4 7 8 8 8 7 8 7 5 2,5	Revision Checked By BS	Detrite as Red Section RET ALMAN AND STRIPT VERTICAL COA RET. WALL She Makeon 2411 GROOT RET. WALL GROUND RET. WALL STRIP FOUNDA 225 WASPERS 241-5 Floor	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Job No. 2(14) Date 20.8 2) 04. 64.2 1,6 x 2,1 11.7 0.L 4,2 4,2 64,2 4,2 64,2 1,6 x 2,1 14.7 0.L 4,2 0.L 64,2 1,2 0.L 1,2 1,2 1,2 1,2 1,2 1,2 1,2 1,2	2/2 Made By S &	Revis Check DS CL CL CL CL CL CL CL CL CL CL CL CL CL
ET WALL 2: Carry Wall, hartsson Rate of the control of the contr	Job Ne. Zi (4 i Date Zi (4 i Date Zo - B - Zi GL 33, 3 1, 8 4, 4 × 2 37, 9 DL 5, 5 4, 4 10, 9	1/2 Made By 58 4 4 4 7 8 8 8 9 8 9 8 9 8 9 9 8 9 9 9 9 9 9 9	Revision Checked By BS	Detrite as Red Section RET ALMAN AND STRIPT VERTICAL COA RET. WALL She Makeon 2411 GROOT RET. WALL GROUND RET. WALL STRIP FOUNDA 225 WASPERS 241-5 Floor	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Jeb No. 2(141 Date 20.8 21 0 0 0 0 0 0 0 1 1 1 0 1 1 1 0 1 1 2 1 0 1 1 0 1 0 1 0 1 0 1 0 1 0 1 1 0 1 1 1 1 1 1 1 1 1 1 1 1 1	2/2 Made By S &	Revis Checki DS CLC
Total Rest Carty Wall, L. = 4,52,4	Job No. 2/14/ 2000 2008 2 308 2 54 54 54 33,3 1/8 1/8 1/8 2 33,3 1/8 1/8 2 33,3 1/8 55 55 55 55 5,5 4,4 4 10,9 0,4 10,5	1/2 Made By 58 4 4 58 4 4 58 4 4 58 4 4 58 4 4 58 4 4 58 58 58 58 58 58 58 58 58 58	Revision Checked By BS	Detrite as Red Section RET ALMAN AND STRIPT VERTICAL COA RET. WALL She Makeon 2411 GROOT RET. WALL GROUND RET. WALL STRIP FOUNDA 225 WASPERS 241-5 Floor	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Job No. 2(14) Date 20.8 2) 04. 64.2 1,6 x 2,1 11.7 0.L 4,2 4,2 64,2 4,2 64,2 1,6 x 2,1 14.7 0.L 4,2 0.L 64,2 1,2 0.L 1,2 1,2 1,2 1,2 1,2 1,2 1,2 1,2	2/2 Made By S &	Revis Check DS CL CL CL CL CL CL CL CL CL CL CL CL CL
ET WALL 2: Carty Wall, h = 1,83 m Respect (carty), h = 7.4 m Respect (carty), h = 7	Job NN. ZI (4 DDA) Date DOM 20.8.2 QL 33.3 1.8 1.4 × 2 33.9 .1.8 .1.4 × 2 .33.9 .1.8 .1.4 × 2 .33.9 .1.8 .1.4 × 2 .33.4 .1.8 .1.4 × 2 .34.9 	1/2 Made By 58 L C C C S S S S S S S S S S S S S S S S	Revision Checked By BS	Detrite as Red Section RET ALMAN AND STRIPT VERTICAL COA RET. WALL She Makeon 2411 GROOT RET. WALL GROUND RET. WALL STRIP FOUNDA 225 WASPERS 241-5 Floor	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Job No. 2(14) Date 20.8 2) 04. 64.2 1,6 x 2,1 11.7 0.L 4,2 4,2 64,2 4,2 64,2 1,6 x 2,1 14.7 0.L 4,2 0.L 64,2 1,2 0.L 1,2 1,2 1,2 1,2 1,2 1,2 1,2 1,2	2/2 Made By S &	Revis Check DS CL CL CL CL CL CL CL CL CL CL CL CL CL
EXAMPLERYS ERUTURAL/OWN LIGHTERS INTERASE REDINGTON COND LONG INTERSON WALLS LOAD TAKE AND STR.P POUNDATIONS ENTRACE COADS ENTRACE COAD	Job No. 2/14/ 2000 2008 2 308 2 54 54 54 33,3 1/8 1/8 1/8 2 33,3 1/8 1/8 2 33,3 1/8 55 55 55 55 5,5 4,4 4 10,9 0,4 10,5	1/2 Made By 58 4 4 58 4 4 58 4 4 58 4 4 58 4 4 58 4 4 58 58 58 58 58 58 58 58 58 58	Revision Checked By BS	Detrite as Red Section RET ALMAN AND STRIPT VERTICAL COA RET. WALL She Makeon 2411 GROOT RET. WALL GROUND RET. WALL STRIP FOUNDA 225 WASPERS 241-5 Floor	TRYS WATCON EGAD, LONDON WALLS LOAD TRAE DON WALLS LOAD TRAE DON TOTAL WALLS LOAD TRAE DON TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL TOTAL	Job No. 2(14) Date 20.8 2) 04. 64.2 1,6 x 2,1 11.7 0.L 4,2 4,2 64,2 4,2 64,2 1,6 x 2,1 14.7 0.L 4,2 0.L 64,2 1,2 0.L 1,2 1,2 1,2 1,2 1,2 1,2 1,2 1,2	2/2 Made By S &	Revis Check DS CL CL CL CL CL CL CL CL CL CL CL CL CL

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STRUCTURAL / CIVIL ENGINEERS	Job No.	Sheet No.	Revision	SYMMETRYS	Job No.	Sheet.No.	Revisio
Title A2	21141			STRUCTURAL / COVIL ENDINEERS	21141		
Title A32 REDINGTON ROAD, LONDON	Date	Made By	Checked By	JOD TITLE 43 & REBINGTON ROAD . LONDON	Date	Made By	Checko
FROM DE AL REDINGTON READ	20 8-24	SB	D S	Section PADSTONE PI	20.8.21	SB	Þ
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2+1+4 Floor:				TRJ 660 www x 330 mm + 1	15 40 40	Deer	Conta
1 KW/m - 3 m		3 * .	3	PAD STOME			Constant of the
(15+1,2) Ky/u2 . 3 w		1	8,1 × 3				
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HE THEORY OF ELASTICITY		V.		8 4 3 5 1 0, 63 V 10 2			
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r = 0, 32 m ($x = 1m$)			~ /3×				
s m < q.4							
0 203 Q h		N.					
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× 1003+ ~ 26.3 KN/ m 2 (for n=0.2)							
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EN CONSIDERED							

8	SYMMETRYS	Project 43a RE	DINGTON ROA	Job no. 21141			
	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for B-RW01				Start page no./Revision 1 B	
Unit 6 Th	Unit 6 The Courtyard, Lynton Road London N8 8SL		Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Cantilever
Stem height	h _{stem} = 2500 mm
Stem thickness	t _{stem} = 225 mm
Angle to rear face of stem	α = 90 deg
Stem density	γ_{stem} = 25 kN/m ³
Toe length	I _{toe} = 2200 mm
Base thickness	t _{base} = 250 mm
Base density	γ_{base} = 25 kN/m ³
Height of retained soil	h _{ret} = 2500 mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 1500 mm
Water density	γ _w = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm clay
Moist density	γ _{mr} = 19 kN/m ³
Saturated density	γ _{sr} = 19 kN/m ³
Characteristic effective shear resistance angle	∮' r.k = 27 deg
Characteristic wall friction angle	$\delta_{r.k}$ = 13.5 deg
Base soil properties	
Soil type	Firm clay
Soil density	γ _b = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{b.k} = 27 deg
Characteristic wall friction angle	δ _{b.k} = 13.5 deg
Characteristic base friction angle	δ _{bb.k} = 18 deg
Presumed bearing capacity	P _{bearing} = 60 kN/m ²
Loading details	
Variable surcharge load	Surcharge _Q = 10 kN/m ²
Vertical line load at 2312 mm	P _{G1} = 37.9 kN/m
	P _{Q1} = 8.4 kN/m

	Project 43a	REDINGTON ROA	AD, LONDON N	W3 7RA	Job no. 2	1141
Structural / civil engineers Symmetrys	Calcs for	B-R	RW01		Start page no./Revision 2 B	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved da 14/01/20
	L.	2022	. Lease L			
	H	22002312	→ 225			
	Prop 8 kN/m ²	2425	B8 kN/m ²	9.6 kN/m ²	0927 J/m²	
Calculate retaining wall geon Base length	netry	I _{base} = I _{toe} +	t _{stem} = 2425 mm			
Base length Saturated soil height	netry	$h_{sat} = h_{water}$	+ d _{cover} = 1500 n	nm		
Base length Saturated soil height Moist soil height	netry	h _{sat} = h _{water} h _{moist} = h _{ret} .	+ d _{cover} = 1500 n - h _{water} = 1000 m	nm		
Base length Saturated soil height Moist soil height Length of surcharge load	-	h _{sat} = h _{water} h _{moist} = h _{ret} · I _{sur} = I _{heel} = 1	+ d _{cover} = 1500 n - h _{water} = 1000 m 0 mm	nm m		
Base length Saturated soil height Moist soil height	-	$\begin{aligned} h_{sat} &= h_{water} \\ h_{moist} &= h_{ret} \\ I_{sur} &= I_{heel} \\ x_{sur_v} &= I_{base} \end{aligned}$	+ d _{cover} = 1500 n - h _{water} = 1000 m	nm m mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo	ent	$\begin{array}{l} h_{sat} = h_{water} \\ h_{moist} = h_{ret} \\ l_{sur} = l_{heel} = 0 \\ x_{sur_v} = l_{base} \\ h_{eff} = h_{base} \\ x_{sur_h} = h_{eff} \end{array}$	+ d _{cover} = 1500 m - h _{water} = 1000 m 0 mm - I _{heel} / 2 = 2425 + d _{cover} + h _{ret} = 27 / 2 = 1375 mm	nm m mm 750 mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo Area of wall stem	ent	$\begin{array}{l} h_{sat} = h_{water} \\ h_{moist} = h_{ret} \\ I_{sur} = I_{heel} = 0 \\ x_{sur_v} = I_{base} \\ h_{eff} = h_{base} \\ x_{sur_h} = h_{eff} \\ A_{stem} = h_{stem} \end{array}$	+ d _{cover} = 1500 m - h _{water} = 1000 m 0 mm - l _{heel} / 2 = 2425 + d _{cover} + h _{ret} = 27 / 2 = 1375 mm _n × t _{stem} = 0.563 m	nm m mm 750 mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone	ent	$\begin{array}{l} h_{sat} = h_{water} \\ h_{moist} = h_{ret} \\ I_{sur} = I_{heel} = 0 \\ x_{sur_v} = I_{base} \\ h_{eff} = h_{base} \\ x_{sur_h} = h_{eff} \\ A_{stem} = h_{stem} \\ x_{stem} = I_{toe} + 0 \end{array}$	+ d _{cover} = 1500 m - h _{water} = 1000 m 0 mm - l _{heel} / 2 = 2425 + d _{cover} + h _{ret} = 27 / 2 = 1375 mm n × t _{stem} = 0.563 m · t _{stem} / 2 = 2313	nm m 750 mm m ² mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base	ent onent ent	$\begin{array}{l} h_{sat} = h_{water} \\ h_{moist} = h_{ret} \\ I_{sur} = I_{heel} = 0 \\ x_{sur_v} = I_{base} \\ h_{eff} = h_{base} \\ x_{sur_h} = h_{eff} \\ A_{stem} = h_{stem} \\ x_{stem} = I_{toe} \\ + \\ A_{base} = I_{base} \end{array}$	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm m × $t_{stem} = 0.563 \text{ m}$ · $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$	nm m 750 mm m ² mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone	ent onent ent	$\begin{array}{l} h_{sat} = h_{water} \\ h_{moist} = h_{ret} \\ I_{sur} = I_{heel} = 0 \\ x_{sur_v} = I_{base} \\ h_{eff} = h_{base} \\ x_{sur_h} = h_{eff} \\ A_{stem} = h_{stem} \\ x_{stem} = I_{toe} \\ + \\ A_{base} = I_{base} \end{array}$	+ d _{cover} = 1500 m - h _{water} = 1000 m 0 mm - l _{heel} / 2 = 2425 + d _{cover} + h _{ret} = 27 / 2 = 1375 mm n × t _{stem} = 0.563 m · t _{stem} / 2 = 2313	nm m 750 mm m ² mm		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base	ent onent ent	$ h_{sat} = h_{water} h_{moist} = h_{ret} - 1 I_{sur} = I_{heel} = 0 X_{sur_v} = I_{base} - 1 A_{sur_h} = h_{eff} / 1 A_{stem} = h_{stem} X_{stem} = I_{toe} + 1 A_{base} = I_{base} \\ X_{base} = I_{base} $	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm m × $t_{stem} = 0.563 \text{ m}$ · $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$	nm m 750 mm m ² mm 1 ²	1 + √[sin(∳'r.k +	+ δ _{r.k}) × sin(α
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory	ent onent ent	$h_{sat} = h_{water}$ $h_{moist} = h_{ret}$ $l_{sur} = l_{heel} = 0$ $x_{sur_v} = l_{base}$ $h_{eff} = h_{base}$ $x_{sur_h} = h_{eff}$ $A_{stem} = h_{stem}$ $x_{stem} = l_{toe} + $ $A_{base} = l_{base}$ $x_{base} = l_{base}$ $K_A = sin(\alpha + 1)$	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm m × $t_{stem} = 0.563 \text{ m}$ + $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$ / 2 = 1213 mm	nm m 750 mm m ² mm n ² × sin(α - δ _{r.k}) × [1 + √[sin(φ'r.k +	- δ _{r.k}) × sin(α
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory	ent onent ent	$\begin{aligned} h_{sat} &= h_{water} \\ h_{moist} &= h_{ret} \\ l_{sur} &= l_{heel} = 0 \\ x_{sur_v} &= l_{base} \\ h_{eff} &= h_{base} \\ x_{sur_h} &= h_{eff} \\ A_{stem} &= h_{sten} \\ x_{stem} &= l_{toe} + 0 \\ A_{base} &= l_{base} \\ x_{base} &= $	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $I_{heel} / 2 = 24255$ + $d_{cover} + h_{ret} = 2775 \text{ m}$ m × $t_{stem} = 0.563 \text{ m}$ + $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$ / 2 = 1213 mm + $\varphi'_{r,k}$ ² / $(sin(\alpha)^2)$	nm m 750 mm m ² mm n ² × sin(α - δ _{r.k}) × [β))]] ²) = 0.340		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient	ent onent ent	$\begin{aligned} h_{sat} &= h_{water} \\ h_{moist} &= h_{ret} \\ l_{sur} &= l_{heel} = 0 \\ x_{sur_v} &= l_{base} \\ h_{eff} &= h_{base} \\ x_{sur_h} &= h_{eff} \\ A_{stem} &= h_{sten} \\ x_{stem} &= l_{toe} + 0 \\ A_{base} &= l_{base} \\ x_{base} &= $	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm n × $t_{stem} = 0.563 \text{ m}$ + $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$ / 2 = 1213 mm + $\phi'_{r,k}$ /2 / $(sin(\alpha)^2$ - $\delta_{r,k}) \times sin(\alpha + \alpha)$	nm m 750 mm m ² mm n ² × sin(α - δ _{r.k}) × [β))]] ²) = 0.340		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient	ent onent ent	$\begin{aligned} h_{sat} &= h_{water} \\ h_{moist} &= h_{ret} \\ l_{sur} &= l_{heel} = 0 \\ x_{sur_v} &= l_{base} \\ h_{eff} &= h_{base} \\ x_{sur_h} &= h_{eff} \\ A_{stem} &= h_{sten} \\ x_{stem} &= l_{toe} + 0 \\ A_{base} &= l_{base} \\ x_{base} &= $	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm n × $t_{stem} = 0.563 \text{ m}$ + $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$ / 2 = 1213 mm + $\phi'_{r,k}$ /2 / $(sin(\alpha)^2$ - $\delta_{r,k}) \times sin(\alpha + \alpha)$	nm m 750 mm m ² mm n ² × sin(α - δ _{r.k}) × [β))]] ²) = 0.340		
Base length Saturated soil height Moist soil height Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient Passive pressure coefficient Bearing pressure check	ent onent ent	$h_{sat} = h_{water}$ $h_{moist} = h_{ret}$ $l_{sur} = l_{heel} = 0$ $x_{sur_v} = l_{base}$ $h_{eff} = h_{base}$ $k_{stem} = h_{stem}$ $x_{stem} = l_{toe} + 0$ $A_{base} = l_{base}$ $x_{base} = l_{base}$ $K_A = sin(\alpha + 0) / (sin(\alpha + 0))$ $(sin(90 + \delta_t))$	+ $d_{cover} = 1500 \text{ m}$ - $h_{water} = 1000 \text{ m}$ 0 mm - $l_{heel} / 2 = 2425$ + $d_{cover} + h_{ret} = 27$ / 2 = 1375 mm n × $t_{stem} = 0.563 \text{ m}$ + $t_{stem} / 2 = 2313$ × $t_{base} = 0.606 \text{ m}$ / 2 = 1213 mm + $\phi'_{r,k}$ /2 / $(sin(\alpha)^2$ - $\delta_{r,k}) \times sin(\alpha + \alpha)$	nm m 750 mm n^2 mm n^2 × sin(α - δ _{r.k}) × [β))]] ²) = 0.340 + δ _{b.k}) × [1 - \sqrt{s}		

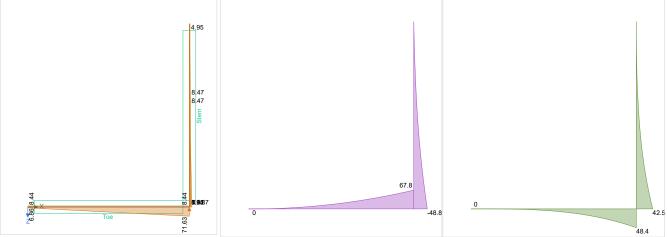
STRUCTURAL / CIVIL ENGINEERS Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL Line loads Total	Calcs for Calcs by SB	B-R Calcs date 14/01/2022	W01 Checked by DS	Checked date			
London N8 8SL			-	Checked date		Start page no./Revision 3 B	
			03	14/01/2022	Approved by DS	Approved date 14/01/2022	
Total		$F_{P_v} = P_{G1} +$	• P _{Q1} = 46.3 kN	J/m			
		$F_{total_v} = F_{ster}$	m + F _{base} + F _{P_}	v + F _{water_v} = 75.5 k	kN/m		
Horizontal forces on wall							
Surcharge load		$F_{sur_h} = K_A \times$	$(\cos(\delta_{r.k}) \times Su)$	rcharge _Q × h _{eff} = 9	0 .1 kN/m		
Saturated retained soil		$F_{sat_h} = K_A \times$	$c\cos(\delta_{r.k}) \times (\gamma_{sr})$	- γ_w) × (h _{sat} + h _{base}	_e)² / 2 = 4.6 kN	l/m	
Water		$F_{water_h} = \gamma_w$	× (h _{water} + d _{cove}	er + h _{base}) ² / 2 = 15	kN/m		
Moist retained soil		F _{moist_h} = K _A	$\times \cos(\delta_{r.k}) \times \gamma_r$	$_{ m nr} imes$ ((h _{eff} - h _{sat} - h _{ba}	$(h_{eff})^2 / 2 + (h_{eff})^2$	- h _{sat} - h _{base}) ×	
		(h _{sat} + h _{base})) = 14.1 kN/m				
Base soil		F _{pass_h} = -K _P	$ \times \cos(\delta_{b,k}) \times \gamma $	$v_b \times (d_{cover} + h_{base})^2$	/ 2 = -2.3 kN/	m	
Total		$F_{total_h} = F_{sur}$	_h + F _{sat_h} + F _w	_{ater_h} + F _{moist_h} + F _p	_{ass_h} = 40.5 kN	l/m	
Moments on wall							
Wall stem		M _{stem} = F _{sten}	n × X _{stem} = 32.5	kNm/m			
Wall base		$M_{base} = F_{base}$	$x \times x_{base} = 18.4$	kNm/m			
Surcharge load		$M_{sur} = -F_{sur_l}$	$h \times X_{sur_h} = -12.$. 5 kNm/m			
Line loads		$M_{P} = (P_{G1} +$	P _{Q1}) × p ₁ = 10	07 kNm/m			
Saturated retained soil		M _{sat} = -F _{sat_t}	h × X _{sat_h} = -2.7	kNm/m			
Water		$M_{water} = -F_{water}$	$_{ater_h} \times \mathbf{x}_{water_h} =$	-8.8 kNm/m			
Moist retained soil		$M_{moist} = -F_{mo}$	 pist_h × X _{moist_h} =	-16.1 kNm/m			
Total		M _{total} = M _{sten}	n + M _{base} + M _{su}	$_{r}$ + M _P + M _{sat} + M _w	_{vater} + M _{moist} = [,]	117.8 kNm/m	
Check bearing pressure							
Propping force		F _{prop_base} = F	- _{total_h} = 40.5 kl	N/m			
Distance to reaction		$\overline{\mathbf{x}} = \mathbf{M}_{\text{total}}$ /	F _{total_v} = 1560 ı	mm			
Eccentricity of reaction		$e = \overline{x} - I_{base}$, / 2 = 348 mm				
Loaded length of base		$I_{load} = I_{base} =$	2425 mm				
Bearing pressure at toe		$q_{toe} = F_{total_v}$	/ $I_{\text{base}} \times (1 - 6)$	× e / I _{base}) = 4.3 kN	l/m²		
Bearing pressure at heel		$q_{heel} = F_{total}$	$_v$ / I _{base} × (1 + 6	$6 \times e / I_{base}$) = 58 kl	N/m²		
Factor of safety		FoS _{bp} = P _{be}	_{aring} / max(q _{toe} ,	q _{heel}) = 1.035			

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

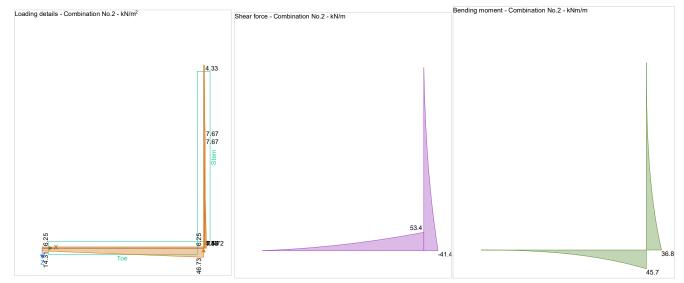
Tedds calculation version 2.9.12

Concrete strength class	C30/37
Characteristic compressive cylinder strength	f _{ck} = 30 N/mm ²
Characteristic compressive cube strength	f _{ck,cube} = 37 N/mm ²
Mean value of compressive cylinder strength	f _{cm} = f _{ck} + 8 N/mm ² = 38 N/mm ²
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm ² × (f_{ck} / 1 N/mm ²) ^{2/3} = 2.9 N/mm ²
5% fractile of axial tensile strength	$f_{ctk,0.05}$ = 0.7 × f_{ctm} = 2.0 N/mm ²
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm ² × (f _{cm} / 10 N/mm ²) ^{0.3} = 32837 N/mm ²
Partial factor for concrete - Table 2.1N	γ _C = 1.50
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15	f_{cd} = $\alpha_{cc} \times f_{ck}$ / γ_C = 17.0 N/mm ²
Maximum aggregate size	h _{agg} = 20 mm

SYMMETRYS	Project 43a RI	EDINGTON ROA	D, LONDON N	Job no. 21141			
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision		
Symmetrys		B-RW01			4 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022	
Ultimate strain - Table 3.1		ε _{cu2} = 0.003	5				
Shortening strain - Table 3.1		ε _{cu3} = 0.003	5				
Effective compression zone heig	ght factor	$\lambda = 0.80$					
Effective strength factor		η = 1.00					
Bending coefficient k1		K ₁ = 0.40					
Bending coefficient k ₂		$K_2 = 1.00 \times (0.6 + 0.0014 / \epsilon_{cu2}) = 1.00$					
Bending coefficient k ₃		K ₃ = 0.40					
Bending coefficient k4		$K_4 = 1.00 \times (0.6 + 0.0014 / \epsilon_{cu2}) = 1.00$					
Reinforcement details							
Characteristic yield strength of r	einforcement	f _{yk} = 500 N/	mm²				
Modulus of elasticity of reinforce	ement	E _s = 20000	0 N/mm ²				
Partial factor for reinforcing stee	el - Table 2.1N	γs = 1.15					
Design yield strength of reinforc	ement	$f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$					
Cover to reinforcement							
Front face of stem		c _{sf} = 40 mm	1				
Rear face of stem		c _{sr} = 50 mm	ı				
Top face of base		c _{bt} = 50 mm	ı				
Bottom face of base		c _{bb} = 75 mn	n				
Loading details - Combination No.1 - kN/m ²	Shear force - Co	ombination No.1 - kN/m		Bending moment - Combin	ation No.1 - KNm/m		



STRUCTURAL / CIVIL ENGINEERS Calcs for Start page no./Revision Symmetrys B-RW01 5 B		Project 43a F		Job no. 21141			
Symmetrys B-RW01 5 B							
Linit 6 The Countrard Lymbon Dood	Symmetrys						
Lander No.001	Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022



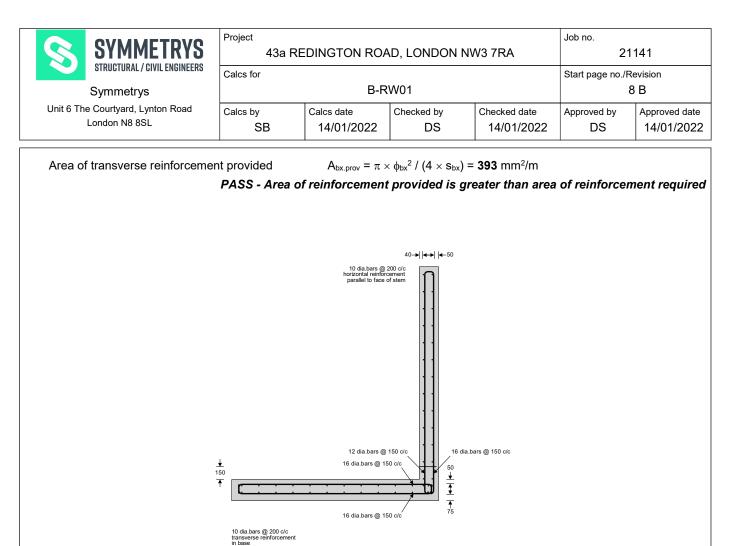
Check stem design at base of stem	
Depth of section	h = 225 mm
Rectangular section in flexure - Section 6.1	
Design bending moment combination 1	M = 42.5 kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2 = 167 \text{ mm}$
	$K = M / (d^2 \times f_{ck}) = 0.051$
	$K' = (2 \times \eta \times \alpha_{\mathrm{cc}}/\gamma_{C}) \times (1 - \lambda \times (\delta - K_1)/(2 \times K_2)) \times (\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 + 0.5 × (1 - 2 × K / ($\eta \times \alpha_{cc}$ / γ_{C})) ^{0.5} , 0.95) × d = 159 mm
Depth of neutral axis	x = 2.5 × (d – z) = 21 mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) = 616 \text{ mm}^2/\text{m}$
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1340 \text{ mm}^2/\text{m}$
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 252 \text{ mm}^2/\text{m}$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h = 9000 \text{ mm}^2/\text{m}$
	$max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.46$
PASS - Area of I	reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4	
Reference reinforcement ratio	ρ₀ = √(f _{ck} / 1 N/mm²) / 1000 = 0.005
Required tension reinforcement ratio	$\rho = A_{sr.req} / d = 0.004$
Required compression reinforcement ratio	ρ' = A _{sr.2.req} / d ₂ = 0.000
Structural system factor - Table 7.4N	K _b = 0.4
Reinforcement factor - exp.7.17	$K_s = min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.prov}), 1.5) = 1.5$
Limiting span to depth ratio - exp.7.16.a	$min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} \ / \ 1 \ N/mm^2)} \times \rho_0 \ / \ \rho + 3.2 \times \sqrt{(f_{ck} \ / \ 1 \ N/mm^2)})$
	N/mm ²) × (ρ_0 / ρ - 1) ^{3/2}], 40 × K _b) = 16
Actual span to depth ratio	h _{stem} / d = 15
	PASS - Span to depth ratio is less than deflection control limit

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STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision				
Symmetrys		B-F	RW01	6 B					
Unit 6 The Courtyard, Lynton Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date			
London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/2022			
Crack control - Section 7.3									
Limiting crack width		w _{max} = 0.3	mm						
Variable load factor - EN1990 -	Table A1.1	ψ2 = 0.6							
Serviceability bending moment		M _{sls} = 26.2							
Tensile stress in reinforcement			$A_{sr.prov} \times z) = 12$	23.3 N/mm ²					
Load duration		Long term							
Load duration factor		k _t = 0.4							
Effective area of concrete in tens	sion		, .	h - x) / 3, h / 2)					
		A _{c.eff} = 680							
Mean value of concrete tensile s	trength		2.9 N/mm ²	•					
Reinforcement ratio			_{ov} / A _{c.eff} = 0.02	U					
Modular ratio	$\alpha_{\rm e} = E_{\rm s} / E_{\rm c}$	_{cm} = 6.091							
Bond property coefficient	k ₁ = 0.8								
Strain distribution coefficient	k ₂ = 0.5								
		k₃ = 3.4 k₄ = 0.425							
Maximum crack spacing - exp.7.	11	κ₄ = 0.425 s _{r.max} = k ₃ × c _{sr} + k ₁ × k ₂ × k ₄ × φ _{sr} / ρ _{p.eff} = 308 mm							
Maximum crack width - exp.7.8				$(f_{ct.eff} / \rho_{p.eff}) \times (1 +$		6 × ~) / E			
	w _k = 3 _{r.max} / w _k = 0.114		(Ict.eff / pp.eff) ~ (I	tte × pp.ett), U.	$0 \times 0_{s} / L_{s}$				
	$W_k - 0.114$ $W_k / W_{max} =$								
				rack width is les	s than limitin	q crack width			
Rectangular section in shear -	Section 6 2					•			
Design shear force	Section 0.2	V = 48.8 ki	N/m						
Boolgh onour loroo			3 / γ _C = 0.120						
			- √(200 mm / d	(2) = 2.000					
Longitudinal reinforcement ratio			r.prov / d, 0.02) =						
	Longitudinal reinforcement ratio		. ,	² × f _{ck} ^{0.5} = 0.542 N	/mm ²				
Design shear resistance - exp.6.	Design shear resistance own 6.25 % 6.2h			0.042 N 10°					
	24 0 0.25	V _{Rd.c} = 115			ck), vmin)∧u				
		$V / V_{Rd.c} = 0.422$							
			PASS - Design shear resistance exceeds design shear force						
Horizontal reinforcement para	llel to face of s		-						
Minimum area of reinforcement	– cl.9.6.3(1)	A _{sx.req} = ma	x(0.25 × A _{sr.prov}	$(, 0.001 \times t_{stem}) = 3$	335 mm²/m				
Maximum spacing of reinforcem	ent – cl.9.6.3(2)	s _{sx_max} = 400 mm							
Transverse reinforcement provid	led	10 dia.bars @ 200 c/c							
Area of transverse reinforcemen	t provided	$A_{sx,prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 393 \text{ mm}^2/\text{m}$							
	PASS - Area o	f reinforcemen	t provided is g	greater than area	of reinforce	ment required			
Check base design at toe									
Depth of section		h = 250 mr	n						
Rectangular section in flexure	- Section 6.1								
Design bending moment combin		M = 48.4 k	Nm/m						
Depth to tension reinforcement		d = h - c _{bb} -	- φ _{bb} / 2 = 167 r	nm					
			× f _{ck}) = 0.058						
				$L \times (\delta - K_1)/(2 \times K_2)$))×(λ × (δ - K ₁)	/(2 × K ₂))			
		K' = 0.207			,				

SYMMETRYS	Project 43a RED	INGTON ROA	D, LONDON NV	V3 7RA	Job no. 2	1141	
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision		
Symmetrys		B-R	W01		etait page ites	7 B	
Unit 6 The Courtyard, Lynton Road	Calcs by C	Calcs date	Checked by	Checked date	Approved by	Approved da	
London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/20	
			K' > K - N	o compression	reinforcem	ent is requir	
Lever arm		z = min(0.5	+ 0.5 × (1 - 2 × 1	$<$ / ($\eta \times \alpha_{cc}$ / γ_{c}))) ^{0.5} , 0.95) × d	= 158 mm	
Depth of neutral axis		x = 2.5 × (d	– z) = 23 mm				
Area of tension reinforcement re	quired	$A_{bb.req} = M /$	$(f_{yd} \times z) = 705 \text{ m}$	ım²/m			
Tension reinforcement provided		16 dia.bars	@ 150 c/c				
Area of tension reinforcement pr	rovided	$A_{bb.prov} = \pi >$	$(4 \times s_{bb})^2 / (4 \times s_{bb}) =$	1340 mm²/m			
Minimum area of reinforcement	- exp.9.1N	A _{bb.min} = ma	$x(0.26 \times f_{ctm} / f_{yk})$	0.0013) × d = 2	2 52 mm²/m		
Maximum area of reinforcement	- cl.9.2.1.1(3)	$A_{bb.max} = 0.0$	04 × h = 10000 m	1m²/m			
		max(A _{bb.req} ,	A _{bb.min}) / A _{bb.prov} =	= 0.526			
	PASS - Area of r	einforcement	provided is gre			-	
				Lib	orary item: Rectai	ngular single ou	
Crack control - Section 7.3							
Limiting crack width	T 11 A4 4	w _{max} = 0.3 r	nm				
Variable load factor - EN1990 -	Table A1.1	$\psi_2 = 0.6$					
Serviceability bending moment Tensile stress in reinforcement		$M_{sis} = 34.6$		$4 \text{ N}/\text{mm}^2$			
Load duration	Long term	$A_{bb.prov} \times z$) = 163	.4 IN/IIIII-				
Load duration factor	k _t = 0.4						
Effective area of concrete in ten		2.5 × (h - d), (h -	x)/3.h/2)				
		A _{c.eff} = 7581	. , .				
Mean value of concrete tensile s	strength	$f_{ct.eff} = f_{ctm} =$					
Reinforcement ratio		$\rho_{p.eff} = A_{bb.pr}$	_{bv} / A _{c.eff} = 0.018				
Modular ratio		$\alpha_{e} = E_{s} / E_{cr}$	n = 6.091				
Bond property coefficient		k ₁ = 0.8					
Strain distribution coefficient		k ₂ = 0.5					
		k ₃ = 3.4					
		k ₄ = 0.425					
Maximum crack spacing - exp.7	.11		$c_{bb} + k_1 \times k_2 \times k_4$				
Maximum crack width - exp.7.8			$max(\sigma_s - k_t \times (f_c$	$_{t.eff}$ / $\rho_{p.eff}$) × (1 +	$\alpha_{e} \times \rho_{p.eff}$), 0.	$.6 \times \sigma_s) / E_s$	
		w _k = 0.2 mn					
		$W_k / W_{max} = 0$).668 - Maximum cra	ck width is los	s than limitir	na crack wi	
Destau milio estita da d	0	PAJJ	- maximulli Uld	on widdi 13 185	5 01011 1111101	ig clack wit	
Rectangular section in shear	Section 6.2	V = 67.8 kN	l/m				
Design shear force			/m / γ _C = 0.120				
				2) = 2 000			
Longitudinal reinforcement ratio		k = min(1 + √(200 mm / d), 2) = 2.000 ρ _I = min(A _{bb.prov} / d, 0.02) = 0.008					
			$N^{1/2}/mm \times k^{3/2} \times k^{3/2}$		/mm ²		
Design shear resistance - exp.6	2a & 6 2h		$(C_{Rd.c} \times k \times (100))$				
- congri on car robiotarioo - oxp.0		$V_{Rd.c} = 115.$			aty , •niiii)∧ u		
		$V / V_{Rd.c} = 0$					
			S - Design shea	ar resistance e	xceeds desig	gn shear fo	
Secondary transverse reinford	cement to base -	Section 9.3					
Minimum area of reinforcement	– cl.9.3.1.1(2)	A _{bx.req} = 0.2	× A _{bb.prov} = 268 r	nm²/m			
Maximum spacing of reinforcem	ent – cl.9.3.1.1(3)	s _{bx_max} = 45	0 mm				



in base

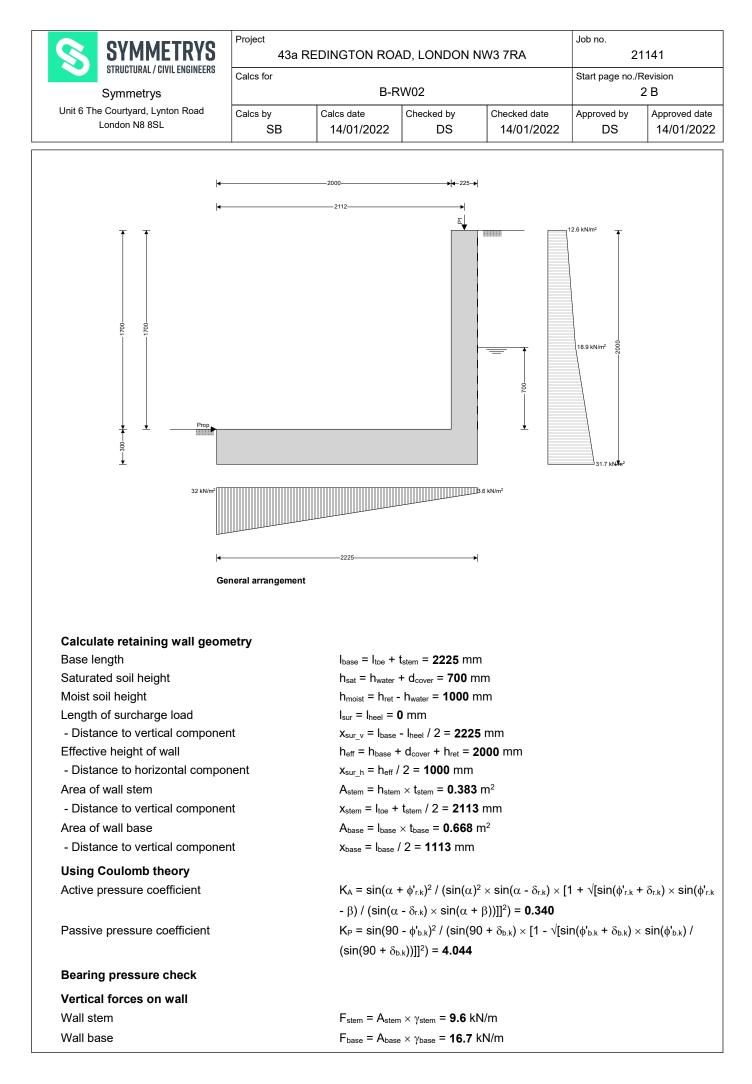
Reinforcement details

8	SYMMETRYS	Project 43a RE	DINGTON ROA	Job no. 21141			
	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for B-RW02				Start page no./Revision 1 B	
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Cantilever
Stem height	h _{stem} = 1700 mm
Stem thickness	t _{stem} = 225 mm
Angle to rear face of stem	α = 90 deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	I _{toe} = 2000 mm
Base thickness	t _{base} = 300 mm
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	h _{ret} = 1700 mm
Angle of soil surface	$\beta = 0 \operatorname{deg}$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 700 mm
Water density	γ _w = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm clay
Moist density	γ _{mr} = 19 kN/m ³
Saturated density	γ _{sr} = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{r.k} = 27 deg
Characteristic wall friction angle	$\delta_{r.k}$ = 13.5 deg
Base soil properties	
Soil type	Firm clay
Soil density	γ _b = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{b.k} = 27 deg
Characteristic wall friction angle	δ _{b.k} = 13.5 deg
Characteristic base friction angle	δ _{bb.k} = 18 deg
Presumed bearing capacity	P _{bearing} = 60 kN/m ²
Loading details	
Permanent surcharge load	Surcharge _G = 26.3 kN/m ²
Variable surcharge load	Surcharge _Q = 11.9 kN/m ²
Vertical line load at 2112 mm	P _{G1} = 10.9 kN/m
	P _{Q1} = 2.5 kN/m



SYMMETRYS	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141			
STRUCTURAL / CIVIL ENGINEERS	Calcs for	B-R	W02			Start page no./Revision 3 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Approved by Approved date DS 14/01/202					
Line loads		F _{P_v} = P _{G1} +	- P _{Q1} = 13.4 ki	N/m				
Total		$F_{total_v} = F_{ster}$	m + F _{base} + F _{P_}	v + F _{water_v} = 39.7	kN/m			
Horizontal forces on wall								
Surcharge load		$F_{sur_h} = K_A$	$\cos(\delta_{r.k}) \times (S)$	urcharge _G + Surch	narge _Q) × h _{eff} =	= 25.2 kN/m		
Saturated retained soil	$F_{sat_h} = K_A \times cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_{w}) \times (h_{sat} + h_{base})^2 / 2 = \textbf{1.5} kN/m$							
Water	$F_{water_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = 4.9 \text{ kN/m}$							
Moist retained soil	$F_{moist_h} = K_{A} \times \cos(\delta_{r,k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{bf})^2 / 2 + (h$							
	(h _{sat} + h _{base})) = 9.4 kN/m							
Base soil	F_{pass_h} = -K _P × cos($\delta_{b,k}$) × γ_b × (d _{cover} + h _{base}) ² / 2 = -3.4 kN/m							
Total	F _{total_h} = F _{sur_h} + F _{sat_h} + F _{water_h} + F _{moist_h} + F _{pass_h} = 37.7 kN/r							
Moments on wall								
Wall stem		M _{stem} = F _{ster}	m × x _{stem} = 20.2	2 kNm/m				
Wall base		$M_{\text{base}} = F_{\text{bas}}$	$_{e} \times x_{base} = 18.6$	រ kNm/m				
Surcharge load		$M_{sur} = -F_{sur}$	$h \times x_{sur_h} = -25$.2 kNm/m				
Line loads		M _P = (P _{G1} +	• P _{Q1}) × p ₁ = 2	8.3 kNm/m				
Saturated retained soil		$M_{sat} = -F_{sat}$	h × X _{sat_h} = -0.5	i kNm/m				
Water		$M_{water} = -F_{w}$	_{ater_h} × X _{water_h} =	- -1.6 kNm/m				
Moist retained soil		$M_{moist} = -F_m$	$_{oist_h} \times \mathbf{X}_{moist_h} =$	- 7.3 kNm/m				
Total		M _{total} = M _{ster}	m + M _{base} + M _s	$_{ur}$ + M _P + M _{sat} + M _v	_{vater} + M _{moist} = 3	32.4 kNm/m		
Check bearing pressure								
Propping force		F _{prop_base} = I	= _{total_h} = 37.7 k	N/m				
Distance to reaction		$\overline{x} = M_{total} /$	F _{total_v} = 817 m	ım				
Eccentricity of reaction		$e = \overline{x} - I_{base}$, / 2 = -296 mr	n				
Loaded length of base		$I_{load} = I_{base} =$	2225 mm					
Bearing pressure at toe		$q_{toe} = F_{total_v}$	/ $I_{base} \times (1 - 6)$	× e / I _{base}) = 32 kN	l/m²			
Bearing pressure at heel		$q_{\text{heel}} = F_{\text{total}}$	$_v$ / I _{base} × (1 + 6	6 × e / I _{base}) = 3.6 k	kN/m²			
Factor of safety		$FoS_{bp} = P_{be}$	_{aring} / max(q _{toe} ,	q _{heel}) = 1.873				

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.12

C30/37
f _{ck} = 30 N/mm ²
f _{ck,cube} = 37 N/mm ²
f _{cm} = f _{ck} + 8 N/mm ² = 38 N/mm ²
f_{ctm} = 0.3 N/mm ² × (f_{ck} / 1 N/mm ²) ^{2/3} = 2.9 N/mm ²
$f_{ctk,0.05}$ = 0.7 × f_{ctm} = 2.0 N/mm ²
E_{cm} = 22 kN/mm ² × (f _{cm} / 10 N/mm ²) ^{0.3} = 32837 N/mm ²
γ _C = 1.50
$\alpha_{cc} = 0.85$
f_{cd} = $\alpha_{cc} \times f_{ck}$ / γ_{C} = 17.0 N/mm ²
h _{agg} = 20 mm

SYMMETRYS	Project 43a F	REDINGTON RO	AD, LONDON	NW3 7RA	Job no.	1141
Structural / Civil Engineers Symmetrys	Calcs for	B-F	RW02	Start page no./Revision 4 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022
Ultimate strain - Table 3.1		ε _{cu2} = 0.003	35			
Shortening strain - Table 3.1		ε _{cu3} = 0.003	35			
Effective compression zone he	ight factor	$\lambda = 0.80$				
Effective strength factor		η = 1.00				
Bending coefficient k ₁		K ₁ = 0.40				
Bending coefficient k ₂		K ₂ = 1.00 ×	(0.6 + 0.0014	/ε _{cu2}) = 1.00		
Bending coefficient k ₃		K ₃ = 0.40				
Bending coefficient k4		K ₄ = 1.00 ×	(0.6 + 0.0014	/ε _{cu2}) = 1.00		
Reinforcement details			·			
Characteristic yield strength of	reinforcement	f _{yk} = 500 N	/mm ²			
Modulus of elasticity of reinforce		$E_{\rm s} = 20000$				
Partial factor for reinforcing ste						
Design yield strength of reinfor		-	= 435 N/mm ²			
	centent	iya — iyk / ys	- 455 N/IIIII			
Cover to reinforcement						
Front face of stem		c _{sf} = 40 mn				
Rear face of stem		c _{sr} = 50 mn				
Top face of base		c _{bt} = 50 mn				
Bottom face of base		c _{bb} = 75 mr	n			
Loading details - Combination No.1 - kN/m ²	Shear force - C	ombination №.1 - kN/m		Bending moment - Combinati		
CL CL SC SC SC SC SC SC SC SC SC SC SC SC SC	8.47 8.47 8.47		32.4	-44.4		32.9
Loading details - Combination No.2 - kN/m ²		Sombination No.2 - kIV/m		Bending moment - Combinati	on No.2 - kNm/m	
Toe	8.70 7.67 7.67 8.90		28.3	-36.2		26.7
ಮ ಕ						42.1

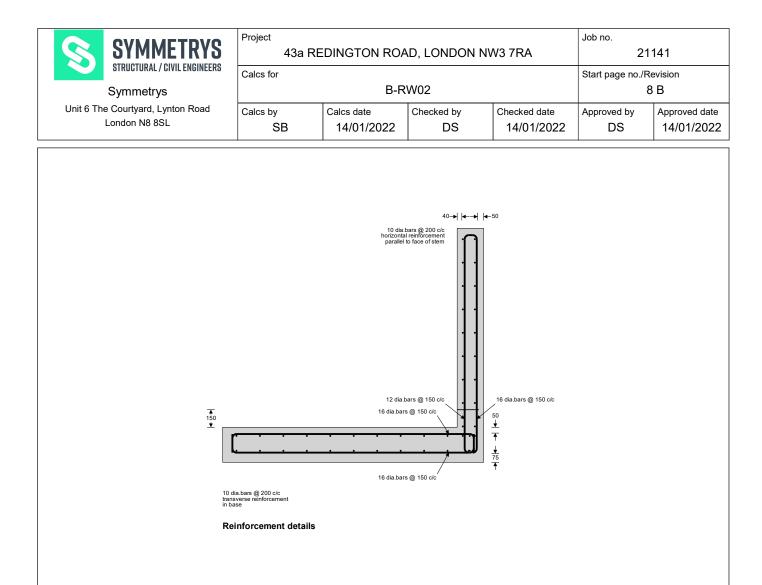
SYMMETRYS	EDINGTON RO	AD, LONDON I	Job no.	1141				
STRUCTURAL / CIVIL ENGINEERS			Start page no./Revision					
Symmetrys	B-F	RW02		5 B				
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs dateChecked byChecked dateApproved by14/01/2022DS14/01/2022DS						
Check stem design at base of Depth of section	fstem	h = 225 mr	n					
•		11 - 223 111						
Rectangular section in flexure		M = 32.9 k	Nm/m					
Design bending moment combin Depth to tension reinforcement			φ _{sr} / 2 = 167 m	m				
Depth to tension reinforcement			$\psi_{sr} / 2 = 107 \text{ m}$ × f _{ck}) = 0.039					
				× (δ - K ₁)/(2 × K ₂)	$(\lambda \times (\delta - K_{4}))$)/(2 × Ka))		
		K' = 0.207	× 0.00/yC/×(1 = X	$\lambda \sim (0 - 10)/(2 \sim 10)$	$\gamma \sim (0 - 1)$	/(Z × 1\2))		
			K'>K-	No compression	n reinforceme	ent is reaui		
Lever arm		z = min(0.5		× K / (η × α _{cc} / γ _c)		-		
Depth of neutral axis			d – z) = 21 mm	(1	, , ,			
Area of tension reinforcement re	equired		(f _{yd} × z) = 477	mm²/m				
Tension reinforcement provided	-		@ 150 c/c					
Area of tension reinforcement p	rovided	$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1340 \text{ mm}^2/\text{m}$						
Minimum area of reinforcement	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 252 \text{ mm}^2/\text{m}$							
Maximum area of reinforcemen	A _{sr.max} = 0.04 × h = 9000 mm ² /m							
		max(A _{sr.req} ,	Asr.min) / Asr.prov	= 0.356				
	PASS - Area o	of reinforcemen	t provided is g	greater than area	of reinforce	ment requi		
				Lik	orary item: Rectar	ngular single ou		
Deflection control - Section 7	.4							
Reference reinforcement ratio		$\rho_0 = \sqrt{f_{ck}} / r_{ck}$	1 N/mm²) / 100	0 = 0.005				
Required tension reinforcement	ratio	$\rho = A_{sr.req} /$	d = 0.003					
Required compression reinforce		$\rho' = A_{sr.2.req}$	/ d ₂ = 0.000					
Structural system factor - Table		K _b = 0.4	_					
Reinforcement factor - exp.7.17		•		\times A _{sr.req} / A _{sr.prov}), '	•			
Limiting span to depth ratio - ex	p.7.16.a			√(f _{ck} / 1 N/mm²) ×	ρ_0 / ρ + 3.2 ×	√(f _{ck} / 1		
			ρ ₀ / ρ - 1) ^{3/2}], 40	0 × K _b) = 16				
Actual span to depth ratio		h _{stem} / d = 10.2 PASS - Span to depth ratio is less than deflection conti						
		PASS	- Span to dep	th ratio is less ti	nan deflectio	n control II		
Crack control - Section 7.3								
Limiting crack width		$W_{max} = 0.3$	mm					
Variable load factor - EN1990 -	adle A1.1	ψ ₂ = 0.6 Μ. – 21 5	kNm/m					
Serviceability bending moment Tensile stress in reinforcement		$M_{sis} = 21.5 \text{ kNm/m}$						
Load duration		$\sigma_s = M_{sls} / (A_{sr,prov} \times z) = 100.9 \text{ N/mm}^2$						
Load duration factor		Long term k _t = 0.4						
Effective area of concrete in ten	ision	к _t = 0.4 A _{c.eff} = min(2.5 × (h - d), (h - x) / 3, h / 2)						
		A _{c.eff} = 6804		,, . .,				
Mean value of concrete tensile	strength		2.9 N/mm ²					
Reinforcement ratio	-		_{ov} / A _{c.eff} = 0.02	0				
Modular ratio		$\alpha_{\rm e} = E_{\rm s} / E_{\rm c}$						
Bond property coefficient		$k_1 = 0.8$						
Strain distribution coefficient		k ₂ = 0.5						
		k ₃ = 3.4						
		k ₄ = 0.425						

SYMMETRYS	-	Project Job no. 43a REDINGTON ROAD, LONDON NW3 7RA						
STRUCTURAL / CIVIL ENGINEERS	Calcs for		Start page no./Revision					
Symmetrys		6 B						
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved dat 14/01/202		
Maximum crack spacing - exp.7	7.11	s _{r.max} = k ₃ ×	c_{sr} + $k_1 \times k_2 \times$	$k_4 \times \phi_{sr}$ / $\rho_{p.eff}$ = 30)8 mm			
Maximum crack width - exp.7.8		Wk = Sr.max >	\propto max(σ_{s} – k _t \times	$(f_{ct.eff} / \rho_{p.eff}) \times (1 +$	$-\alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s$) / Es		
		w _k = 0.093	mm					
		$w_k / w_{max} =$						
		PASS	- Maximum o	crack width is les	s than limitin	ng crack wid		
Rectangular section in shear	- Section 6.2							
Design shear force		V = 44.4 ki						
			3 / γ _C = 0.120					
		k = min(1 +	- √(200 mm / d	l), 2) = 2.000				
Longitudinal reinforcement ratio)		_{r.prov} / d, 0.02) =					
			^{/2} × f _{ck} ^{0.5} = 0.542 N					
Design shear resistance - exp.6	$V_{\text{Rd.c}}$ = max($C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_I \times f_{\text{ck}})^{1/3}$, v_{min}) × d							
		V _{Rd.c} = 115.7 kN/m						
		$V / V_{Rd.c} = 0$						
Havizantal vainfavooment nev	allal ta faca of a		-	hear resistance e	xceeas aesig	gn snear for		
Horizontal reinforcement par Minimum area of reinforcement				_v , 0.001 × t _{stem}) = 3	835 mm ² /m			
Maximum spacing of reinforcen	. ,	-		$v_{\rm r}$, 0.001 \wedge tstem) – \langle	55 11111 /111			
Transverse reinforcement provi	-	@ 200 c/c						
Area of transverse reinforceme			0) = 393 mm²/m				
	-			greater than area	of reinforce	ment requir		
Check base design at toe		h - 200 mm	_					
Depth of section		h = 300 mr	[]					
Rectangular section in flexur								
Design bending moment combi		M = 44.4 k						
Depth to tension reinforcement			- φ _{bb} / 2 = 217 ι	mm				
			\times f _{ck}) = 0.031					
		κ – (2 × η Κ' = 0.207	× α _{cc} /γ _C)×(1 - /	$\lambda \times (\delta - K_1)/(2 \times K_2)$	$)) \times (\lambda \times (0 - \kappa_1))$	$/(2 \times R_2))$		
		R - 0.207	K' > K -	No compression	n reinforceme	ent is requir		
Lever arm		z = min(0.5)		× K / (η × α_{cc} / γ_{c})		•		
Depth of neutral axis			d – z) = 27 mm		, , , ,			
Area of tension reinforcement r	equired							
Tension reinforcement provided	-	A _{bb.req} = M / (f _{yd} × z) = 495 mm²/m 16 dia.bars @ 150 c/c						
Area of tension reinforcement p			-) = 1340 mm²/m				
Minimum area of reinforcement	t - exp.9.1N	$A_{bb.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 327 \text{ mm}^2/\text{m}$						
Maximum area of reinforcemen	-		04 × h = 1200 0					
		max(A _{bb.req}	, A _{bb.min}) / A _{bb.pr}	_{ov} = 0.369				
	PASS - Area o	f reinforcemen	t provided is	greater than area Lil	of reinforce brary item: Rectar	-		
Crack control - Section 7.3								
Limiting crack width		w _{max} = 0.3	mm					
Variable load factor - EN1990 -	- Table A1.1	ψ2 = 0.6						
Serviceability bending moment		M _{sls} = 32 k	Nm/m					
T 11 1 1 1 1 1 1 1		•• • •						

 σ_{s} = M_{sls} / (A_{bb.prov} \times z) = 115.9 N/mm²

Tensile stress in reinforcement

SYMMETRYS	Project 43a REI		AD, LONDON	NW3 7RA	Job no. 2	1141	
STRUCTURAL / CIVIL ENGINEERS							
Symmetrys	Calcs for	B-RW02 Start page no./Revision 7 B					
Unit 6 The Courtyard, Lynton Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/202	
Load duration		Long term					
Load duration factor		k _t = 0.4					
Effective area of concrete in ter	ision	A _{c.eff} = min(2.5 × (h - d), (l	h - x) / 3, h / 2)			
		A _{c.eff} = 909	58 mm²/m				
Mean value of concrete tensile	strength	$f_{ct.eff} = f_{ctm} =$	2.9 N/mm ²				
Reinforcement ratio		$\rho_{p.eff} = A_{bb.pi}$	ov / A _{c.eff} = 0.0	15			
Modular ratio		α_{e} = E _s / E _c	m = 6.091				
Bond property coefficient		k ₁ = 0.8					
Strain distribution coefficient		k ₂ = 0.5					
		k ₃ = 3.4					
		k ₄ = 0.425					
Maximum crack spacing - exp.7	.11	$s_{r.max} = k_3 \times$	$c_{bb} \textbf{+} k_1 \times k_2 \times$	$k_4 \times \varphi_{bb}$ / $\rho_{p.eff}$ = 4	40 mm		
Maximum crack width - exp.7.8		$w_k = s_{r.max} \times$	$\max(\sigma_s - k_t \times$	(f _{ct.eff} / $\rho_{p.eff}$) × (1 +	$+ \alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s$) / E _s	
		w _k = 0.153 mm					
		$w_k / w_{max} =$	0.51				
		PASS	- Maximum c	rack width is les	s than limitin	ng crack wid	
Rectangular section in shear	- Section 6.2						
Design shear force		V = 32.4 kM	l/m				
		$C_{Rd,c} = 0.18$	3 / γ _C = 0.120				
		k = min(1 +	√(200 mm / d), 2) = 1.960			
Longitudinal reinforcement ratio		ρι = min(A _{bl}	o.prov / d, 0.02) :	= 0.006			
-			. ,	² × f _{ck} ^{0.5} = 0.526 N	l/mm²		
Design shear resistance - exp.6	.2a & 6.2b	$V_{\text{Rd.c}} = \max(C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{\text{ck}})^{1/3}, v_{\text{min}}) \times d$					
ů i		V _{Rd.c} = 135 .			, . ,		
		V / V _{Rd.c} = (
		PAS	SS - Design sl	near resistance e	exceeds desig	gn shear for	
Secondary transverse reinfor	cement to base ·	Section 9.3					
Minimum area of reinforcement	– cl.9.3.1.1(2)	A _{bx.req} = 0.2	× A _{bb.prov} = 26	8 mm²/m			
Maximum spacing of reinforcem	nent – cl.9.3.1.1(3) s _{bx_max} = 45	0 mm				
Transverse reinforcement provi	ded	10 dia.bars	@ 200 c/c				
Area of transverse reinforcement	nt provided	$A_{bx.prov} = \pi$	$<\phi_{bx}^2$ / (4 \times s _{bx})) = 393 mm²/m			
	PASS - Area of	reinforcemen	t provided is	greater than area	of reinforce	ment requir	

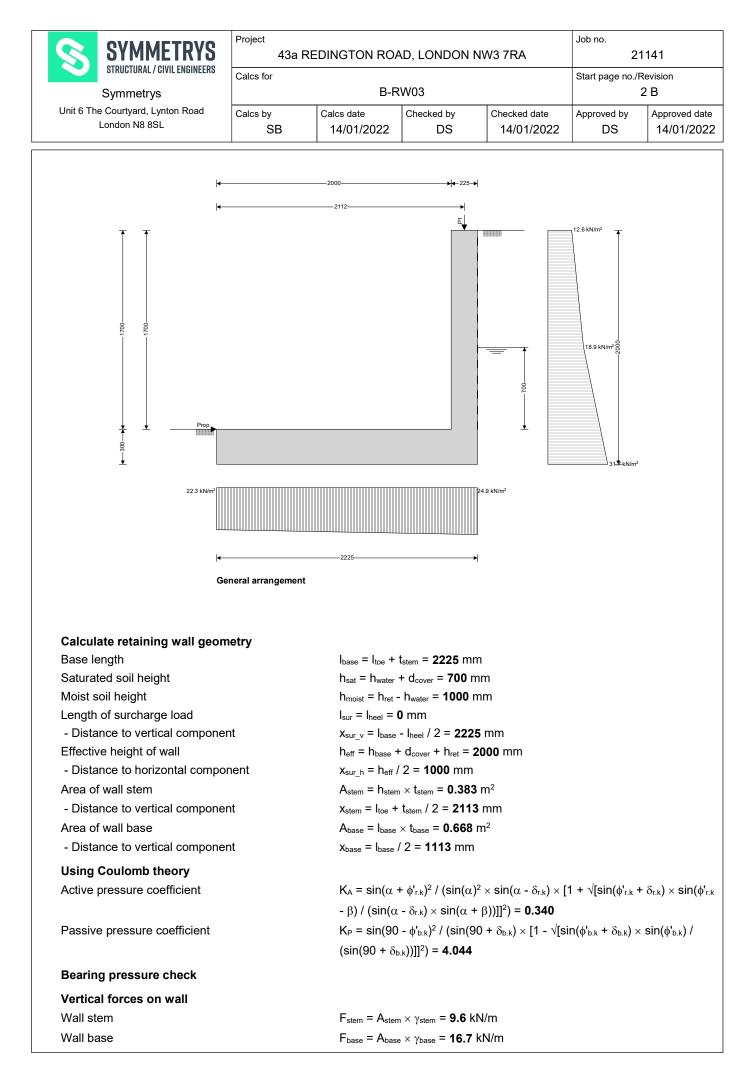


8	SYMMETRYS	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for B-RW03				Start page no./Revision 1 B	
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Cantilever
Stem height	h _{stem} = 1700 mm
Stem thickness	t _{stem} = 225 mm
Angle to rear face of stem	α = 90 deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	I _{toe} = 2000 mm
Base thickness	t _{base} = 300 mm
Base density	γ_{base} = 25 kN/m ³
Height of retained soil	h _{ret} = 1700 mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 700 mm
Water density	γ _w = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm clay
Moist density	γ _{mr} = 19 kN/m ³
Saturated density	γ _{sr} = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{r.k} = 27 deg
Characteristic wall friction angle	$\delta_{r.k}$ = 13.5 deg
Base soil properties	
Soil type	Firm clay
Soil density	γ _b = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{b.k} = 27 deg
Characteristic wall friction angle	δ _{b.k} = 13.5 deg
Characteristic base friction angle	δ _{bb.k} = 18 deg
Presumed bearing capacity	P _{bearing} = 60 kN/m ²
Loading details	
Permanent surcharge load	Surcharge _G = 26.3 kN/m ²
Variable surcharge load	Surcharge _Q = 11.9 kN/m ²
Vertical line load at 2112 mm	P _{G1} = 21.4 kN/m
	P _{Q1} = 4.8 kN/m



	Project 43a	REDINGTON ROA	NW3 7RA	Job no. 21141					
Symmetrys	Calcs for	B-R	W03		Start page no./Revision 3 B				
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022			
Line loads	F _{P_v} = P _{G1} + P _{Q1} = 26.2 kN/m								
Total		$F_{total_v} = F_{ste}$	m + F _{base} + F _{P_}	v + F _{water_v} = 52.5	κN/m				
Horizontal forces on wall									
Surcharge load		$F_{sur_h} = K_A >$	$c\cos(\delta_{r.k}) \times (S)$	urcharge _G + Surch	$harge_{Q}) \times h_{eff} =$	= 25.2 kN/m			
Saturated retained soil		$F_{sat_h} = K_A \times$	$\cos(\delta_{r.k}) \times (\gamma_{sr})$	$_{\rm r}$ - $\gamma_{\rm w}$) $ imes$ (h _{sat} + h _{base}	∍)² / 2 = 1.5 kN	N/m			
Water		$F_{water_h} = \gamma_w$	\times (h _{water} + d _{cove}	_{er} + h _{base})² / 2 = 4.9	kN/m				
Moist retained soil		$F_{moist_h} = K_A$	$\times \text{ cos}(\delta_{r.k}) \times \gamma_{r}$	$_{ m mr} imes$ (($h_{ m eff}$ - $h_{ m sat}$ - $h_{ m b}$	$_{ase})^2$ / 2 + (h _{eff}	- h_{sat} - h_{base}) ×			
		(h _{sat} + h _{base})) = 9.4 kN/m							
Base soil	F_{pass_h} = -K _P × cos($\delta_{b,k}$) × γ_b × (d _{cover} + h _{base}) ² / 2 = -3.4 kN/m								
Total		F _{total_h} = F _{sur}	_{_h} + F _{sat_h} + F _w	_{vater_h} + F _{moist_h} + F _p	_{bass_h} = 37.7 k№	N/m			
Moments on wall									
Wall stem		M _{stem} = F _{ster}	n × X _{stem} = 20.2	2 kNm/m					
Wall base		$M_{\text{base}} = F_{\text{base}}$	$x_{base} = 18.6$	k Nm/m					
Surcharge load		$M_{sur} = -F_{sur}$	h × X _{sur_h} = -25	.2 kNm/m					
Line loads		M _P = (P _{G1} +	P _{Q1}) × p ₁ = 58	5.3 kNm/m					
Saturated retained soil		$M_{sat} = -F_{sat}$	h × X _{sat_h} = -0.5	kNm/m					
Water		$M_{water} = -F_{water}$	$_{ater_h} \times \mathbf{X}_{water_h} =$	- 1.6 kNm/m					
Moist retained soil		$M_{moist} = -F_{moist}$	_{pist_h} × x _{moist_h} =	- 7.3 kNm/m					
Total		$M_{total} = M_{ster}$	n + M _{base} + M _{su}	$_{ur}$ + M _P + M _{sat} + M _v	_{vater} + M _{moist} =	59.4 kNm/m			
Check bearing pressure									
Propping force		F _{prop_base} = F	- _{total_h} = 37.7 k	N/m					
Distance to reaction		$\overline{\mathbf{x}} = \mathbf{M}_{\text{total}}$ /	F _{total_v} = 1133	mm					
Eccentricity of reaction		$e = \overline{x} - I_{base}$, / 2 = 20 mm						
Loaded length of base		$I_{load} = I_{base} =$	2225 mm						
Bearing pressure at toe		$q_{toe} = F_{total_v}$	/ $I_{\text{base}} \times$ (1 - 6	× e / I _{base}) = 22.3 k	xN/m²				
Bearing pressure at heel		$q_{\text{heel}} = F_{\text{total}}$	$_{\rm v}$ / I _{base} \times (1 + 6	6 × e / I _{base}) = 24.9	kN/m ²				
Factor of safety		FoS _{bp} = P _{be}	_{aring} / max(q _{toe} ,	q _{heel}) = 2.413					

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.12

C30/37
f _{ck} = 30 N/mm ²
f _{ck,cube} = 37 N/mm ²
f _{cm} = f _{ck} + 8 N/mm ² = 38 N/mm ²
$f_{ctm} = 0.3 \; N/mm^2 \times (f_{ck} \; / \; 1 \; N/mm^2)^{2/3} = \textbf{2.9} \; N/mm^2$
$f_{ctk,0.05}$ = 0.7 × f_{ctm} = 2.0 N/mm ²
E_{cm} = 22 kN/mm ² × (f _{cm} / 10 N/mm ²) ^{0.3} = 32837 N/mm ²
γ _C = 1.50
$\alpha_{cc} = 0.85$
f_{cd} = $\alpha_{cc} \times f_{ck}$ / γ_{C} = 17.0 N/mm ²
h _{agg} = 20 mm

SYMMETRYS	Project 43a	REDINGTON RO	NW3 7RA				
Structural / Civil Engineers	Calcs for	Calcs for B-RW03				Start page no./Revision 4 B	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022	
Ultimate strain - Table 3.1		ε _{cu2} = 0.003	35				
Shortening strain - Table 3.1		ε _{cu3} = 0.003	35				
Effective compression zone hei	ght factor	$\lambda = 0.80$					
Effective strength factor		η = 1.00					
Bending coefficient k1		K ₁ = 0.40					
Bending coefficient k ₂		K ₂ = 1.00 ×	(0.6 + 0.0014	/ε _{cu2}) = 1.00			
Bending coefficient k ₃		K ₃ = 0.40					
Bending coefficient k4		K ₄ = 1.00 ×	(0.6 + 0.0014	/ε _{cu2}) =1.00			
Reinforcement details			·				
Characteristic yield strength of	reinforcement	f _{yk} = 500 N/	/mm ²				
Modulus of elasticity of reinforc		$E_{\rm s} = 20000$					
Partial factor for reinforcing stee			V I WIIIII				
-		-	- 425 N/mm ²				
Design yield strength of reinford	cement	$I_{yd} = I_{yk} / \gamma_S$	= 435 N/mm ²				
Cover to reinforcement							
Front face of stem		c _{sf} = 40 mn					
Rear face of stem		c _{sr} = 50 mn					
Top face of base		c _{bt} = 50 mn	n				
Bottom face of base		c _{bb} = 75 mr	n				
Sector Too	3.892 8/47 8/47 8.47		43.8	44.4		32.9	
.oading details - Combination No.2 - kN/mP	8.10 7.67 7.67 7.67	Combination No.2 - kN/m	35.3	Bending moment - Combinate	on No.2 - kNm/m	26.7	

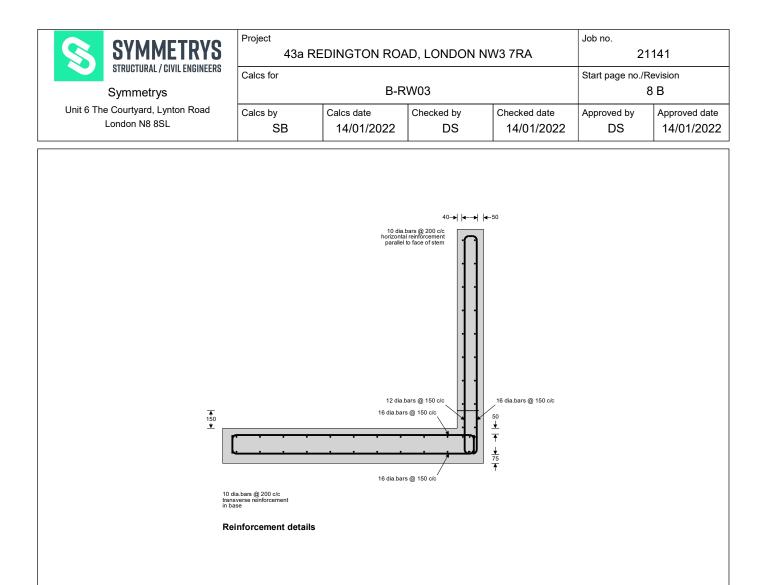
SYMMETRYS	Project 43a RI	EDINGTON RO	AD, LONDON	Job no. 21141				
STRUCTURAL / CIVIL ENGINEERS	GINEERS Calcs for Start page							
Symmetrys		B-F	RW03			5 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Start page no. Start page no. Approved by DS $(\lambda \times (\delta - K_{1})) \times (\lambda \times (\delta - K_{2})) \times (\delta - K_{2}) \times (\delta - K_$	Approved da 14/01/20		
Check stem design at base of	f stem							
Depth of section		h = 225 mr	n					
Rectangular section in flexure	e - Section 6.1							
Design bending moment combined	nation 1	M = 32.9 k	Nm/m					
Depth to tension reinforcement		d = h - c _{sr} -	ϕ_{sr} / 2 = 167 m	ım				
			× f _{ck}) = 0.039					
		, ,	× α _{cc} /γ _C)×(1 - λ	$\lambda \times (\delta - K_1)/(2 \times K_2)$))×(λ × (δ - K ₁))/(2 × K ₂))		
		K' = 0.207						
				-		•		
Lever arm) ^{0.3} , 0.95) × d	= 159 mm		
Depth of neutral axis			i – z) = 21 mm					
Area of tension reinforcement re-	•		$(f_{yd} \times z) = 477$	mm²/m				
Tension reinforcement provideo Area of tension reinforcement p			@ 150 c/c	$-4240 \text{ mm}^{2}/\text{m}$				
		$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1340 \text{ mm}^2/\text{m}$						
Minimum area of reinforcement	-	A _{sr.min} = max(0.26 × f _{ctm} / f _{yk} , 0.0013) × d = 252 mm²/m A _{sr.max} = 0.04 × h = 9000 mm²/m						
Maximum area of reinforcemen	1 - 01.9.2.1.1(3)							
	PASS - Aroa o		A _{sr.min}) / A _{sr.prov}		of reinforce	mont roqui		
	1 A00 - Alea 0	in relinior cellien	i provided is			-		
Deflection control - Section 7	.4							
Reference reinforcement ratio		$\rho_0 = \sqrt{f_{ck}} / f_{ck}$	1 N/mm ²) / 100	0 = 0.005				
Required tension reinforcement	ratio	$\rho = A_{sr.reg} /$	d = 0.003					
Required compression reinforce	ement ratio	$\rho' = A_{sr.2.req}$	/ d ₂ = 0.000					
Structural system factor - Table	7.4N	K _b = 0.4						
Reinforcement factor - exp.7.17		K₅ = min(50	00 N/mm² / (f _{yk}	\times A _{sr.req} / A _{sr.prov}), \cdot	1.5) = 1.5			
Limiting span to depth ratio - ex	p.7.16.a	$min(K_s \times K_l)$	5 × [11 + 1.5 ×	$\sqrt{(f_{ck} / 1 N/mm^2)} \times$	$ ho_0$ / $ ho$ + 3.2 $ imes$	√(f _{ck} / 1		
		N/mm²) × (ρ ₀ / ρ - 1) ^{3/2}], 4	0 × K _b) = 16				
Actual span to depth ratio		h _{stem} / d = 10.2						
		PASS	- Span to dep	oth ratio is less th	han deflectio	n control li		
Crack control - Section 7.3								
Limiting crack width		w _{max} = 0.3	mm					
Variable load factor - EN1990 -	Table A1.1	$\psi_2 = 0.6$						
Serviceability bending moment		M _{sis} = 21.5	kNm/m					
Tensile stress in reinforcement		σ_{s} = M _{sls} / ($A_{sr.prov} \times z) = 1$	00.9 N/mm²				
Load duration		Long term						
Load duration factor		$k_t = 0.4$						
Effective area of concrete in ter	ision		, .	h - x) / 3, h / 2)				
	-4	$A_{c.eff} = 680$						
Mean value of one of the "	strength		2.9 N/mm ²	0				
Mean value of concrete tensile		$O_{n off} \equiv A_{cr pr}$	_{ov} / A _{c.eff} = 0.02	U				
Reinforcement ratio			- 6 004					
Reinforcement ratio Modular ratio		$\alpha_{\rm e}$ = E _s / E _c	_{em} = 6.091					
Reinforcement ratio Modular ratio Bond property coefficient		$\alpha_{e} = E_{s} / E_{c}$ k ₁ = 0.8	m = 6.091					
Reinforcement ratio Modular ratio		$\alpha_{\rm e}$ = E _s / E _c	m = 6.091					

SYMMETRYS	Project 43a RE	43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141				
STRUCTURAL / CIVIL ENGINEERS	Calcs for					Revision				
Symmetrys		B-F	2W03		ate Approved by Ap			6 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved dat 14/01/202				
Maximum crack spacing - exp.7	7.11	$s_{r.max} = k_3 \times$	c_{sr} + $k_1 \times k_2 \times$	$k_4 \times \phi_{sr} / \rho_{p.eff}$ = 30	8 mm					
Maximum crack width - exp.7.8		$w_k = s_{r.max}$	\propto max(σ_s – k _t \times	(f _{ct.eff} / $\rho_{p.eff}$) × (1 +	$\cdot \alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s$) / E _s				
		w _k = 0.093	mm							
		$w_k / w_{max} =$								
		PASS	- Maximum c	crack width is les	s than limitin	ig crack wid				
Rectangular section in shear	- Section 6.2									
Design shear force		V = 44.4 ki								
			3 / γ _C = 0.120							
		•	· √(200 mm / d							
Longitudinal reinforcement ratio)		. _{prov} / d, 0.02) =							
				$f^{2} \times f_{ck}^{0.5} = 0.542 \text{ N}$						
Design shear resistance - exp.6	5.2a & 6.2b			$100 \text{ N}^2/\text{mm}^4 \times \rho_1 \times f_0$	$_{\rm ck})^{1/3}, V_{\rm min}) \times d$					
		$V_{Rd.c} = 115$								
		$V / V_{Rd.c} = 0$		hear resistance e	vcoode doeir	an shoar for				
Horizontal reinforcement par	allel to face of s		-	iear resistance e	xceeus uesi	jii shear ior				
Minimum area of reinforcement				v, 0.001 × t _{stem}) = 3	335 mm²/m					
Maximum spacing of reinforcen	. ,			, ere ere stenry	•••					
Transverse reinforcement provi		10 dia.bars								
Area of transverse reinforceme	nt provided	$A_{sx.prov} = \pi$	$\times \phi_{sx}^2 / (4 \times s_{sx})$) = 393 mm²/m						
	PASS - Area or	f reinforcemen	t provided is	greater than area	of reinforce	ment requir				
Check base design at toe										
Depth of section		h = 300 mr	n							
Rectangular section in flexur	e - Section 6.1									
Design bending moment combi	nation 1	M = 43.1 k	Nm/m							
Depth to tension reinforcement		d = h - c _{bb} -	- φ _{bb} / 2 = 217 ι	mm						
		K = M / (d ²	× f _{ck}) = 0.031							
		K' = (2 × η	× α _{cc} /γ _C)×(1 - λ	$\lambda \times (\delta - K_1)/(2 \times K_2)$))×(λ × (δ - K ₁)	/(2 × K ₂))				
		K' = 0.207								
				No compression		•				
Lever arm				\times K / ($\eta \times \alpha_{cc}$ / γ_{c})) ^{0.5} , 0.95) × d	= 206 mm				
Depth of neutral axis			l – z) = 27 mm							
Area of tension reinforcement r	-	-	′ (f _{yd} × z) = 481	l mm²/m						
Tension reinforcement provided			@ 150 c/c)						
Area of tension reinforcement p				.) = 1340 mm²/m	24					
Minimum area of reinforcement	-			f _{yk} , 0.0013) × d = 3	327 mm²/m					
Maximum area of reinforcemen	n - ci.9.2.1.1(3)		04 × h = 12000							
	PASS - Area of		A _{bb.min}) / A _{bb.pr}	_∾ = 0.359 greater than area	of reinforce	ment requir				
	i AUU - Alea Ul	, ennor ceniell	PIOVIDED IS	-	prary item: Rectar	-				
Crack control - Section 7.3										
Limiting crack width		w _{max} = 0.3	mm							
Variable load factor - EN1990 -	- Table A1.1	$\psi_2 = 0.6$								
Serviceability bending moment		M _{sls} = 31.1	kNm/m							
Townsile stars a in a lafer (•	40 C N//mama?						

 σ_{s} = M_{sls} / (A_{bb.prov} \times z) = 112.6 N/mm²

Tensile stress in reinforcement

SYMMETRYS	Project 43a REI	DINGTON RO	AD, LONDON	NW3 7RA	Job no. 2	1141	
STRUCTURAL / CIVIL ENGINEERS	Calcs for	Start page no./Revision					
Symmetrys		B-F	W03			7 B	
Unit 6 The Courtyard, Lynton Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved dat	
London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/202	
Load duration		Long term					
Load duration factor		k _t = 0.4					
Effective area of concrete in tens	sion	A _{c.eff} = min(2.5 × (h - d), (l	h - x) / 3, h / 2)			
		A _{c.eff} = 909	58 mm²/m				
Mean value of concrete tensile s	trength	f _{ct.eff} = f _{ctm} =	2.9 N/mm ²				
Reinforcement ratio		$\rho_{p.eff} = A_{bb.pl}$	ov / A _{c.eff} = 0.0 1	15			
Modular ratio		$\alpha_{e} = E_{s} / E_{c}$	m = 6.091				
Bond property coefficient		k ₁ = 0.8					
Strain distribution coefficient		k ₂ = 0.5					
		k ₃ = 3.4					
		k ₄ = 0.425					
Maximum crack spacing - exp.7.	.11	s _{r.max} = k ₃ ×	c_{bb} + $k_1 \times k_2 \times$	$k_4 \times \phi_{bb} / \rho_{p.eff} = 4$	40 mm		
Maximum crack width - exp.7.8		$w_k = s_{r.max} \times$	$max(\sigma_s - k_t \times$	$(f_{ct.eff} / \rho_{p.eff}) \times (1 +$	$-\alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s$) / E _s	
		w _k = 0.148 mm					
		$w_k / w_{max} =$	0.495				
		PASS	- Maximum c	rack width is les	s than limitin	ng crack wid	
Rectangular section in shear -	Section 6.2						
Design shear force		V = 43.8 kM	l/m				
		$C_{Rd,c} = 0.18$	8 / γ _C = 0.120				
		k = min(1 +	√(200 mm / d), 2) = 1.960			
Longitudinal reinforcement ratio		$\rho_l = \min(A_{bl})$	_{o.prov} / d, 0.02) :	= 0.006			
		v _{min} = 0.035	$5 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2}$	² × f _{ck} ^{0.5} = 0.526 N	/mm ²		
Design shear resistance - exp.6.	.2a & 6.2b	V _{Rd.c} = max	$(C_{Rd.c} \times k \times (10))$	00 N ² /mm ⁴ \times $\rho_{l} \times$ f	$_{\rm ck})^{1/3}, V_{\rm min}) \times d$		
		V _{Rd.c} = 135	1 kN/m				
		$V / V_{Rd.c} = 0$).324				
		PAS	SS - Design sl	near resistance e	xceeds desig	gn shear for	
Secondary transverse reinford	ement to base -	Section 9.3					
Minimum area of reinforcement	– cl.9.3.1.1(2)	$A_{bx.req} = 0.2$	× A _{bb.prov} = 26	8 mm²/m			
Maximum spacing of reinforcem	ent – cl.9.3.1.1(3) s _{bx_max} = 45	0 mm				
Transverse reinforcement provid	led	10 dia.bars	@ 200 c/c				
Area of transverse reinforcemen	t provided	$A_{bx.prov} = \pi$	$<\phi_{bx}^{2}/(4 \times s_{bx})$) = 393 mm²/m			
	PASS - Area of	reinforcemen	t provided is	greater than area	of reinforce	ment requir	



\$	SYMMETRYS	Project 43a RE	DINGTON ROA	Job no. 21141			
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Unit 6 The Courtyard, Lynton Road London N8 8SL		Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022

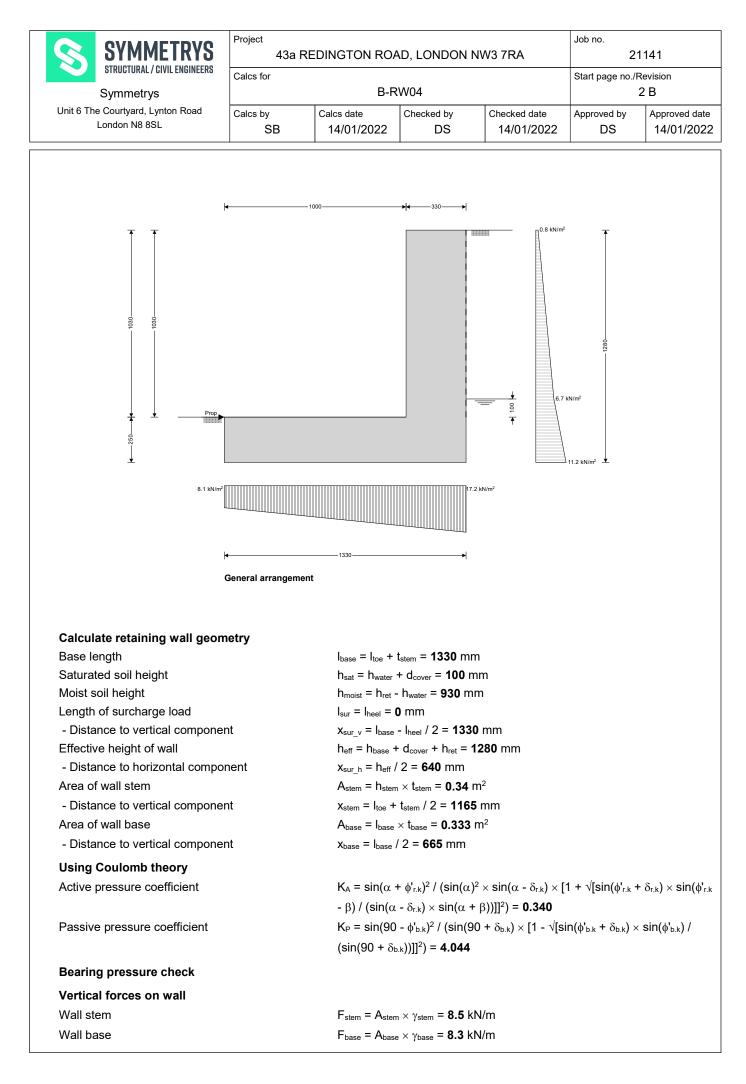
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Cantilever
Stem height	h _{stem} = 1030 mm
Stem thickness	t _{stem} = 330 mm
Angle to rear face of stem	α = 90 deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	l _{toe} = 1000 mm
Base thickness	t _{base} = 250 mm
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	h _{ret} = 1030 mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 100 mm
Water density	γ _w = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm clay
Moist density	γ _{mr} = 19 kN/m ³
Saturated density	γ_{sr} = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{r.k} = 27 deg
Characteristic wall friction angle	$\delta_{r.k}$ = 13.5 deg
Base soil properties	
Soil type	Firm clay
Soil density	$\gamma_{\rm b}$ = 19 kN/m ³
Characteristic effective shear resistance angle	φ' _{b.k} = 27 deg
Characteristic wall friction angle	$\delta_{b.k}$ = 13.5 deg
Characteristic base friction angle	$\delta_{bb.k} = 18 \text{ deg}$
Presumed bearing capacity	P _{bearing} = 60 kN/m ²
Loading details	
Variable surcharge load	Surcharge _Q = 2.5 kN/m ²

Note:

The retaining wall has been designed to support lateral pressures only. Assuming the slab (retaining wall base) has sufficient stiffness, this will transfer the vertical load to a mass concrete strip foundation uniformly. Refer to "Addendum to structural calculation package" for strip foundation and load bearing pressure check.



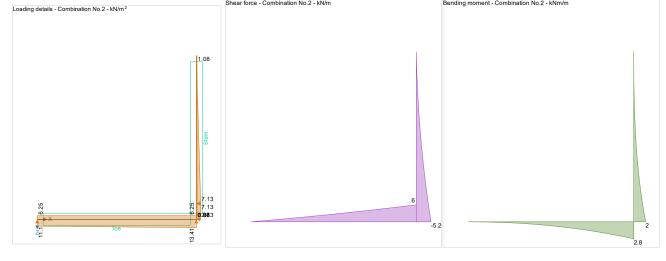
SYMMETRYS	Project 43a	REDINGTON ROA	Job no. 2	1141				
STRUCTURAL / CIVIL ENGINEERS	Calcs for							
Symmetrys		B-R	W04		3 B			
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022		
Total	F _{total_v} = F _{stem} + F _{base} + F _{water_v} = 16.8 kN/m							
Horizontal forces on wall								
Surcharge load		$F_{sur_h} = K_A >$	$\propto \cos(\delta_{r.k}) imes Su$	$rcharge_Q \times h_{eff} = 1$	l .1 kN/m			
Saturated retained soil		$F_{sat_h} = K_A \times$	$c\cos(\delta_{r.k}) imes (\gamma_{sr})$	- γ_w) × (h _{sat} + h _{base}	_e)² / 2 = 0.2 kN	l/m		
Water		$F_{water_h} = \gamma_w$	\times (h _{water} + d _{cove}	er + h _{base})² / 2 = 0.6	3 kN/m			
Moist retained soil		$F_{moist_h} = K_A$	$\times \text{ cos}(\delta_{r.k}) \times \gamma_r$	$_{ m mr} imes$ (($h_{ m eff}$ - $h_{ m sat}$ - $h_{ m back}$	_{ase}) ² / 2 + (h _{eff}	- h_{sat} - h_{base}) ×		
		(h _{sat} + h _{base})) = 4.8 kN/m					
Base soil	$F_{pass_h} = -K_P \times cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = -2.3 \text{ kN/m}$							
Total	F _{total_h} = F _{sur_h} + F _{sat_h} + F _{water_h} + F _{moist_h} + F _{pass_h} = 4.3 kN/m							
Moments on wall								
Wall stem		$M_{stem} = F_{ster}$	n × x _{stem} = 9.9	kNm/m				
Wall base		$M_{\text{base}} = F_{\text{base}}$	$x_{base} = 5.5$	kNm/m				
Surcharge load		$M_{sur} = -F_{sur}$	h × X _{sur_h} = -0.7	′ kNm/m				
Saturated retained soil		M _{sat} = -F _{sat}	n × x _{sat_h} = 0 kN	Nm/m				
Water		M _{water} = -F _{wa}	$_{ater_h} \times \mathbf{X}_{water_h} =$	• -0.1 kNm/m				
Moist retained soil		$M_{moist} = -F_{mos}$	_{pist_h} × X _{moist_h} =	-2.1 kNm/m				
Total		M _{total} = M _{ster}	n + M _{base} + M _{su}	ur + M _{sat} + M _{water} +	M _{moist} = 12.5 k	Nm/m		
Check bearing pressure								
Propping force		$F_{prop_base} = F$	t _{otal_h} = 4.3 kN	/m				
Distance to reaction		$\overline{\mathbf{x}} = \mathbf{M}_{\text{total}}$ /	F _{total_v} = 744 m	ım				
Eccentricity of reaction		$e = \bar{x} - I_{base}$, / 2 = 79 mm					
Loaded length of base		$I_{load} = I_{base} =$	1330 mm					
Bearing pressure at toe		$q_{toe} = F_{total_v}$	/ $I_{\text{base}} \times (1$ - 6	× e / I _{base}) = 8.1 kN	N/m ²			
Bearing pressure at heel		$q_{heel} = F_{total}$	$_{\rm v}$ / I _{base} \times (1 + 6	$\delta \times e / I_{base}$) = 17.2	kN/m ²			
Factor of safety		$FoS_{bp} = P_{be}$	_{aring} / max(q _{toe} ,	q _{heel}) = 3.496				
	PASS -	Allowable bearin	g pressure ex	xceeds maximun	n applied bea	ring pressure		

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.12

Concrete strength class	C30/37
Characteristic compressive cylinder strength	f _{ck} = 30 N/mm ²
Characteristic compressive cube strength	f _{ck,cube} = 37 N/mm ²
Mean value of compressive cylinder strength	f _{cm} = f _{ck} + 8 N/mm ² = 38 N/mm ²
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm ² × (f_{ck} / 1 N/mm ²) ^{2/3} = 2.9 N/mm ²
5% fractile of axial tensile strength	$f_{ctk,0.05}$ = 0.7 × f_{ctm} = 2.0 N/mm ²
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm ² × (f _{cm} / 10 N/mm ²) ^{0.3} = 32837 N/mm ²
Partial factor for concrete - Table 2.1N	γc = 1.50
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15	f_{cd} = $\alpha_{cc} \times f_{ck}$ / γ_{C} = 17.0 N/mm ²
Maximum aggregate size	h _{agg} = 20 mm
Ultimate strain - Table 3.1	ε _{cu2} = 0.0035
Shortening strain - Table 3.1	ε _{cu3} = 0.0035

SYMMETRYS		Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141		
STRUCTURAL/CIVIL ENGINEERS Symmetrys	Calcs for B-RW04				Start page no./Revision 4 B			
Unit 6 The Courtyard, Lynton Road	Calaa hu							
London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	14/01/202		
Effective compression zone hei	ght factor	λ = 0.80						
Effective strength factor		η = 1.00						
Bending coefficient k1		K ₁ = 0.40						
Bending coefficient k ₂		K ₂ = 1.00 ×	(0.6 + 0.0014/	ε _{cu2}) = 1.00				
Bending coefficient k ₃		K ₃ = 0.40						
Bending coefficient k ₄			(0.6 + 0.0014/	ε _{cu2}) =1.00				
Reinforcement details				,				
Characteristic yield strength of	reinforcement	f., = 500 N	/mm ²					
Modulus of elasticity of reinforce								
Partial factor for reinforcing stee								
Design yield strength of reinforce								
	Sement	iya — iyk / ys						
Cover to reinforcement								
Front face of stem	$c_{sf} = 40 \text{ mm}$							
Rear face of stem		c _{sr} = 50 mm						
Top face of base Bottom face of base		c _{bt} = 50 mn c _{bb} = 75 mr						
Dollom lace of base		C _{bb} – 75 mi						
adias dataila. Cambinatias Na 4, UNV=2	Shear force - Co	ombination No.1 - kN/m		Bending moment - Combination	on No.1 - kNm/m			
pading details - Combination No.1 - kN/m ²								
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SYMMETRYS	EDINGTON RO	AD, LONDON	Job no. 21141					
STRUCTURAL / CIVIL ENGINEERS				Start page no./Revision				
Symmetrys		B-RW04				5 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved da 14/01/20		
Check stem design at base of	stem							
Depth of section		h = 330 mr	n					
Rectangular section in flexure	e - Section 6.1							
Design bending moment combined	nation 1	M = 2.2 kN	m/m					
Depth to tension reinforcement			φ _{sr} / 2 = 272 m	nm				
			× f _{ck}) = 0.001					
			× α _{cc} /γ _C)×(1 - λ	$\lambda \times (\delta - K_1)/(2 \times K_2)$))×(λ × (δ - K ₁))/(2 × K ₂))		
		K' = 0.207						
				No compression		•		
Lever arm				\times K / ($\eta \times \alpha_{cc}$ / γ_{c})) [,] , U.95) × d	= 258 mm		
Depth of neutral axis			l – z) = 34 mm					
Area of tension reinforcement re Tension reinforcement provided	-	A _{sr.req} = M / 16 dia.bars	$(f_{yd} \times z) = 20 r$	111(1-/11)				
Area of tension reinforcement provided			-	$-1340 \text{ mm}^2/\text{m}$				
Minimum area of reinforcement		$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ 1340 mm ² /m $A_{sr,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ 410 mm ² /m						
Maximum area of reinforcement	-							
	1 - 01.9.2.1.1(5)	A _{sr.max} = 0.04 × h = 13200 mm²/m max(A _{sr.req} , A _{sr.min}) / A _{sr.prov} = 0.306						
	PASS - Area o			greater than area	of reinforce	ment requi		
				-	orary item: Rectar	-		
Deflection control - Section 7	.4							
Reference reinforcement ratio		$\rho_0 = \sqrt{f_{ck}} / f_{ck}$	1 N/mm ²) / 100	00 = 0.005				
Required tension reinforcement	ratio	$\begin{split} \rho &= A_{sr.req} / d = \textbf{0.000} \\ \rho' &= A_{sr.2.req} / d_2 = \textbf{0.000} \\ K_b &= \textbf{0.4} \\ K_s &= \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), \ 1.5) = \textbf{1.5} \end{split}$						
Required compression reinforce	ement ratio							
Structural system factor - Table	7.4N							
Reinforcement factor - exp.7.17								
Limiting span to depth ratio - ex	p.7.16.a	$min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 N/mm^2)})$						
		N/mm ²) × (ρ_0 / ρ - 1) ^{3/2}], 40 × K _b) = 16						
Actual span to depth ratio		h _{stem} / d = 3.8						
		PASS	- Span to dep	oth ratio is less th	han deflectio	n control li		
Crack control - Section 7.3								
Limiting crack width		w _{max} = 0.3	mm					
Variable load factor - EN1990 -	Table A1.1	$\psi_2 = 0.6$						
Serviceability bending moment			M _{sis} = 1.4 kNm/m					
Tensile stress in reinforcement		$\sigma_{s} = M_{sls} / (A_{sr.prov} \times z) = 4.1 \text{ N/mm}^{2}$						
Load duration		Long term						
Load duration factor	aion	k _t = 0.4 A _{c.eff} = min(2.5 × (h - d), (h - x) / 3, h / 2)						
Effective area of concrete in ten	5011			n - x) / 3, n / 2)				
Mean value of concrete tensile	strength	A _{c.eff} = 98667 mm ² /m f _{ct.eff} = f _{ctm} = 2.9 N/mm ²						
Reinforcement ratio		$\rho_{p,eff} = \Lambda_{sr.prov} / \Lambda_{c.eff} = 0.014$						
Modular ratio		$\rho_{\text{p.eff}} = A_{\text{sr.prov}} / A_{\text{c.eff}} = 0.014$ $\alpha_{\text{e}} = E_{\text{s}} / E_{\text{cm}} = 6.091$						
Bond property coefficient		$\alpha_{\rm e} = E_{\rm s} / E_{\rm cm} = 6.091$ k ₁ = 0.8						
		k ₂ = 0.5						
Strain distribution coefficient								
Strain distribution coefficient		k ₃ = 3.4						

SYMMETRYS	Project 43a RE	EDINGTON RO	AD, LONDON	NW3 7RA	Job no.	1141	
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./	Revision	
Symmetrys	B-RW04				6 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved dat 14/01/202	
Maximum crack spacing - exp.7	7.11	$s_{r.max} = k_3 \times$	c_{sr} + $k_1 \times k_2 \times$	$k_4 \times \phi_{sr} / \rho_{p.eff}$ = 37	' 0 mm		
Maximum crack width - exp.7.8		W _k = S _{r.max} >	$\propto \max(\sigma_s - \mathbf{k}_t \times$	(f _{ct.eff} / $\rho_{p.eff}$) × (1 +	$\alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s$) / Es	
		w _k = 0.005	mm				
		$w_k / w_{max} =$	0.015				
		PASS	- Maximum o	crack width is les	s than limitin	ng crack wid	
Rectangular section in shear	- Section 6.2						
Design shear force		V = 5.8 kN/	′m				
		$C_{Rd,c} = 0.18$	8 / γ _C = 0.120				
		k = min(1 +	· √(200 mm / d), 2) = 1.857			
Longitudinal reinforcement ratio)	ρι = min(A _{si}	. _{prov} / d, 0.02) =	= 0.005			
		v _{min} = 0.038	$5 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2}$	¹² × f _{ck} ^{0.5} = 0.485 N	/mm²		
Design shear resistance - exp.6	6.2a & 6.2b	V _{Rd.c} = max	$(C_{Rd.c} \times k \times (10))$	00 N ² /mm ⁴ \times $\rho_{l} \times$ f	$_{ck})^{1/3}, v_{min}) imes d$		
		V _{Rd.c} = 148	. 8 kN/m				
		V / V _{Rd.c} = 0.039					
		PAS	SS - Design sl	hear resistance e	xceeds desig	gn shear for	
Horizontal reinforcement par							
Minimum area of reinforcement	. ,	$A_{sx,req} = max(0.25 \times A_{sr,prov}, 0.001 \times t_{stem}) = 335 \text{ mm}^2/\text{m}$					
Maximum spacing of reinforcen	. ,	-					
Transverse reinforcement provi		10 dia.bars @ 200 c/c					
Area of transverse reinforceme	-) = 393 mm²/m			
	PASS - Area o	f reinforcemen	t provided is	greater than area	of reinforce	ment requir	
Check base design at toe							
Depth of section		h = 250 mr	n				
Rectangular section in flexur	e - Section 6.1						
Design bending moment combi	nation 1	M = 2.9 kN	m/m				
Depth to tension reinforcement		d = h - c _{bb} - φ _{bb} / 2 = 167 mm					
		K = M / (d ²	× f _{ck}) = 0.003				
		K' = (2 × η K' = 0.207	× α _{cc} /γ _C)×(1 - λ	$\lambda \times (\delta - K_1)/(2 \times K_2)$))×(λ × (δ - K ₁)	/(2 × K ₂))	
			K' > K -	No compression	n reinforceme	ent is requir	
Lever arm		z = min(0.5	+ 0.5 × (1 - 2	\times K / ($\eta \times \alpha_{cc}$ / γ_{c})) ^{0.5} , 0.95) × d	= 159 mm	
Depth of neutral axis		$x = 2.5 \times (c$	l – z) = 21 mm	l			
Area of tension reinforcement r	equired		' (f _{yd} × z) = 42				
Tension reinforcement provideo	-	-	@ 150 c/c				
Area of tension reinforcement p	provided	$A_{bb,prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 1340 \text{ mm}^2/\text{m}$					
Minimum area of reinforcement	t - exp.9.1N	-		f _{yk} , 0.0013) × d = 2	2 52 mm²/m		
Maximum area of reinforcemen	-		04 × h = 1000				
			A _{bb.min}) / A _{bb.pr}				
	PASS - Area o			greater than area	o f reinforce prary item: Rectar	-	
Crack control - Section 7.3					,	J 3 54	
Limiting crack width		w _{max} = 0.3	mm				
Variable load factor - EN1990 -	- Table A1.1	ψ ₂ = 0.6					
Serviceability bending moment		Ψ2	Nm/m				
Tensile stress in reinforcement	$\sigma = M \cdot ((\Lambda \cdot \cdot \cdot \cdot \cdot z) = 97 \text{ N}/\text{mm}^2$						

 σ_{s} = M_{sls} / (A_{bb.prov} \times z) = 9.7 N/mm²

Tensile stress in reinforcement

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STRUCTURAL / CIVIL ENGINEERS								
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London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/202		
Load duration		Long term						
Load duration factor		k _t = 0.4						
Effective area of concrete in ter	ision	A _{c.eff} = min(2.5 × (h - d), (l	h - x) / 3, h / 2)				
		A _{c.eff} = 763	75 mm²/m					
Mean value of concrete tensile	strength	$f_{ct.eff} = f_{ctm} =$	2.9 N/mm ²					
Reinforcement ratio		$\rho_{p.eff} = A_{bb.p}$	ov / A _{c.eff} = 0.0 1	18				
Modular ratio		$\alpha_{e} = E_{s} / E_{c}$	m = 6.091					
Bond property coefficient		k ₁ = 0.8						
Strain distribution coefficient		k ₂ = 0.5						
		k ₃ = 3.4						
		k ₄ = 0.425						
Maximum crack spacing - exp.7	.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} \text{ / } \rho_{p.eff} = \textbf{410 mm}$						
Maximum crack width - exp.7.8		$w_{k} = s_{r.max} \times max(\sigma_{s} - k_{t} \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_{e} \times \rho_{p.eff}), 0.6 \times \sigma_{s}) / E_{s}$						
		w _k = 0.012 mm						
		w _k / w _{max} = 0.04						
		PASS	- Maximum c	rack width is les	s than limitin	ng crack wid		
Rectangular section in shear	- Section 6.2							
Design shear force		V = 7.2 kN/m						
		$C_{Rd,c} = 0.18 / \gamma_{C} = 0.120$						
		k = min(1 +	√(200 mm / d), 2) = 2.000				
Longitudinal reinforcement ratio			$p_{\rm l} = \min(A_{\rm bb, prov} / d, 0.02) = 0.008$					
-		$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2} \times f_{ck}^{0.5} = 0.542 \text{ N/mm}^2$						
Design shear resistance - exp.6	.2a & 6.2b	$V_{\text{Rd.c}} = \max(C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{\text{ck}})^{1/3}, v_{\text{min}}) \times d$						
5		$V_{\text{Rd,c}} = 115.7 \text{ kN/m}$						
		$V / V_{Rd.c} = 0$						
				hear resistance e	xceeds desig	gn shear for		
Secondary transverse reinfor	cement to base ·	Section 9.3						
Minimum area of reinforcement	– cl.9.3.1.1(2)	$A_{bx.req} = 0.2$	× A _{bb.prov} = 26	8 mm²/m				
Maximum spacing of reinforcem	nent – cl.9.3.1.1(3							
Transverse reinforcement provi	ded	10 dia.bars @ 200 c/c						
Area of transverse reinforcement	nt provided	$A_{bx.prov} = \pi$	$<\phi_{bx}^{2}/(4 \times s_{bx})$) = 393 mm²/m				
	PASS - Area of	reinforcemen	t provided is	greater than area	of reinforce	ment requir		

SYMMETRYS STRUCTURAL / CIVIL ENGINEERS Symmetrys	Project 43a REDINGTON ROAD, LONDON NW3 7RA Calcs for B-RW04				Job no. 21141 Start page no./Revision 8 B	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022
		40 10 dia bars @ 200 c/c hotizontal reinforcement parallel to face of stem				
	10 dia.bars @ 200 c/c transverse reinforcement	12 dia.bars @ 150 c/c 16 dia.bars @ 150 c/c 16 dia.bars @ 150 c/c	السا	16 dia.bars @ 150 c/c 50 ↑ ↓ 75		

Reinforcement details

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	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for	B-R	B-RW05			evision B
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Cantilever
Stem height	h _{stem} = 2700 mm
Stem thickness	t _{stem} = 330 mm
Angle to rear face of stem	α = 90 deg
Stem density	γ_{stem} = 25 kN/m ³
Toe length	I _{toe} = 2000 mm
Base thickness	t _{base} = 250 mm
Base density	γ_{base} = 25 kN/m ³
Height of retained soil	h _{ret} = 2700 mm
Angle of soil surface	$\beta = 0 \text{ deg}$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 1700 mm
Water density	γ _w = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm clay
Moist density	γ _{mr} = 19 kN/m ³
Saturated density	γ _{sr} = 19 kN/m ³
Characteristic effective shear resistance angle	∮' _{r.k} = 27 deg
Characteristic wall friction angle	$\delta_{r.k}$ = 13.5 deg
Base soil properties	
Soil type	Firm clay
Soil density	γ _b = 19 kN/m ³
Characteristic effective shear resistance angle	∮' _{b.k} = 27 deg
Characteristic wall friction angle	δ _{b.k} = 13.5 deg
Characteristic base friction angle	δ _{bb.k} = 18 deg
Presumed bearing capacity	P _{bearing} = 60 kN/m ²
Loading details	
Variable surcharge load	Surcharge _Q = 2.5 kN/m ²
Vertical line load at 2165 mm	P _{G1} = 1.2 kN/m
	P _{Q1} = 3.1 kN/m

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STRUCTURAL / CIVIL ENGINEERS	Calcs for	RD	2///05		Start page no./I	Revision 2 B	
Symmetrys Unit 6 The Courtyard, Lynton Road	B-RW05 Calcs by Calcs date Checked by Checked date						
London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	14/01/2022	Approved by DS	Approved d 14/01/20	
	 ∢	20002165	> <_330_ >				
T T	1.			0.8 kN/m²	Ŧ		
			=	7.1 kN/m ²			
2700					2950		
					Ĭ		
	Prop						
¥ ↓ 520 ↓					1 k4¶/m²		
	30.9 kN/m ²		4.4 kN/m ²				
	◀	2330	▶				
	General arr	angement					
Calculate retaining wall geon	netry						
Base length		$I_{base} = I_{toe} +$	t _{stem} = 2330 mm				
Saturated soil height		$h_{sat} = h_{water} + d_{cover} = 1700 \text{ mm}$					
	$h_{\text{moist}} = h_{\text{ret}} - h_{\text{water}} = 1000 \text{ mm}$						
Moist soil height							
Length of surcharge load	4	$I_{sur} = I_{heel} = 0$	0 mm				
Length of surcharge load - Distance to vertical compone	nt	I _{sur} = I _{heel} = (x _{sur_v} = I _{base}	0 mm - I _{heel} / 2 = 2330	mm			
Length of surcharge load - Distance to vertical compone Effective height of wall		I _{sur} = I _{heel} = 1 x _{sur_v} = I _{base} + h _{eff} = h _{base} +	0 mm - I _{heel} / 2 = 2330 ⊦ d _{cover} + h _{ret} = 29	mm			
Length of surcharge load - Distance to vertical compone		$\begin{split} I_{sur} &= I_{heel} = 0 \\ x_{sur_v} &= I_{base} \\ h_{eff} &= h_{base} + 0 \\ x_{sur_h} &= h_{eff} \end{split}$	0 mm - I _{heel} / 2 = 2330	mm 950 mm			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo	onent	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = h_{stem}$	0 mm - I _{heel} / 2 = 2330 ⊦ d _{cover} + h _{ret} = 29 / 2 = 1475 mm	mm 9 50 mm n ²			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo Area of wall stem	onent	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = h_{stem}$ $x_{stem} = I_{toe} + 0$	0 mm - I _{heel} / 2 = 2330 + d _{cover} + h _{ret} = 29 / 2 = 1475 mm n × t _{stem} = 0.891 µ	mm 9 50 mm n ² mm			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo Area of wall stem - Distance to vertical compone	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = h_{sten}$ $x_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$	0 mm - I _{heel} / 2 = 2330 + d _{cover} + h _{ret} = 29 / 2 = 1475 mm n × t _{stem} = 0.891 n t t _{stem} / 2 = 2165	mm 9 50 mm n ² mm			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo Area of wall stem - Distance to vertical compone Area of wall base	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = h_{sten}$ $x_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$	0 mm - I _{heel} / 2 = 2330 + d _{cover} + h _{ret} = 29 / 2 = 1475 mm h × t _{stem} = 0.891 H + t _{stem} / 2 = 2165 × t _{base} = 0.583 m	mm 9 50 mm n ² mm			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$ $x_{base} = I_{base} + 0$ $K_A = sin(\alpha + 0)$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 25$ / 2 = 1475 mm h × $t_{stem} = 0.891$ f + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ n / 2 = 1165 mm + $\phi'_{r.k}$ /2 / $(sin(\alpha)^2)$	mm 9 50 mm n^2 mm n^2 × sin(α - $\delta_{r,k}$) × [1 + √[sin(¢'r.k +	⊦ δ _{r.k}) × sin(α	
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $x_{base} = I_{base} + 0$ $x_{base} = I_{base} + 0$ $K_A = \sin(\alpha + 0) + 0$ $(\sin(\alpha + 0)) + 0$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 29$ / 2 = 1475 mm h × $t_{stem} = 0.891$ H + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ m / 2 = 1165 mm + $\phi'_{r.k} / 2 / (sin(\alpha)^2)$ - $\delta_{r.k} / x sin(\alpha + 1)$	mm 950 mm n^2 mm n^2 × sin($\alpha - \delta_{r,k}$) × [β))]] ²) = 0.340			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compo Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$ $x_{base} = I_{base} + 0$ $K_A = \sin(\alpha + 0)$ $K_B = \sin(90)$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 29$ / 2 = 1475 mm h × $t_{stem} = 0.891$ f + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ n / 2 = 1165 mm + $\phi'_{r.k}$ ² / $(sin(\alpha)^2$ - $\delta_{r.k}) × sin(\alpha + 1)$ - $\phi'_{b.k}$ ² / $(sin(90)^2)$	mm 950 mm n^2 mm n^2 × sin($\alpha - \delta_{r,k}$) × [β))]] ²) = 0.340			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$ $x_{base} = I_{base} + 0$ $K_A = \sin(\alpha + 0)$ $K_B = \sin(90)$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 29$ / 2 = 1475 mm h × $t_{stem} = 0.891$ H + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ m / 2 = 1165 mm + $\phi'_{r.k} / 2 / (sin(\alpha)^2)$ - $\delta_{r.k} / x sin(\alpha + 1)$	mm 950 mm n^2 mm n^2 × sin($\alpha - \delta_{r,k}$) × [β))]] ²) = 0.340			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient Passive pressure coefficient	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $A_{base} = I_{base}$ $x_{base} = I_{base} + 0$ $K_A = \sin(\alpha + 0)$ $K_B = \sin(90)$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 29$ / 2 = 1475 mm h × $t_{stem} = 0.891$ f + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ n / 2 = 1165 mm + $\phi'_{r.k}$ ² / $(sin(\alpha)^2$ - $\delta_{r.k}) × sin(\alpha + 1)$ - $\phi'_{b.k}$ ² / $(sin(90)^2)$	mm 950 mm n^2 mm n^2 × sin($\alpha - \delta_{r,k}$) × [β))]] ²) = 0.340			
Length of surcharge load - Distance to vertical compone Effective height of wall - Distance to horizontal compone Area of wall stem - Distance to vertical compone Area of wall base - Distance to vertical compone Using Coulomb theory Active pressure coefficient Passive pressure coefficient Bearing pressure check	nent nt	$I_{sur} = I_{heel} = 0$ $x_{sur_v} = I_{base}$ $h_{eff} = h_{base} + 0$ $x_{sur_h} = h_{eff} + 0$ $A_{stem} = I_{toe} + 0$ $A_{base} = I_{base} + 0$ $K_{base} = I_{base} + 0$ $K_{A} = sin(\alpha + 0) + 0$ $K_{P} = sin(90)$ $(sin(90 + \delta t))$	0 mm - $I_{heel} / 2 = 2330$ + $d_{cover} + h_{ret} = 29$ / 2 = 1475 mm h × $t_{stem} = 0.891$ f + $t_{stem} / 2 = 2165$ × $t_{base} = 0.583$ n / 2 = 1165 mm + $\phi'_{r.k}$ ² / $(sin(\alpha)^2$ - $\delta_{r.k}) × sin(\alpha + 1)$ - $\phi'_{b.k}$ ² / $(sin(90)^2)$	mm 950 mm n^{2} mm n^{2} × sin($\alpha - \delta_{r,k}$) × [β))]] ²) = 0.340 + $\delta_{b,k}$) × [1 - \sqrt{s}			

SYMMETRYS	Project 43a	REDINGTON ROA	AD, LONDON	NW3 7RA	Job no. 2	1141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for	Calcs for B-RW05				Start page no./Revision 3 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved dat 14/01/202		
Line loads		F _{P_v} = P _{G1} + P _{Q1} = 4.3 kN/m						
Total		$F_{total_v} = F_{ster}$	m + F _{base} + F _{P_}	_v + F _{water_v} = 41.1	kN/m			
Horizontal forces on wall								
Surcharge load		F _{sur_h} = K _A >	$(\cos(\delta_{r.k}) \times Su)$	$rcharge_Q \times h_{eff} = 2$	2 .4 kN/m			
Saturated retained soil		F _{sat_h} = K _A ⇒	$< \cos(\delta_{r.k}) \times (\gamma_s)$	$_{\rm r}$ - $\gamma_{\rm w}$) $ imes$ (h _{sat} + h _{base}	_e)² / 2 = 5.8 kN	l/m		
Water		$F_{water_h} = \gamma_w$	\times (h _{water} + d _{cov}	_{er} + h _{base})² / 2 = 18	. 7 kN/m			
Moist retained soil	$F_{\text{moist }h} = K_{\text{A}} \times \cos(\delta_{r,k}) \times \gamma_{\text{mr}} \times ((h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}})^2 / 2 + (h_{\text{eff}} - h_{\text{sat}} - h_{\text{base}})^2 / 2$					- h _{sat} - h _{base}) :		
	$(h_{sat} + h_{base})) = 15.4 \text{ kN/m}$							
Base soil	$F_{pass_h} = -K_P \times cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = -2.3 \text{ kN/m}$					m		
Total	F _{total_h} = F _{sur_h} + F _{sat_h} + F _{water_h} + F _{moist_h} + F _{pass_h} = 39.9 kN/m					N/m		
Moments on wall								
Wall stem		M _{stem} = F _{ster}	m × X _{stem} = 48.2	2 kNm/m				
Wall base		$M_{base} = F_{bas}$	$_{e} \times x_{base} = 17 k$	۸m/m				
Surcharge load		$M_{sur} = -F_{sur}$	$h \times \mathbf{X}_{sur_h} = -3.6$	3 kNm/m				
Line loads		M _P = (P _{G1} +	· P _{Q1}) × p ₁ = 9 .	. 3 kNm/m				
Saturated retained soil		$M_{sat} = -F_{sat}$	h × X _{sat_h} = -3.7	/ kNm/m				
Water		$M_{water} = -F_{w}$	_{ater_h} × X _{water_h} =	= -12.1 kNm/m				
Moist retained soil		$M_{moist} = -F_m$	$_{oist_h} \times \mathbf{X}_{moist_h} =$	19.1 kNm/m				
Total		M _{total} = M _{ster}	m + M _{base} + M _s	ur + M _P + M _{sat} + M _v	_{vater} + M _{moist} = 3	35.9 kNm/m		
Check bearing pressure								
Propping force		F _{prop_base} = I	= _{total_h} = 39.9 k	N/m				
Distance to reaction		$\overline{x} = M_{total}$ /	F _{total_v} = 874 m	nm				
Eccentricity of reaction		$e = \overline{x} - I_{base}$	∍ / 2 = -291 mr	n				
Loaded length of base		$I_{load} = I_{base} =$	2330 mm					
Bearing pressure at toe		$q_{toe} = F_{total_v}$	/ $I_{base} \times (1 - 6)$	\times e / I _{base}) = 30.9 k	:N/m²			
Bearing pressure at heel		$q_{heel} = F_{total}$	$_v$ / I _{base} × (1 + 6	$6 \times e / I_{base}$) = 4.4 k	xN/m²			
Factor of safety		FoS _{bp} = P _{be}	earing / max(q _{toe} ,	q _{heel}) = 1.942				

RETAINING WALL DESIGN

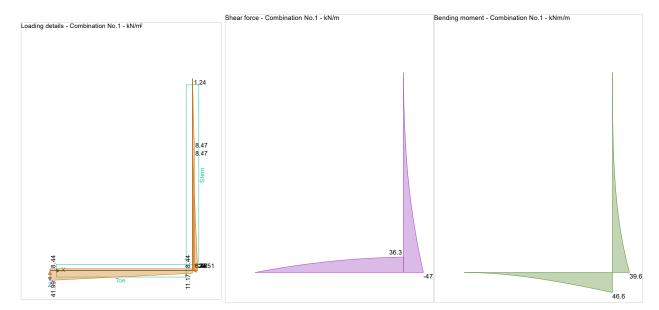
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.12

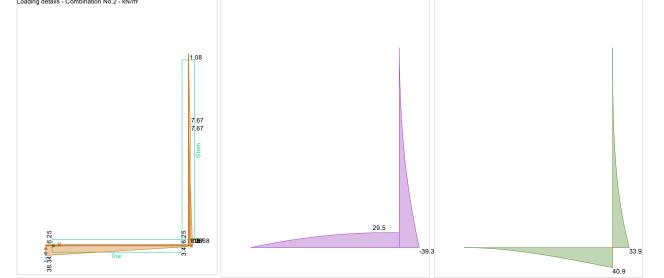
Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

C30/37
f _{ck} = 30 N/mm ²
f _{ck,cube} = 37 N/mm ²
$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 38 \text{ N/mm}^2$
$f_{ctm} = 0.3 \ N/mm^2 \times (f_{ck} \ / \ 1 \ N/mm^2)^{2/3} = \textbf{2.9} \ N/mm^2$
$f_{ctk,0.05}$ = 0.7 × f_{ctm} = 2.0 N/mm ²
E_{cm} = 22 kN/mm ² × (f _{cm} / 10 N/mm ²) ^{0.3} = 32837 N/mm ²
γ _C = 1.50
$\alpha_{cc} = 0.85$
f_{cd} = $\alpha_{cc} \times f_{ck}$ / γ_{C} = 17.0 N/mm ²
h _{agg} = 20 mm

Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL	Project 43a R Calcs for Calcs by SB	EDINGTON ROA B-R Calcs date 14/01/2022	AD, LONDON I W05 Checked by DS	NW3 7RA Checked date 14/01/2022	Start page no./F	1141 Revision 4 B Approved date 14/01/2022
Ultimate strain - Table 3.1 Shortening strain - Table 3.1 Effective compression zone heig Effective strength factor Bending coefficient k ₁ Bending coefficient k ₂ Bending coefficient k ₃ Bending coefficient k ₄	$\varepsilon_{cu2} = 0.0035$ $\varepsilon_{cu3} = 0.0035$ $\lambda = 0.80$ $\eta = 1.00$ $K_1 = 0.40$ $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$ $K_3 = 0.40$ $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$					
 Bending coefficient k₄ Reinforcement details Characteristic yield strength of reinforcement Modulus of elasticity of reinforcement Partial factor for reinforcing steel - Table 2.1N Design yield strength of reinforcement Cover to reinforcement Front face of stem Rear face of stem Top face of base Bottom face of base 		$f_{yk} = 500 \text{ N/}$ $E_s = 20000$ $\gamma_S = 1.15$ $f_{yd} = f_{yk} / \gamma_S$ $c_{sf} = 40 \text{ mm}$ $c_{sr} = 50 \text{ mm}$ $c_{bt} = 50 \text{ mm}$ $c_{bb} = 75 \text{ mm}$	0 N/mm ² = 435 N/mm ² 1 1			



8	SYMMETRYS	Project 43a	REDINGTON RO	Job no. 21141			
	STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision	
	Symmetrys	B-RW05				5 B	
Unit 6 Th	Unit 6 The Courtyard, Lynton Road		Calcs date	Checked by	Checked date	Approved by	Approved date
	London N8 8SL		14/01/2022	DS	14/01/2022	DS	14/01/2022
Loading de	Loading details - Combination No.2 - kN/m ² Shear force - Combination No.2 - kN/m Bending moment - Combination No.2 - kN/m						



Check stem design at base of stem	
Depth of section	h = 330 mm
Rectangular section in flexure - Section 6.1	
Design bending moment combination 1	M = 39.6 kNm/m
Depth to tension reinforcement	d = h - c _{sr} - φ _{sr} / 2 = 272 mm
	$K = M / (d^2 \times f_{ck}) = 0.018$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_{C}) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$
	K' = 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 + 0.5 × (1 - 2 × K / ($\eta \times \alpha_{cc}$ / γ_{C})) ^{0.5} , 0.95) × d = 258 mm
Depth of neutral axis	x = 2.5 × (d – z) = 34 mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) = 352 \text{ mm}^2/\text{m}$
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1340 \text{ mm}^2/\text{m}$
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 410 \text{ mm}^2/\text{m}$
Maximum area of reinforcement - cl.9.2.1.1(3)	A _{sr.max} = 0.04 × h = 13200 mm ² /m
	$max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.306$

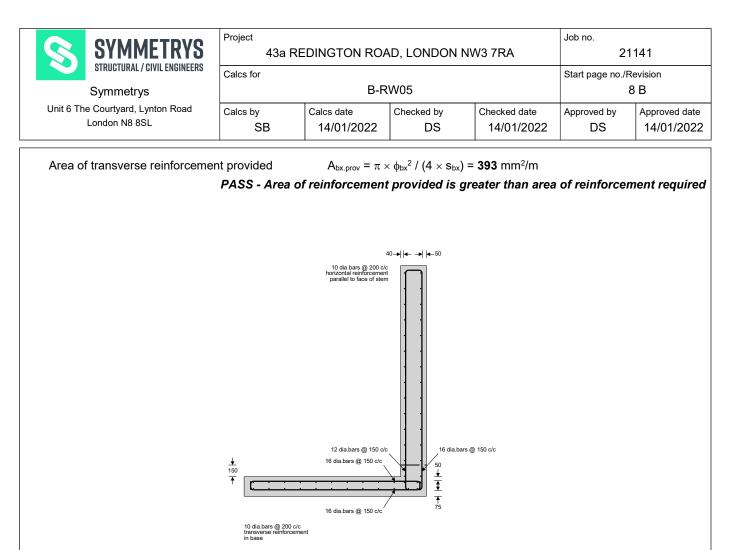
PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

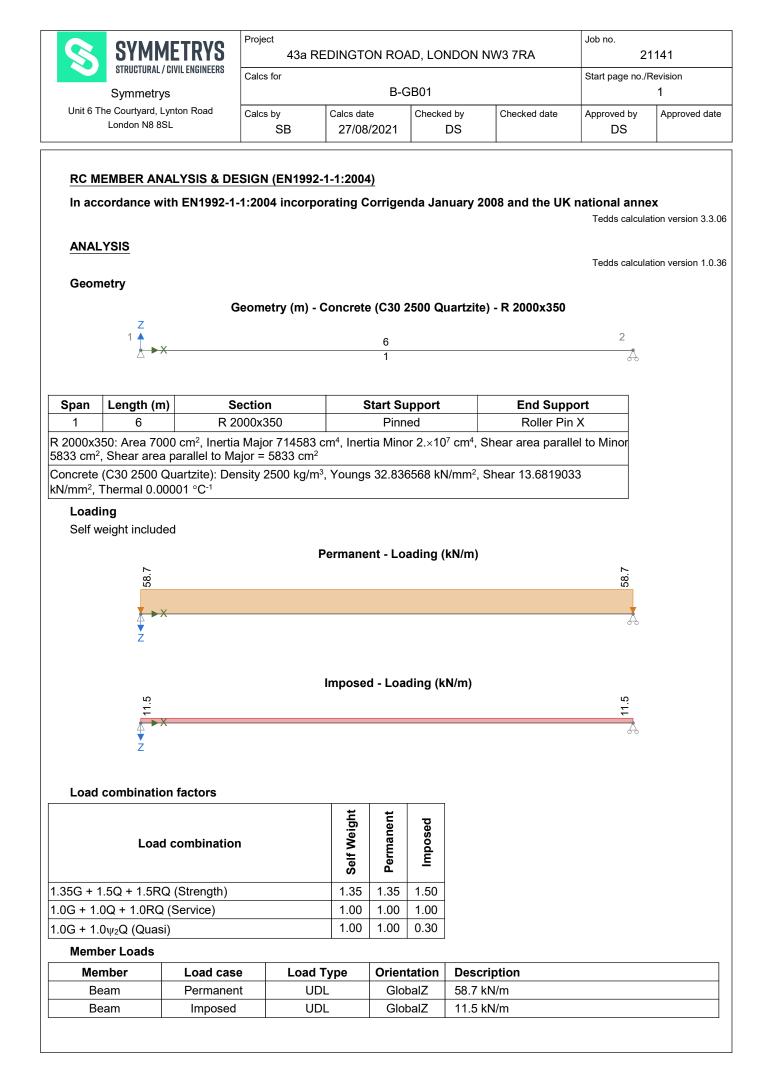
Deflection control - Section 7.4	
Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 N/mm^2) / 1000} = 0.005$
Required tension reinforcement ratio	ρ = A _{sr.req} / d = 0.001
Required compression reinforcement ratio	ρ' = A _{sr.2.req} / d ₂ = 0.000
Structural system factor - Table 7.4N	K _b = 0.4
Reinforcement factor - exp.7.17	$K_s = min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = 1.5$
Limiting span to depth ratio - exp.7.16.a	$min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} \ / \ 1 \ N/mm^2)} \times \rho_0 \ / \ \rho + 3.2 \times \sqrt{(f_{ck} \ / \ 1 \ N/mm^2)})$
	N/mm ²) × (ρ_0 / ρ - 1) ^{3/2}], 40 × K _b) = 16
Actual span to depth ratio	h _{stem} / d = 9.9
	PASS - Span to depth ratio is less than deflection control limit

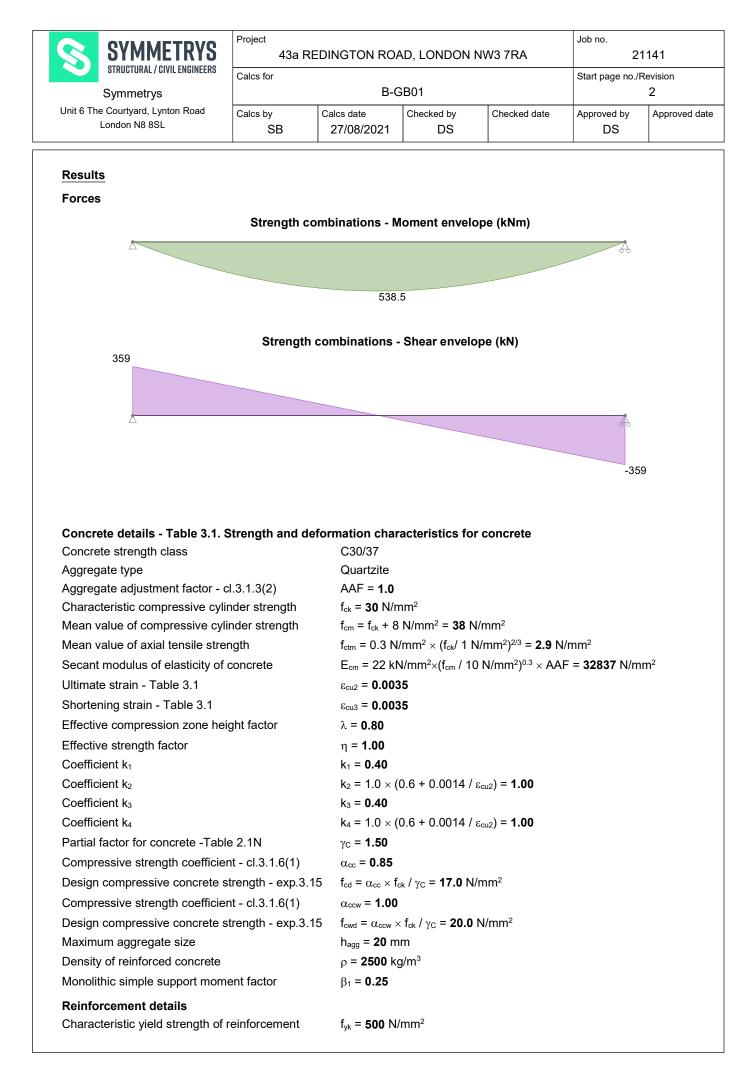
	Project				Job no.			
SYMMETRYS	43a RE	DINGTON RO	2	1141				
STRUCTURAL / CIVIL ENGINEERS	Calcs for	B-F	RW05		Start page no./	Revision 6 B		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 14/01/2022	Checked by DS	Checked date 14/01/2022	Approved by DS	Approved date 14/01/2022		
		1	1			- I		
Crack control - Section 7.3 Limiting crack width		w _{max} = 0.3	mm					
Variable load factor - EN1990 -	Table A1 1							
	Table AT.T	$\psi_2 = 0.6$	kNm/m					
Serviceability bending moment Tensile stress in reinforcement		$M_{sls} = 27.8$	$A_{sr.prov} \times z$) = 80	12 N/mm ²				
Load duration		Long term	Asr.prov × Z) – OU	J.2 IN/IIIII ⁻				
Load duration factor		k _t = 0.4						
Effective area of concrete in ten	sion	•	2.5 × (h - d), (h	(1-x)/(3 h/2)				
	501	$A_{c.eff} = 9860$		(, , , , , , , , , , , , , , , , , , ,				
Mean value of concrete tensile s	trength		2.9 N/mm ²					
Reinforcement ratio	-		_{ov} / A _{c.eff} = 0.01	4				
Modular ratio		$\alpha_{\rm e} = E_{\rm s} / E_{\rm c}$						
Bond property coefficient		k ₁ = 0.8						
Strain distribution coefficient		k ₂ = 0.5						
		k ₃ = 3.4						
		k ₄ = 0.425						
Maximum crack spacing - exp.7	11	$s_{r.max} = k_3 \times$	$c_{sr} \textbf{+} \textbf{k}_1 \times \textbf{k}_2 \times$	$k_4 imes \phi_{sr}$ / $\rho_{p.eff}$ = 37	'0 mm			
Maximum crack width - exp.7.8		w _k = s _{r.max} >	\propto max(σ_s – k _t ×	(f _{ct.eff} / $\rho_{p.eff}$) × (1 +	$-\alpha_{e} \times \rho_{p.eff}$), 0.	$6 imes \sigma_s$) / E _s		
		w _k = 0.089	mm					
		$w_k / w_{max} =$						
Rectangular section in shear	Section 6.2	PASS	- Waximum Ci	rack width is les	s than limitin	ig crack width		
Design shear force	Section 6.2	V = 47 kN/	m					
			3 / γ _C = 0.120					
			√(200 mm / d)	2) = 1.857				
Longitudinal reinforcement ratio				,				
				² × f _{ck} ^{0.5} = 0.485 N	/mm ²			
Design shear resistance - exp.6	2a & 6.2b			$10 \text{ N}^2/\text{mm}^4 \times \rho_1 \times f_0$				
	24 4 0.25	V _{Rd.c} = 148						
		$V / V_{Rd.c} = 0$						
				ear resistance e	xceeds desig	n shear force		
Horizontal reinforcement para	llel to face of s	tem - Section S	9.6					
Minimum area of reinforcement	– cl.9.6.3(1)	A _{sx.req} = ma	$x(0.25 imes A_{sr.prov})$, 0.001 \times t _{stem}) = 3	335 mm²/m			
Maximum spacing of reinforcem	ent – cl.9.6.3(2)	s _{sx_max} = 40	0 mm					
Transverse reinforcement provid	led	10 dia.bars	@ 200 c/c					
Area of transverse reinforcemer	t provided	$A_{sx,prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 393 \text{ mm}^2/\text{m}$						
	PASS - Area of	f reinforcemen	t provided is g	greater than area	of reinforce	ment required		
Check base design at toe								
Depth of section		h = 250 mr	n					
Rectangular section in flexure	- Section 6.1							
Design bending moment combir	ation 1	M = 46.6 k	Nm/m					
Depth to tension reinforcement		d = h - c _{bb} -	- φ _{bb} / 2 = 167 n	nm				
		K = M / (d ²	× f _{ck}) = 0.056					
		K' = (2 × η	× α _{cc} /γ _C)×(1 - λ	\times (δ - K ₁)/(2 \times K ₂)))×(λ × (δ - K ₁)	/(2 × K ₂))		
		K' = 0.207						

	SYMMETRYS	Project 43a RE	DINGTON ROA	AD. LONDON N	IW3 7RA	Job no. 2	1141		
\sim	STRUCTURAL / CIVIL ENGINEERS	Calcs for	Start page no./Revision						
	Symmetrys		B-R	W05		otart pago no.,	7 B		
Unit 6 Th	e Courtyard, Lynton Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved dat		
	London N8 8SL	SB	14/01/2022	DS	14/01/2022	DS	14/01/202		
				K' > K -	No compressior	reinforceme	ent is requir		
Lever a	arm		z = min(0.5		· Κ / (η × α _{cc} / γ _C)		•		
Depth of	of neutral axis		x = 2.5 × (d	l – z) = 22 mm					
Area of	f tension reinforcement r	equired	$A_{bb.req} = M/$	$(f_{yd} \times z) = 676$	mm²/m				
Tensio	n reinforcement provideo	ł	16 dia.bars	@ 150 c/c					
Area of	f tension reinforcement p	provided	$A_{bb.prov} = \pi$	$\times \phi_{bb}^2$ / (4 \times s _{bb})	= 1340 mm²/m				
Minimu	im area of reinforcement	- exp.9.1N	A _{bb.min} = ma	$x(0.26 \times f_{ctm} / f_{y})$	_{/k} , 0.0013) × d = 2	2 52 mm²/m			
Maxim	um area of reinforcemen	t - cl.9.2.1.1(3)	$A_{bb.max} = 0.0$	04 × h = 10000	mm²/m				
			max(A _{bb.req} ,	A _{bb.min}) / A _{bb.prov}	, = 0.504				
		PASS - Area of	reinforcemen	t provided is g	reater than area				
					Lik	orary item: Rectar	ngular single ou		
	control - Section 7.3								
	g crack width		w _{max} = 0.3 I	nm					
	e load factor - EN1990 -	- Table A1.1	ψ ₂ = 0.6						
	eability bending moment		M _{sls} = 34.1		20.0 N1/mm ²				
Load d	e stress in reinforcement		$\sigma_s = M_{sls} / (A_{bb,prov} \times z) = 160.8 \text{ N/mm}^2$						
	uration factor		Long term k _t = 0.4						
	/e area of concrete in ter	sion	-	2.5 × (h - d), (h	-x)/3 h/2)				
LIICOUN		131011	$A_{c.eff} = 7612$. , .	- x) / 3, 11 / 2)				
Mean v	value of concrete tensile	strength	$f_{ct.eff} = f_{ctm} =$						
Reinfor	rcement ratio	U U		ov / A _{c.eff} = 0.01	8				
Modula	ar ratio		$\alpha_{e} = E_{s} / E_{c}$	m = 6.091					
Bond p	roperty coefficient		k ₁ = 0.8						
Strain o	distribution coefficient		k ₂ = 0.5						
			k ₃ = 3.4						
			k ₄ = 0.425						
Maxim	um crack spacing - exp.7	7.11			$k_4 \times \phi_{bb} / \rho_{p.eff} = 4$				
Maxim	um crack width - exp.7.8				$(f_{ct.eff} / \rho_{p.eff}) \times (1 +$	$\alpha_{e} \times \rho_{p.eff}$), 0.	$6 \times \sigma_s) / E_s$		
			w _k = 0.198						
			W _k / W _{max} =		aak width in laa	a than limitir	a araak wi		
_			FA33		ack width is les	5 (11411 11111)	IY CIACK WIG		
	ngular section in shear	- Section 6.2	\/ _ 00 0 \ \	1/22					
Design	shear force		V = 36.3 kM						
				3 / γc = 0.120 · √(200 mm / d)	2) - 2 000				
Longit	idinal rainforcoment refi	, ,		. ,	-				
Longitt	udinal reinforcement ratio	,		$_{0.prov} / d, 0.02) =$	v.008 × f _{ck} ^{0.5} = 0.542 N	/mm ²			
Decian	shear resistance - exp.6	3 2 2 8 6 2 h							
มสราชท	shear resistance - exp.t).2a & U.2D	V _{Rd.c} = max V _{Rd.c} = 115		$0 \text{ N}^2/\text{mm}^4 \times \rho_1 \times f_0$	ck/∵, vmin/× O			
			$V_{Rd.c} - 113$ V / V _{Rd.c} = (
					ear resistance e	xceeds desid	an shear foi		
Secon	dary transverse reinfor	cement to base		<u>.</u>					
	im area of reinforcement			× A _{bb.prov} = 268	mm²/m				
	um spacing of reinforcen	. ,	-						
waximi									

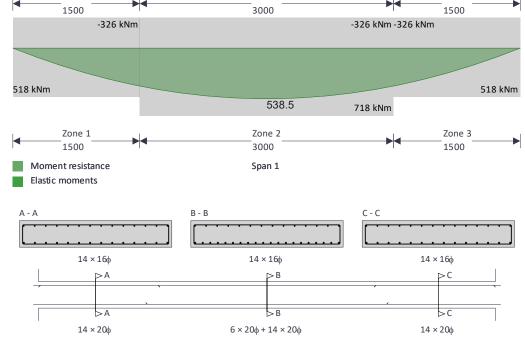


Reinforcement details





	Project 43a RF	EDINGTON ROA	Job no. 21141					
STRUCTURAL / CIVIL ENGINEERS			D, LONDON I					
	Calcs for		DOA		Start page no./			
Symmetrys			B01			3		
Unit 6 The Courtyard, Lynton Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
London N8 8SL	SB	27/08/2021	DS		DS			
Partial factor for reinforcing stee	l - Table 2.1N	γs = 1.15						
Design yield strength of reinforc	ement	$f_{yd} = f_{yk} / \gamma_S$	= 435 N/mm ²					
Nominal cover to reinforceme	nt							
Nominal cover to top reinforcem	ent	c _{nom_t} = 50 r	c _{nom_t} = 50 mm					
Nominal cover to bottom reinfore	cement	C _{nom_b} = 35	c _{nom_b} = 35 mm					
Nominal cover to side reinforcer	nent	C _{nom_s} = 35	mm					
Fire resistance								
Standard fire resistance period		R = 60 min						
Number of sides exposed to fire		3						
Minimum width of beam - EN19	92-1-2 Table 5.5	b _{min} = 120 r	nm					
Beam - Span 1								
Rectangular section details								
Section width		b = 2000 mm						
Section depth		h = 350 mm						
			PASS - M	inimum dimens	sions for fire r	esistance m		
Moment design								
Zone 1		Zone 2		Zone 3	3			



Zone 1 (0 mm - 1500 mm) Positive moment - section 6.1

Design bending moment	M = abs(M _{m1_s1_z1_max_red}) = 403.9 kNm
Effective depth of tension reinforcement	d = 293 mm
Redistribution ratio	$\delta = \min(M_{\text{pos}_red_z1} / M_{\text{pos}_z1}, 1) = 1.000$
	$K = M / (b \times d^2 \times f_{ck}) = 0.078$
	$K' = (2 \times \eta \times \alpha_{\mathtt{cc}} \ / \ \gamma_{\mathtt{C}}) \times (1 - \lambda \times (\delta - k_1) \ / \ (2 \times k_2)) \times (\lambda \times (\delta - k_1) \ / \ (2 \times k_2))$
	= 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 × d × [1 + (1 - 2 × K / ($\eta \times \alpha_{cc}$ / γ_{C})) ^{0.5}], 0.95 × d) = 271 mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = 55 mm$

SYMMETRYS	Project 43a RE		AD, LONDON I	NW3 7RA	Job no. 2'	1141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for			Start page no./Revision				
Symmetrys		B-0	GB01			4		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da		
Area of tension reinforcement re	equired	$A_{s,req} = M /$	(f _{yd} × z) = 3426	mm ²				
Tension reinforcement provided		$14\times 20\varphi$						
Area of tension reinforcement p	rovided	A _{s,prov} = 43	98 mm ²					
Minimum area of reinforcement	- exp.9.1N	A _{s,min} = max	$x(0.26 imes f_{ctm} \ / \ f_{yt})$, 0.0013) $ imes$ b $ imes$ d	d = 883 mm²			
Maximum area of reinforcement	t - cl.9.2.1.1(3)	$A_{s,max} = 0.0$	4 × b × h = 280	000 mm²				
	PASS - Area of	f reinforcement	t provided is g	greater than are	a of reinforce	ment requi		
Crack control - Section 7.3								
Maximum crack width		w _k = 0.3 mr	m					
Design value modulus of elastic	ity reinf – 3.2.7(4	e) E _s = 20000	0 N/mm ²					
Mean value of concrete tensile s	strength	$f_{ct,eff} = f_{ctm} =$	2.9 N/mm ²					
Stress distribution coefficient		k _c = 0.4						
Non-uniform self-equilibrating st	ress coefficient	k = min(ma	ıx(1 + (300 mm	- min(h, b)) × 0.	.35 / 500 mm, 0	0.65), 1) = 0		
Actual tension bar spacing		s _{bar} = (b - (2	$2 \times (c_{nom_s} + \phi_m)$	1_s1_z1_v) +	$_{z1_b_{L1}} \times N_{m1_{s1_z}}$	z1_b_L1)) /		
		(N _{m1_s1_z1_b_}	_∟1 - 1) + φ _{m1_s1_}	_{z1_b_L1} = 145.1 m	m			
Maximum stress permitted - Tak	ole 7.3N	σs = 284 N/	/mm²					
Steel to concrete modulus of ela	ast. ratio	$\alpha_{cr} = E_s / E_s$	_{cm} = 6.09					
Distance of the Elastic NA from	bottom of beam	$y = (b \times h^2)$	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$					
		171 mm						
Area of concrete in the tensile z	one	$A_{ct} = b \times y =$	= 342685 mm ²					
Minimum area of reinforcement	required - exp.7.	1 $A_{sc,min} = k_c$	$\times \ k \times f_{\text{ct,eff}} \times A_{\text{ct}}$	/ σ _s = 1349 mm²				
PASS	- Area of tensio	n reinforceme	nt provided ex	ceeds minimur	m required for	crack con		
Quasi-permanent moment		M _{QP} = max	$(\beta_1 \times abs(M_{m1_s}))$	1_z2_neg_quasi), abs	(M _{m1_s1_z1_pos_qua}	_{asi})) =		
		267.7 kNm						
Permanent load ratio		$R_{PL} = M_{QP}$ /	M = 0.66					
Service stress in reinforcement		$\sigma_{sr} = f_{yd} \times A$	$A_{s,req}$ / $A_{s,prov} \times R$	_{PL} = 225 N/mm ²				
Maximum bar spacing - Tables		S _{bar,max} = 21						
	PAS	S - Maximum k	oar spacing ex	ceeds actual b	ar spacing for	crack con		
Zone 1 (0 mm - 1500 mm) Neg	ative moment -	section 6.1						
Design bending moment		M = max(β·	$_1 \times abs(M_{m1_s1_r})$	_{max_red}), abs(M _{m1_}	_s1_z1_min_red)) = 1	34.6 kNm		
Effective depth of tension reinfo	rcement	d = 280 mn	n					
Redistribution ratio		δ = 1 = 1.0	00					
		K = M / (b >	$\times d^2 \times f_{ck}$) = 0.0	29				
		K' = (2 × η	$\times \alpha_{cc} / \gamma_{C}) \times (1 + 1)$	- λ × (δ - k ₁) / (2	\times k ₂)) \times (λ \times (δ	- k1) / (2 × k		
		= 0.207						
			K' > K -	No compressio	on reinforceme	ent is requi		
Lever arm		z = min(0.5	$5 \times d \times [1 + (1 -$	$2\times K$ / ($\eta\times\alpha_{cc}$ /	′ γc)) ^{0.5}], 0.95 ×	d) = 266 m		
Depth of neutral axis		x = 2 × (d -	z) / λ = 35 mm	1				
Area of tension reinforcement re	equired	$A_{s,req} = M /$	(f _{yd} × z) = 1164	mm²				
Tension reinforcement provided		$14\times16\phi$						
Area of tension reinforcement p	rovided	A _{s,prov} = 28 1	15 mm²					
Minimum area of reinforcement	- exp.9.1N	A _{s,min} = max	$x(0.26 \times f_{ctm} / f_{yt})$	k, 0.0013) $ imes$ b $ imes$ o	d = 843 mm²			
Maximum area of reinforcement	t - cl.9.2.1.1(3)	$A_{s,max} = 0.0$	4 × b × h = 280)00 mm²				
	D400	f reinforcemen	t provided is a	reater than are	a of reinforce	ment requi		
	PASS - Area of					-		
Crack control - Section 7.3	PASS - Area of		, p			-		

SYMMETRYS	Project 43a RE	EDINGTON ROA	AD, LONDON I	NW3 7RA	Job no. 2	1141			
STRUCTURAL / CIVIL ENGINEERS	Calcs for			Start page no./Revision					
Symmetrys		B-0	GB01			5			
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da			
Design value modulus of elastici	ty reinf – 3.2.7(4	4) E _s = 20000	0 N/mm ²						
Mean value of concrete tensile s	trength	$f_{ct,eff} = f_{ctm} =$	2.9 N/mm ²						
Stress distribution coefficient		k _c = 0.4							
Non-uniform self-equilibrating st	ress coefficient	k = min(ma	x(1 + (300 mm	ı - min(h, b)) × 0.	.35 / 500 mm, 0	0.65), 1) = 0			
Actual tension bar spacing		s _{bar} = (b - (2	$2 \times (c_{nom_s} + \phi_m)$	1_s1_z1_v) +	$_{z1_t_L1} \times N_{m1_s1_z}$	1_t_L1)) /			
		(N _{m1_s1_z1_t_}	L1 - 1) + φm1_s1_z	₂1_t_L1 = 145.4 mr	n				
Maximum stress permitted - Tab	le 7.3N	σs = 284 N/	′mm²						
Steel to concrete modulus of ela	st. ratio	$\alpha_{cr} = E_s / E_s$	_{cm} = 6.09						
Distance of the Elastic NA from	bottom of beam	y = (b × h² , 173 mm	/ 2 + A _{s,prov} × (o	u _{cr} - 1) × (h - d)) /	$(b \times h + A_{s,prov})$	× (a _{cr} - 1)) =			
Area of concrete in the tensile zo	one	$A_{ct} = b \times y =$	= 345787 mm ²						
Minimum area of reinforcement	required - exp.7	.1 $A_{sc,min} = k_c$	$\times \mathbf{k} imes \mathbf{f}_{ct,eff} imes \mathbf{A}_{ct}$	/ σ _s = 1363 mm²	!				
PASS	Area of tensio	on reinforceme	nt provided ex	ceeds minimu	m required for	crack con			
Quasi-permanent moment		M _{QP} = max	$(\beta_1 \times abs(M_{m1_s}))$	1_z2_pos_quasi), abs	(M _{m1_s1_z1_neg_qu}	_{asi})) =			
		89.2 kNm							
Permanent load ratio		$R_{PL} = M_{QP}$ /	M = 0.66						
Service stress in reinforcement		σ_{sr} = f _{yd} × A	σ_{sr} = f _{yd} × A _{s,req} / A _{s,prov} × R _{PL} = 119 N/mm ²						
Maximum bar spacing - Tables 7	7.3N	S _{bar,max} = 30	0 mm						
	PAS	SS - Maximum b	oar spacing ex	ceeds actual b	ar spacing for	crack con			
Minimum bar spacing (Section	า 8.2)								
Top bar spacing			2 × (c _{nom_s} + φ _m . _{L1} - 1) = 129.4 μ	1_s1_z1_v) +	$_{z1_t_L1} imes N_{m1_s1_z}$	1_t_L1)) /			
Minimum allowable top bar spac	ing	s _{top,min} = ma		≺ k _{s1} , h _{agg} + k _{s2} , 2 al bar spacing e					
Bottom bar spacing			2 × (C _{nom_s} + φ _m . _{L1} - 1) = 125.1	1_s1_z1_v) + φ _{m1_s1_} ; mm	$_{z1_b_{L1}} \times N_{m1_{s1_{s1_{s1_{s1_{s1_{s1_{s1_{s1_{s1_{s$	21_b_L1)) /			
Minimum allowable bottom bar s	pacing			× k _{s1} , h _{agg} + k _{s2} , ź	20mm) = 25.0 ı	mm			
	F3	- bot, min		al bar spacing e					
Zone 2 (1500 mm - 4500 mm) F	Positive mome								
Design bending moment			m1_s1_z2_max_red) =	= 538.5 kNm					
Effective depth of tension reinfor	cement	d = 293 mn		A)					
Redistribution ratio			os_red_z2 / Mpos_z2	-					
			$\langle d^2 \times f_{ck} \rangle = 0.1$						
		K' = (2 × η = 0.207	$\times \alpha_{cc} / \gamma_{C}) \times (1)$	- λ × (δ - k ₁) / (2	\times k ₂)) \times (λ \times (δ	- k ₁) / (2 × k			
			K' > K -	No compressio	on reinforceme	ent is requi			
Lever arm		z = min(0.5	× d × [1 + (1 -	$2 \times K$ / ($\eta \times \alpha_{cc}$ /	′ γ _C)) ^{0.5}], 0.95 ×	d) = 263 mi			
Depth of neutral axis		x = 2 × (d -	z) / λ = 75 mm	ı					
Area of tension reinforcement re	quired	$A_{s,req} = M /$	(f _{yd} × z) = 4711	mm ²					
Tension reinforcement provided		6 × 20φ + 1	$4 imes 20 \phi$						
Area of tension reinforcement pr	ovided	A _{s,prov} = 628	33 mm²						
Minimum area of reinforcement	- exp.9.1N	A _{s,min} = max	$\kappa(0.26 \times f_{ctm} \ / \ f_{y})$	_k , 0.0013) × b × 0	d = 883 mm²				
Maximum area of reinforcement	- cl.9.2.1.1(3)	$A_{s,max} = 0.0$	4 × b × h = 280)00 mm²					
	PASS - Area o								

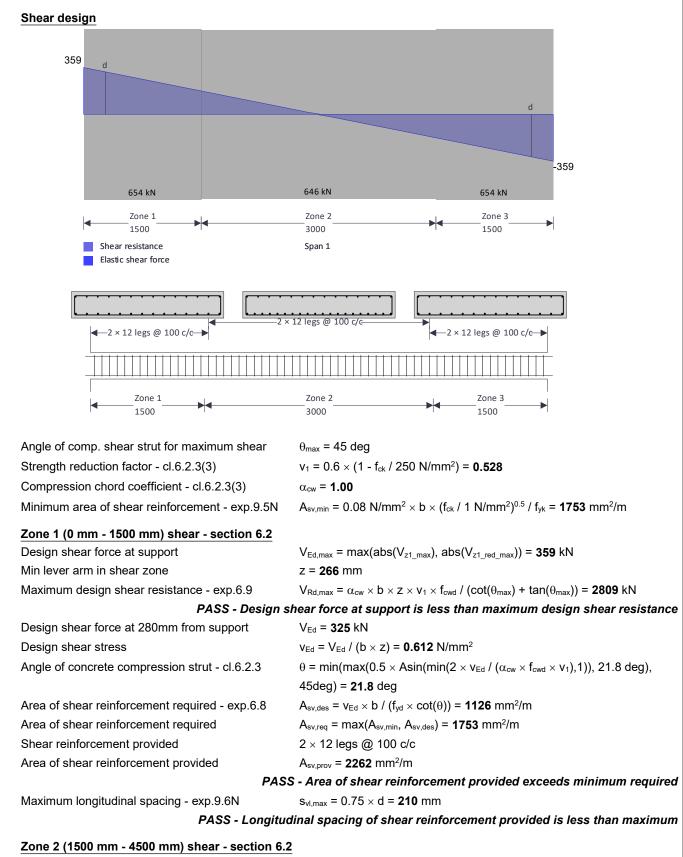
SYMMETRYS	Project 43a RI	EDINGTON RO	AD, LONDON I	NW3 7RA	Job no.	1141	
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision		
Symmetrys		B-C	GB01			6	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date	
Crack control - Section 7.3							
Maximum crack width		w _k = 0.3 mi	m				
Design value modulus of elastic	ity reinf – 3.2.7(
Mean value of concrete tensile			2.9 N/mm ²				
Stress distribution coefficient		k _c = 0.4					
Non-uniform self-equilibrating st	ress coefficient	k = min(ma	ax(1 + (300 mm	$h - min(h, b)) \times 0.$	35 / 500 mm, 0	.65), 1) = 0.9	
Actual tension bar spacing		s _{bar} = (b - (2	$2 \times (c_{nom_s} + \phi_m)$	1_s1_z2_v) +	$_{z2_b_{L1}} \times N_{m1_{s1_z}}$	2_b_L1 +	
		фт1_s1_z1_b_L	$_1 \times N_{m1_s1_z1_b_c}$	1)) / ((N _{m1_s1_z2_b_l}	_1 + N _{m1_s1_z1_b_L}	.1) - 1) +	
		фm1_s1_z2_b_L	₁ = 99.3 mm				
Maximum stress permitted - Tal	ole 7.3N	σ _s = 321 N					
Steel to concrete modulus of ela		$\alpha_{cr} = E_s / E$	_{cm} = 6.09				
Distance of the Elastic NA from	bottom of beam			ι _{cr} - 1) × (h - d)) /	$(b \times h + A_{s.prov})$	× (α _{cr} - 1)) =	
		170 mm		, , , ,,	X 4	(
Area of concrete in the tensile z	one	$A_{ct} = b \times y$	= 339687 mm ²				
Minimum area of reinforcement	required - exp.7	-					
				ceeds minimur		crack contro	
Quasi-permanent moment				_{si}) = 356.9 kNm			
Permanent load ratio		$R_{PL} = M_{QP}$	M = 0.66				
Service stress in reinforcement		σ_{sr} = f _{yd} × A	$A_{s,req}$ / $A_{s,prov} imes R$	R _{PL} = 216 N/mm ²			
Maximum bar spacing - Tables	7.3N	S _{bar,max} = 22	:9.9 mm				
	PAS	SS - Maximum b	oar spacing ex	ceeds actual ba	ar spacing for	crack contro	
Deflection control - Section 7.	.4						
Reference reinforcement ratio		$\rho_{m0} = (f_{ck} / T)$	1 N/mm ²) ^{0.5} / 10	000 = 0.00548			
Required tension reinforcement	ratio	$\rho_{m} = A_{s,req} /$	(b × d) = 0.008	304			
Required compression reinforce	ement ratio	ρ' _m = A _{s2,req}	/ (b × d) = 0.00	0000			
Structural system factor - Table	7.4N	K _b = 1.0	. ,				
Basic allowable span to depth ra	atio	span_to_de	$epth_{basic} = K_b \times$	[11 + 1.5 × (f _{ck} /	1 N/mm²) ^{0.5} × ρ	_{m0} / (ρ _m - ρ' _m)	
				ρ _{m0}) ^{0.5} / 12] = 16			
Reinforcement factor - exp.7.17			,	00 N/mm² / f _{yk} , 1.			
Flange width factor		F1 = 1 = 1 .					
Long span supporting brittle par	tition factor	F2 = 1 = 1 .	000				
Allowable span to depth ratio			epth _{allow} = min(s	span_to_depth _{bas}	$_{sic} imes K_s imes F1 imes F$	2, 40 × K_b) =	
Astrological IIIII		22.134					
Actual span to depth ratio			epth _{actual} = L _{m1_s}		ic within the -	llowable lim	
		PASS	s - Actual spal	n to depth ratio	is within the a	nowable lim	
Minimum bar spacing (Sectio	n 8.2)						
Top bar spacing			2 × (c _{nom_s} + φ _m . _{L1} - 1) = 129.4 Ι	$f_{1_{s1_{z2_v}}} + \phi_{m1_{s1_{z2}}}$ mm	$_{z2_{L}L1} \times N_{m1_{s1_{z2}}}$	2_t_L1)) /	
Minimum allowable top bar space	cing	s _{top,min} = ma		× k _{s1} , h _{agg} + k _{s2} , 2 al bar spacing e			
Bottom bar spacing		s _{bot} = (b - (2		1_s1_z2_v) + φm1_s1_z			
Dottorn bar opaoing				1)) / ((N _{m1_s1_z2_b_l}			
Bottom bal opaoling		mm					
Minimum allowable bottom bar	spacing	mm _{Shot min} = ma	aX(dm1 e1 -2 b 14	\times k _{s1} , h _{agg} + k _{s2} , 2	20mm) = 25 0 r	nm	

SYMMETRYS		Project 43a R	EDINGTON ROA	Job no. 21141			
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Unit 6 Th	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

Design bending moment	M = abs(M _{m1_s1_z3_max_red}) = 403.9 kNm
Effective depth of tension reinforcement	d = 293 mm
Redistribution ratio	$\delta = \min(M_{\text{pos}_{red}_{z3}} / M_{\text{pos}_{z3}}, 1) = 1.000$
	$K = M / (b \times d^2 \times f_{ck}) = 0.078$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_{C}) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$
	= 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 × d × [1 + (1 - 2 × K / ($\eta \times \alpha_{cc}$ / γ_c)) ^{0.5}], 0.95 × d) = 271 mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = 55 mm$
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = 3426 \text{ mm}^2$
Tension reinforcement provided	$14 imes 20 \phi$
Area of tension reinforcement provided	A _{s,prov} = 4398 mm ²
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = max(0.26 \times f_{ctm} \ / \ f_{yk}, \ 0.0013) \times b \times d = \textbf{883} \ mm^2$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = 28000 \text{ mm}^2$
PASS - Area of re	einforcement provided is greater than area of reinforcement required
Crack control - Section 7.3	
Maximum crack width	w _k = 0.3 mm
Design value modulus of elasticity reinf $-3.2.7(4)$	E _s = 200000 N/mm ²
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = 2.9 \text{ N/mm}^2$
Stress distribution coefficient	k _c = 0.4
Non-uniform self-equilibrating stress coefficient	k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = 0.97
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) /$
	(N _{m1_s1_z3_b_L1} - 1) + φ _{m1_s1_z3_b_L1} = 145.1 mm
Maximum stress permitted - Table 7.3N	σ_s = 284 N/mm ²
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = 6.09$
Distance of the Elastic NA from bottom of beam	y = (b × h ² / 2 + A _{s,prov} × (α_{cr} - 1) × (h - d)) / (b × h + A _{s,prov} × (α_{cr} - 1)) =
	171 mm
Area of concrete in the tensile zone	A _{ct} = b × y = 342685 mm ²
Minimum area of reinforcement required - exp.7.1	$A_{sc,min}$ = $k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = 1349 mm ²
PASS - Area of tension	reinforcement provided exceeds minimum required for crack control
Quasi-permanent moment	$M_{QP} = max(\beta_1 \times abs(M_{m1 s1 z2 neg quasi}), abs(M_{m1 s1 z3 pos quasi})) =$
	267.7kNm
Permanent load ratio	$R_{PL} = M_{QP} / M = 0.66$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \textbf{225} \text{ N/mm}^2$
Maximum bar spacing - Tables 7.3N	s _{bar,max} = 219.4 mm
PASS	Maximum bar spacing exceeds actual bar spacing for crack control
Zone 3 (4500 mm - 6000 mm) Negative moment	- section 6.1
Design bending moment	$M = \max(\beta_1 \times abs(M_{m1_s1_max_red}), abs(M_{m1_s1_z3_min_red})) = 134.6 \text{ kNm}$
Effective depth of tension reinforcement	d = 280 mm
Redistribution ratio	$\delta = 1 = 1.000$
	$K = M / (b \times d^2 \times f_{ck}) = 0.029$
	$\mathbf{K'} = (2 \times \eta \times \alpha_{cc} / \gamma_{C}) \times (1 - \lambda \times (\delta - k_{1}) / (2 \times k_{2})) \times (\lambda \times (\delta - k_{1}) / (2 \times k_{2}))$
	= 0.207
	K' > K - No compression reinforcement is required

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	Symmetrys	Calcs for	В	-GB01		Start page no./	Start page no./Revision 8			
	e Courtyard, Lynton Road ∟ondon N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved			
Lever a	rm		z = min(0	.5 × d × [1 + (1 -	\cdot 2 × K / (η × α_{cc} /	[/] γc)) ^{0.5}], 0.95 ×	d) = 266 m			
Depth o	f neutral axis		x = 2 × (d	- z) / λ = 35 mn	n					
Area of	tension reinforcement	required	$A_{s,req} = M$	/ (f _{vd} × z) = 116 4	4 mm ²					
	n reinforcement provide	-	$14 \times 16\phi$,						
	tension reinforcement		$A_{s,prov} = 2$	815 mm²						
	m area of reinforcemer	-			_{/k} , 0.0013) × b × 0	d = 843 mm²				
	Im area of reinforceme	-		.04 × b × h = 28						
maxima		()			greater than are	a of reinforce	ment requ			
. .		TAGE AREA			greater than are		inchi i cyu			
	control - Section 7.3									
	m crack width		w _k = 0.3 r							
•	value modulus of elast	•	()	00 N/mm ²						
	alue of concrete tensile	e strength		= 2.9 N/mm ²						
	distribution coefficient		k _c = 0.4	(4 (222		05 / 500				
	iform self-equilibrating	stress coefficient		$k = min(max(1 + (300 mm - min(h, b)) \times 0.35 / 500 mm, 0.65), 1) = 0$						
Actual to	ension bar spacing			$\mathbf{s}_{bar} = (\mathbf{b} - (2 \times (\mathbf{c}_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times \mathbf{N}_{m1_s1_z3_t_L1})) / $						
			(N _{m1_s1_z3_}	<u>t_L1</u> - 1) + φ _{m1_s1_}	_{z3_t_L1} = 145.4 mr	n				
Maximu	m stress permitted - T	able 7.3N	σs = 284	N/mm ²						
Steel to	concrete modulus of e	elast. ratio	$\alpha_{cr} = E_s /$	E _{cm} = 6.09						
Distanc	e of the Elastic NA fror	n bottom of bean	$y = (b \times h)$	2 / 2 + A _{s,prov} × (a	α _{cr} - 1) × (h - d)) /	$(b \times h + A_{s,prov})$	\times (α_{cr} - 1))			
			173 mm							
Area of	concrete in the tensile	zone	$A_{ct} = b \times y$	y = 345787 mm ²	2					
Minimur	m area of reinforcemer	nt required - exp.	7.1 A _{sc,min} = k	$_{c} imes k imes f_{ct,eff} imes A_{ct}$	/ σ _s = 1363 mm ²					
	PAS	S - Area of tensi	on reinforcem	ent provided e	xceeds minimui	m required for	r crack con			
Quasi-p	ermanent moment		M _{QP} = ma	$\mathbf{x}(\beta_1 \times \mathbf{abs}(\mathbf{M}_{m1}))$	s1_z2_pos_quasi), abs	(M _{m1_s1_z3_neg_qu}	_{iasi})) =			
			89.2 kNm							
Perman	ent load ratio		$R_{PL} = M_{QF}$	⊳ / M = 0.66						
Service	stress in reinforcemer	nt	σ_{sr} = f _{yd} ×	$A_{s,req} / A_{s,prov} \times F$	R _{PL} = 119 N/mm ²					
Maximu	im bar spacing - Table	s 7.3N	-	s _{bar,max} = 300 mm						
					xceeds actual b	ar spacing for	r crack con			
Minimu	ım bar spacing (Secti									
	spacing	··· ···	e, - (h	(2 × (c + *	1_s1_z3_v) + \$\phi_m1_s1_1		a () () (
ioh ngi	эраону			(2 × (C _{nom_s} + φ _m _{t_L1} - 1) = 129.4		zʒ_t_L1 × I№m1_s1_ż	z3_t_L1 <i>)) /</i>			
Minimur	m allowable top bar sp	acing	s _{top,min} = n		× k _{s1} , h _{agg} + k _{s2} , 2 al bar spacing e					
Bottom	bar spacing			(2 × (c _{nom_s} + φ _m _{b_L1} - 1) = 125.1	_{1_s1_z3_v}) + φ _{m1_s1_} mm	$_{z3_b_{L1}} imes N_{m1_{s1_{c1}}}$	_{z3_b_L1})) /			
Minimur	m allowable bottom ba	r spacing			\times k _{s1} , h _{agg} + k _{s2} , 2	20mm) = 25.0	mm			

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Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date



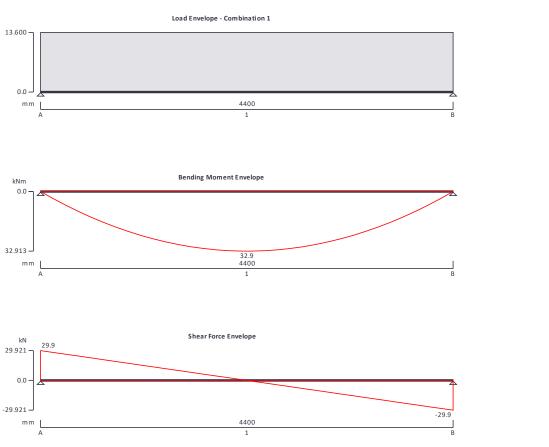
 $V_{Ed,max} = max(abs(V_{z2_max}), abs(V_{z2_red_max})) = 179 \text{ kN}$

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STRUCTURAL / CIVIL E	NGINEERS	Calcs for				Start page no./Revision			
Symmetrys			B-G	GB01		otart page no./i	10		
Unit 6 The Courtyard, Lynton	Road	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved da		
London N8 8SL		SB	27/08/2021	DS		DS			
Min lever arm in shear	zone		z = 263 mm	ı					
Maximum design shea	r resistanc	e - exp.6.9	$V_{Rd,max} = \alpha_c$	$_{w} \times b \times z \times v_{1} \times v_{1}$	f_{cwd} / (cot(θ_{max}) -	+ tan(θ _{max})) = 2	776 kN		
		PASS - Desig	n shear force a	t support is le	ess than maxim	um design sh	ear resistar		
Design shear force with	hin zone		V _{Ed} = 179 k	N					
Design shear stress			$v_{Ed} = V_{Ed} / ($	(b × z) = 0.341	N/mm ²				
Angle of concrete com	pression s	trut - cl.6.2.3	$\theta = \min(\max$	x(0.5 imes Asin(m	iin(2 × v _{Ed} / (α_{cw} :	× f_{cwd} × v_1),1)), 2	21.8 deg),		
			45deg) = 2	1.8 deg					
Area of shear reinforce	ement requ	ired - exp.6.8	$A_{sv,des} = v_{Ed}$	imes b / (f _{yd} $ imes$ cot	(θ)) = 628 mm²/n	า			
Area of shear reinforce	ement requ	ired	A _{sv,req} = ma	x(A _{sv,min} , A _{sv,des}) = 1753 mm²/m				
Shear reinforcement pr	rovided		2 × 12 legs @ 100 c/c						
Area of shear reinforce	ement prov	ided	A _{sv,prov} = 22	62 mm²/m					
		PA	ASS - Area of si	hear reinforce	ement provided	exceeds mini	mum requi		
Maximum longitudinal	spacing - e	exp.9.6N	$s_{vl,max} = 0.7$	5 × d = 220 mr	n				
		PASS - Longi	tudinal spacing	g of shear reir	nforcement prov	vided is less ti	han maxim		
Zone 3 (4500 mm - 60	00 mm) s	hear - section	6.2						
Design shear force at s	support		V _{Ed,max} = ma	ax(abs(V _{z3_max})	, abs(V _{z3_red_max}))	= 359 kN			
Min lever arm in shear	zone		z = 266 mm	า					
Maximum design shea	r resistanc	e - exp.6.9	$V_{Rd,max} = \alpha_c$	$_{w} \times b \times z \times v_{1} \times v_{1}$	f_{cwd} / (cot(θ_{max}) -	+ tan(θ _{max})) = 2	809 kN		
		PASS - Desig	n shear force a	t support is le	ess than maxim	um design sh	ear resistai		
Design shear force at 2	280mm fro	m support	V _{Ed} = 325 k	N					
Design shear stress			$v_{Ed} = V_{Ed} / ($	b × z) = 0.612	N/mm ²				
Angle of concrete com	pression s	trut - cl.6.2.3	θ = min(max(0.5 × Asin(min(2 × v _{Ed} / ($\alpha_{cw} × f_{cwd} × v_1$),1)), 21.8 deg),						
			45deg) = 2	1.8 deg					
Area of shear reinforce	ement requ	ired - exp.6.8	$A_{sv,des} = v_{Ed}$	\times b / (f_{yd} \times cot	(θ)) = 1126 mm²/	'n			
Area of shear reinforce	ement requ	ired	A _{sv,req} = ma	$x(A_{sv,min}, A_{sv,des})$) = 1753 mm²/m				
Shear reinforcement pr	rovided		2×12 legs	@ 100 c/c					
Area of shear reinforce	ement prov	ided	A _{sv,prov} = 22	62 mm²/m					
		PA	ASS - Area of s	hear reinforce	ement provided	exceeds mini	mum requi		
Maximum longitudinal	spacing - e	exp.9.6N	$s_{vl,max} = 0.75$	5 × d = 210 mr	n				
		PASS - Longi	tudinal spacing	g of shear reir	forcement prov	vided is less ti	han maxim		

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	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for	G-G	SB01		Start page no./R	evision 1
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

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



Support conditions		
Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	
Applied loading		
Beam loads	Permanent self weight o	of beam $ imes$ 1
	Ground floor. 1KN/m2 x	4.8m/2 - Permanent full UDL 2.4 kN/m
	Ground floor + partitions	s. 2.7KN/m2 x 4.8m/2 - Variable full UDL 6.5
	kN/m	
Load combinations		
Load combination 1	Support A	Permanent × 1.35
		Variable \times 1.50
		Permanent × 1.35
		Variable \times 1.50
	Support B	Permanent × 1.35

TEDDS calculation version 3.0.14

SYMMETRYS STRUCTURAL / CIVIL ENGINEERS		EDINGTON RO	21141			
Symmetrys	Calcs for	G-G	SB01		Start page no./Revis	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved dat
		<u> </u>		Variat	le × 1.50	
Analysis results					0 1	
Maximum moment Maximum shear		M _{max} = 32.9 V _{max} = 29.9			0 kNm -29.9 kN	
Deflection		ν _{max} – 29.9 δ _{max} = 4.8 r		ν _{min} – δ _{min} =		
Maximum reaction at support A		$R_{A max} = 29$			= 29.9 kN	
Unfactored permanent load read	tion at support <i>i</i>	_				
Unfactored variable load reaction		R _{A_Variable} =				
Maximum reaction at support B		 R _{B_max} = 29		$R_{B_{min}}$	= 29.9 kN	
Unfactored permanent load read	tion at support l	B R _{B_Permanent}	= 6.3 kN			
Unfactored variable load reaction	n at support B	R _{B_Variable} =	14.3 kN			
Section details						
Section type		UC 203x20)3x46 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled p	roducts of stru	ictural steels				
Nominal thickness of element		•	t _w) = 11.0 mm			
Nominal yield strength		f _y = 355 N/r				
Nominal ultimate tensile strength	ו	f _u = 470 N/				
Modulus of elasticity		E = 210000	0 N/mm ²			
203.2	- 1	+	-7.2			
	◀────	203.6				
Partial factors - Section 6.1						
Resistance of cross-sections		γ _{M0} = 1.00				
Resistance of members to instal	oility	γ _{M1} = 1.00				
Resistance of tensile members t	o fracture	γ _{M2} = 1.10				
Lateral restraint						
		Span 1 has	s lateral restrai	nt at supports on	ly	
Effective length factors						
Effective length factor in major a	xis	K _y = 1.000				
Effective length factor in minor a	xis	K _z = 1.000				
Effective length factor for torsion		K _{LT.A} = 1.00	00			
		K _{LT.B} = 1.00	20			

SYMMETRYS	Project 43a F	REDINGTON RO	AD, LONDON	NW3 7RA	Job no.	1141
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./	
Symmetrys		G-0	G-GSB01			3
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved d
Classification of cross sectio	ons - Section 5		V/mm² / f _y] = 0 .	81		
Internal compression parts s	ubject to bend	ing - Table 5.2 (sheet 1 of 3)			
Width of section	-	c = d = 160).8 mm			
		c / t _w = 27.4	4×ε <= 72×ε	Class ²	1	
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section		c = (b - t _w -	2×r)/2 = 88	mm		
		c / t _f = 9.8	×ε<=10×ε	Class 2	2	
					Sec	tion is clas
Check shear - Section 6.2.6						
Height of web		h _w = h - 2 >	< t _f = 181.2 mm	1		
Shear area factor		η = 1.000				
		h _w / t _w < 72	!×ε/η			
			- 1	Shear buckling	resistance c	an be igno
Design shear force		V _{Ed} = max(abs(V _{max}), abs	(V _{min})) = 29.9 kN		U
Shear area - cl 6.2.6(3)		A _v = max(A	$-2 \times b \times t_{f} + ($	t_w + 2 × r) × t_f , η >	< h _w × t _w) = 16 9	98 mm ²
Design shear resistance - cl 6.2	2.6(2)	$V_{c,Rd} = V_{pl,F}$	$A_{d} = A_{v} \times (f_{y} / \sqrt{[}$	3]) / γ _{M0} = 347.9 k	N	
				near resistance e		gn shear fo
Check bending moment majo	or (v-v) axis - S	ection 6.2.5				
Design bending moment	(j j)		(abs(M _{s1_max}), a	abs(M _{s1_min})) = 32 .	. 9 kNm	
		M _{Ed} = max		abs(M₅1_min)) = 32. γ _{M0} = 176.6 kNm	9 kNm	
Design bending moment Design bending resistance mor	nent - eq 6.13	$M_{Ed} = max$ $M_{c,Rd} = M_{pl,}$			9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t	nent - eq 6.13	M_{Ed} = max $M_{c,Rd}$ = $M_{pl,}$ ling			9 kNm	
Design bending moment Design bending resistance mor	nent - eq 6.13	$M_{Ed} = max$ $M_{c,Rd} = M_{pl,}$	$_{Rd} = W_{pl.y} \times f_y /$		9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t	nent - eq 6.13	$M_{Ed} = max$ $M_{c,Rd} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$	$_{Rd} = W_{pl.y} \times f_y /$		9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6	nent - eq 6.13	$M_{Ed} = max$ $M_{c,Rd} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$	Rd = W _{pl.y} × f _y /		9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor	nent - eq 6.13	$M_{Ed} = max$ $M_{c,Rd} = M_{pl}$ ling $k_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{[1 - (l_{c})]^{2}}$ $v = 0.3$	Rd = W _{pl.y} × f _y /	γ _{M0} = 176.6 kNm	9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio	nent - eq 6.13	$M_{Ed} = \max_{M_{c,Rd}} M_{c,Rd} = M_{pl}$ ling $k_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{1 - (k_{c}^{2})}$ v = 0.3 G = E / [2 > 0]	Rd = W _{pl.y} × f _y / = 1.132 z / l _y)] = 0.813	γ _{M0} = 176.6 kNm	9 kNm	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length	nent - eq 6.13	$M_{Ed} = \max_{M_{c,Rd}} M_{c,Rd} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{1 - (l_c + l_c)^2}$ $G = E / [2 > l_c + 1.0 \times L]$	$R_{d} = W_{pl.y} \times f_{y} / S_{z}^{2} = 1.132$ $(z / l_{y}) = 0.813$ (1 + v) = 807 $s_{s1} = 4400 \text{ mm}$	γ _{M0} = 176.6 kNm 7 69 N/mm ²		× E × Iz)] =
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus	nent - eq 6.13	$M_{Ed} = \max_{M_{c,Rd}} M_{c,Rd} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{1 - (l_c + l_c)^2}$ $G = E / [2 > l_c + 1.0 \times L]$	$R_{d} = W_{pl.y} \times f_{y} / I_{z}$ $= 1.132$ $I_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $I_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$	γ _{M0} = 176.6 kNm		× E × Iz)] =
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length	nent - eq 6.13 torsional buck	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ ling $k_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{1 - (k_{c}^{2})}$ $G = E / [2 \times k_{c}^{2}]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times 326.5$ kNm	$R_{d} = W_{pl.y} \times f_{y} / I_{z}$ $= 1.132$ $I_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $I_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$	γ _{M0} = 176.6 kNm 7 69 N/mm² ×g)×√[I _w / I _z + L		× E × Iz)] =
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment	nent - eq 6.13 torsional buck	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ ling $k_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{1 - (k_{c}^{2})}$ $G = E / [2 \times k_{c}^{2}]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times 326.5$ kNm	$R_{d} = W_{pl.y} \times f_{y} / R_{d}$ $R_{d} = 1.132$	γ _{M0} = 176.6 kNm 7 69 N/mm² ×g)×√[I _w / I _z + L		× E × Iz)] =
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors	nent - eq 6.13 torsional buck	$\begin{split} M_{Ed} &= \max \\ M_{c,Rd} &= M_{pl}, \\ \\ \text{ling} \\ k_c &= 0.94 \\ C_1 &= 1 \ / \ k_c^2 \\ g &= \sqrt{[1 - (l:} \\ v &= 0.3 \\ G &= E \ / \ [2 \ 2 \\ L &= 1.0 \ \times L \\ M_{cr} &= C_1 \ \times \\ & 326.5 \ kNm \\ \overline{\lambda}_{LT} &= \sqrt{(W)} \\ \overline{\lambda}_{LT,0} &= 0.4 \end{split}$	$R_{d} = W_{pl,y} \times f_{y} / S_{z} = 1.132$ $S_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $S_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = 0$	γ _{M0} = 176.6 kNm 7 69 N/mm² ×g)×√[I _w / I _z + L	$L^2 imes G imes I_t / (\pi^2)$	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio	nent - eq 6.13 torsional buck	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{1 - (42)^2}$ $G = E / [2 + 22)^2$ $L = 1.0 \times L$ $M_{cr} = C_1 \times 326.5$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	$R_{d} = W_{pl,y} \times f_{y} / S_{z} = 1.132$ $S_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $S_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = 0$	γ _{M0} = 176.6 kNm 769 N/mm² × g) × √[I _w / I _z + L 0.735	$L^2 imes G imes I_t / (\pi^2)$	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio	nent - eq 6.13 torsional buck	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{1 - (42)^2}$ $G = E / [2 + 22)^2$ $L = 1.0 \times L$ $M_{cr} = C_1 \times 326.5$ kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	$R_{d} = W_{pl,y} \times f_{y} / S_{z} = 1.132$ $S_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $S_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = 0$	γ _{M0} = 176.6 kNm 769 N/mm² × g) × √[I _w / I _z + L 0.735	$L^2 imes G imes I_t / (\pi^2)$	
Design bending moment Design bending resistance mor Slenderness ratio for lateral to Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for buckling Buckling curve - Table 6.5	nent - eq 6.13 torsional buck	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl},$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{1 - (k_c^2)}$ $g = \sqrt{1 - (k_c^2)}$ $L = 1.0 \times L$ $M_{cr} = C_1 \times$ 326.5 kNm $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT,0} = 0.4$ 3.2.1	$R_{d} = W_{pl,y} \times f_{y} / S_{z} = 1.132$ $S_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $S_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = 0$	γ _{M0} = 176.6 kNm 769 N/mm² × g) × √[I _w / I _z + L 0.735	$L^2 imes G imes I_t / (\pi^2)$	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio	nent - eq 6.13 torsional buck t sional buckling ng - Section 6.3	$M_{Ed} = \max_{M_c,Rd} = M_{pl}$ ling $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{[1 - (l]_2]}$ $Q = C_1 > 1$ $M_{cr} = C_1 > 1$ $M_{cr} = C_1 > 1$ $M_{cr} = C_1 > 1$ $M_{cr} = \sqrt{(W)}$ $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ B.2.1	$R_{d} = W_{pl,y} \times f_{y} / $ $R_{d} = 1.132$ $R_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $R_{z} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = 0$	γ _{M0} = 176.6 kNm 769 N/mm² × g) × √[I _w / I _z + L 0.735	$L^2 imes G imes I_t / (\pi^2)$	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for buckling Buckling curve - Table 6.5 Imperfection factor - Table 6.3	nent - eq 6.13 torsional buckt t sional buckling ng - Section 6.3	$M_{Ed} = \max_{M_c,Rd} = M_{pl}$ $k_c = 0.94$ $C_1 = 1 / k_c^2$ $g = \sqrt{[1 - (l]_2]}$ $Q = 0.3$ $G = E / [2]_2$ $L = 1.0 \times L$ $M_{or} = C_1 \times 1$ 326.5 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $B.2.1$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$	$R_{d} = W_{pl.y} \times f_{y} / R_{d}$ $R_{d} = 1.132$ $R_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $R_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl.y} \times f_{y} / M_{cr} = 0$ $\overline{\lambda}_{L}\tau > \overline{\lambda}_{L}\tau, o - L_{s}$	γ _{M0} = 176.6 kNm 769 N/mm² × g) × √[I _w / I _z + L 0.735 ateral torsional L	-²×G×It / (π² buckling canr	
Design bending moment Design bending resistance mor Slenderness ratio for lateral to Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for buckling Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination factor	nent - eq 6.13 torsional buckt t sional buckling ng - Section 6.3	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ $M_{c,Rd} = M_{pl}$ $K_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{[1 - (!]}$ v = 0.3 G = E / [2 > $L = 1.0 \times L$ $M_{cr} = C_{1} \times$ 326.5 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ 3.2.1 b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times$	$R_{d} = W_{pl,y} \times f_{y} / R_{d}$ $= 1.132$ $= 1.132$ $= 1.132$ $= 0.813$ $(1 + v) = 807$ $= 1.132$ $= 1$	γ _{M0} = 176.6 kNm 769 N/mm ² × g) × √[I _w / I _z + L 0.735 ateral torsional L τ - $\overline{\lambda}_{LT,0}$) + β × $\overline{\lambda}_{L}$ -	$L^2 \times G \times I_t / (\pi^2$ buckling cannot buckling buck	
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled section	nent - eq 6.13 torsional buckt t sional buckling ng - Section 6.3	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ $k_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{[1 - (l]_{c}]_{c}}$ $Q = 0.3$ $G = E / [2 \times L]_{c}$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times L$ 326.5 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = \min(1)$	$R_{d} = W_{pl,y} \times f_{y} / R_{d}$ $R_{d} = 1.132$ $R_{d} = 1.132$ $R_{d} = 0.813$ $(1 + v) = 807$ $R_{s1} = 4400 \text{ mm}$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $R_{pl,y} \times f_{y} / M_{cr} = R_{d}$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L_{d}$ $[1 + \alpha_{LT} \times (\overline{\lambda}_{L})]$ $[1 + \alpha_{LT} + \sqrt{(\phi_{LT})^{2}}]$	γ _{M0} = 176.6 kNm 769 N/mm ² × g) × √[I _w / I _z + L 0.735 ateral torsional L τ - $\overline{\lambda}_{LT,0}$) + β × $\overline{\lambda}_{LT}$ - β × $\overline{\lambda}_{LT}$)], 1, 1 /	$L^2 \times G \times I_t / (\pi^2)$ buckling cannot buckling cannot buckl	not be igno
Design bending moment Design bending resistance mor Slenderness ratio for lateral to Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for buckling Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor	nent - eq 6.13 torsional buck t sional buckling ng - Section 6.3 ions ctor	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ $M_{c,Rd} = M_{pl}$ $K_{c} = 0.94$ $C_{1} = 1 / K_{c}^{2}$ $g = \sqrt{[1 - (H_{c}^{2}]]}$ Q = 0.3 G = E / [2 > L = 1.0 × L $M_{cr} = C_{1} × 326.5 \text{ kNm}$ $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $B_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 × \chi_{LT} = \min(1 - 1)^{2}$	$R_{d} = W_{pl,y} \times f_{y} / R_{d}$ $= 1.132$	γ _{M0} = 176.6 kNm 769 N/mm ² × g) × √[I _w / I _z + L 0.735 ateral torsional L τ - $\overline{\lambda}_{LT,0}$) + β × $\overline{\lambda}_{L^{-1}}$ - β × $\overline{\lambda}_{L}$ τ ²)], 1, 1 / [1 - 2 × ($\overline{\lambda}_{LT}$ - 0.8	$L^2 \times G \times I_t / (\pi^2)$ buckling cannot buckling cannot buckl	not be igno
Design bending moment Design bending resistance mor Slenderness ratio for lateral t Correction factor - Table 6.6 Curvature factor Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac	nent - eq 6.13 torsional buck t sional buckling ng - Section 6.3 ions ctor eq 6.58	$M_{Ed} = \max_{M_{c,Rd}} M_{Ed} = M_{pl}$ $M_{c,Rd} = M_{pl}$ $K_{c} = 0.94$ $C_{1} = 1 / k_{c}^{2}$ $g = \sqrt{[1 - (H_{c}^{2}]}$ V = 0.3 $G = E / [2 \times L]$ $M_{cr} = C_{1} \times \frac{326.5 \text{ kNm}}{\lambda_{LT}} = \sqrt{(W_{c}^{2})}$ $\lambda_{LT} = \sqrt{(W_{c}^{2})}$ $\lambda_{LT,0} = 0.4$ 3.2.1 b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = \min(1 - \chi_{LT,mod})$	$R_{d} = W_{pl,y} \times f_{y} / R_{d}$ $R_{d} = 1.132$ $R_{z} / I_{y} = 0.813$ $(1 + v) = 807$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times F_{y} / M_{cr} = \frac{1}{\lambda_{LT}} \times \lambda_{LT,0} - L_{s}$ $[1 + \alpha_{LT} \times (\lambda_{LT}^{2})$ $I - [\alpha_{LT} + \sqrt{(\alpha_{LT}^{2})}]$ $R_{d} = 0.5 \times (1 - k_{c}) \times 10^{-1}$ $R_{d} = 0.813$	γ _{M0} = 176.6 kNm 769 N/mm ² × g) × √[I _w / I _z + L 0.735 ateral torsional L τ - $\overline{\lambda}_{LT,0}$) + β × $\overline{\lambda}_{L^{-1}}$ - β × $\overline{\lambda}_{L}$ τ ²)], 1, 1 / [1 - 2 × ($\overline{\lambda}_{LT}$ - 0.8	$L^2 \times G \times I_t / (\pi^2)$ buckling cannot buckling cannot buckl	not be igno

8	SYMMETRYS	Project 43a REDINGTON ROAD, LONDON NW3 7RA				Job no. 21141	
	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for	G-G	SB01		Start page no./Re	evision 4
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

Maximum deflection span 1

 δ_{lim} = L_{s1} / 250 = **17.6** mm

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 4.758 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

8	SYMMETRYS	Project 43a RE	EDINGTON ROA	D, LONDON N	V3 7RA	Job no. 21	141
	STRUCTURAL / CIVIL ENGINEERS	Calcs for G-GSC01				Start page no./Revision	
	Symmetrys		-0-0	5001			1
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.06

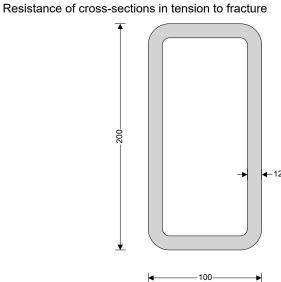
Design summary

Description	Unit	Provided	Required	Utilisation	Result
Shear resistance (y-y)	kN	916	1	0.001	PASS
Shear resistance (z-z)	kN	458	8	0.017	PASS
Axial compression	kN	2381	154	0.065	PASS
Bending resistance (z-z)	kNm	87	23	0.265	PASS
Combined bending & axial	kNm	87	23	0.265	PASS
Buckling in compression	kN	1560	154	0.099	PASS
Combined buckling				0.280	PASS

Partial factors - Section 6.1

Resistance of cross-sections

Resistance of members to instability



γ_{M0} = **1** γ_{M1} = **1**

γ_{M2} = **1.1**

RHS 200x100x12.5 (Tata Steel Celsius (Gr355 Gr420)) Section depth, h, 200 mm Section breadth, b, 100 mm Mass of section, Mass, 52.7 kg/m Section thickness, t, 12.5 mm Area of section, A, 6707 mm² Radius of gyration about y-axis, i_y, 68.377 mm Radius of gyration about y-axis, i_y, 68.377 mm Radius of gyration about y-axis, i_y, 68.377 mm Blastic section modulus about y-axis, W_{el,y}, 313596 mm³ Plastic section modulus about y-axis, W_{pl,y}, 408228 mm³ Plastic section modulus about y-axis, W_{pl,y}, 408228 mm³ Plastic section modulus about y-axis, I_y, 3135964 mm⁴ Second moment of area about y-axis, I_z, 10039542 mm⁴

Column details

Column section	RHS 200x100x12.5
Steel grade	User defined
Yield strength	f _y = 355 N/mm ²
Ultimate strength	f _u = 470 N/mm ²
Modulus of elasticity	E = 210 kN/mm ²
Poisson's ratio	v = 0.3
Shear modulus	G = E / $[2 \times (1 + v)]$ = 80.8 kN/mm ²
Column geometry	

System length for buckling - Major axis $L_y = 3000 \text{ mm}$ System length for buckling - Minor axis $L_z = 3000 \text{ mm}$ The column is not part of a sway frame in the direction of the minor axisThe column is not part of a sway frame in the direction of the major axis

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STRUCTURAL / CIVIL ENGINEERS	Calcs for		Start page no./I	Revision			
Symmetrys		G-G	SC01		2		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved d	
Column loading							
Axial load			<n (compress<="" td=""><td>ion)</td><td></td><td></td></n>	ion)			
Major axis moment at end 1 - B		$M_{y,Ed1} = 0.0$					
Major axis moment at end 2 - T	ор	$M_{y,Ed2} = 0.0$) kNm				
Minor axis moment at end 1 - B	ottom	M _{z,Ed1} = 23	0 kNm				
Minor axis moment at end 2 - T		$M_{z,Ed2} = 2.0$					
	- 1-			ingle curvature			
Major axis shear force		V _{v,Ed} = 1 kN	-	J			
Minor axis shear force		V _{z,Ed} = 8 kN	١				
Buckling length for flexural b	uckling - Maj	or axis					
End restraint factor	- •	K _y = 1.000					
Buckling length		$L_{cr_y} = L_y \times$	K _y = 3000 mm	1			
Buckling length for flexural b	uckling - Min	or axis					
End restraint factor		K _z = 1.000					
Buckling length		$L_{cr_z} = L_z \times$	K _z = 3000 mm	1			
Web section classification (T	able 5.2)						
Coefficient depending on fy		ε = √(235 Ν	$M/mm^2 / f_y) = 0.$.814			
Depth between fillets		c _w = h - 3 ×	: t = 162.5 mm	I			
Ratio of c/t		$ratio_w = c_w$					
Length of web taken by axial loa			$d / (2 \times f_y \times t),$				
For class 1 & 2 proportion in co	mpression		I _w /2) / c _w = 0.5				
Limit for class 1 web		Limit _{1w} = (3	96 × ε) / (13 ×	α - 1) = 52.02	The	web is clas	
Flange section classification	(Table 5.2)						
Depth between fillets		$c_f = b - 3 \times$	t = 62.5 mm				
Ratio of c/t		ratio _f = c _f /	t = 5.00				
Conservatively assume uniform	compression	in flange					
Limit for class 1 flange		Limit _{1f} = 33	×ε = 26.85				
Limit for class 2 flange		Limit _{2f} = 38	ε = 30.92				
Limit for class 3 flange		Limit _{3f} = 42	2×ε = 34.17				
					The fla	nge is clas	
Overall section classification					T 1.	41au 1- 1	
Resistance of cross section (cl. 6.2)				i ne sec	tion is clas	
Shear - Major axis (cl. 6.2.6)							
Design shear force		V _{y,Ed} = 1.0	kN				
Shear area		$A_{vy} = A \times h$	/ (b + h) = 44	72 mm ²			
Plastic shear resistance		$V_{pl,y,Rd} = A_v$	$_{y} imes$ (f _y / $\sqrt{(3)}$) / γ	γ _{M0} = 916.5 kN			
		V _{y,Ed} / V _{pl,y,F}					
				r resistance exco	-		
		$V_{y,Ed} \leq 0.5 \times V$	r _{pl,y,Rd} - No red	luction in f _y requ	ured for bend	ng/axial fo	
Shear - Minor axis (cl. 6.2.6)							
Design shear force		V _{z,Ed} = 8.0	kN				

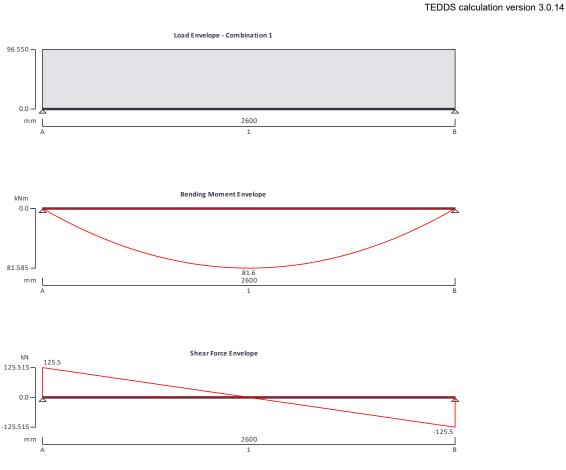
	SYMMETRYS	Project 43a R	EDINGTON RO	AD. LONDON I	NW3 7RA	Job no.	1141	
\sim	STRUCTURAL / CIVIL ENGINEERS	Calcs for		,	-	Start page no./		
	Symmetrys		G-G	SC01		otart page no./i	3	
Unit 6 The	e Courtyard, Lynton Road	Calcs by	Calcs date Checked by Checked date			Approved by	Approved date	
	London N8 8SL	SB	27/08/2021	DS		DS		
Shear a	area		$A_{vz} = A \times b$	/ (b + h) = 22 :	36 mm ²			
Plastic	shear resistance		V _{pl,z,Rd} = A _{vz}	$z \times (f_y / \sqrt{3}) / \gamma$	мо = 458.2 kN			
			V _{z,Ed} / V _{pl,z,F}					
				PASS - Shear	resistance exc	eeds the desig	n shear forc	
			V _{z,Ed} <= 0.5×V	/ _{pl,z,Rd} - No red	uction in fy requ	ired for bendi	ng/axial forc	
Compr	ression (cl. 6.2.4)							
Design			N _{Ed} = 154 k	٢N				
-	resistance		N _{c.Rd} = N _{pl.F}	$a_{d} = A \times f_{y} / \gamma_{M0}$	= 2381 kN			
0			N _{Ed} / N _{c.Rd} =	•				
			,		lesign resistand	ce exceeds the	e design forc	
Bendir	ng - Major axis(cl. 6.2.5))		-	-		-	
	bending moment	/	$M_{z Ed} = max$	(abs(M₂ ∈d1), a	bs(M _{z,Ed2})) = 23.0	0 kNm		
•	modulus			= 244.7 cm ³				
Design	resistance			$_{z}$ \times f _y / γ_{M0} = 86.	9 kNm			
0			M _{z,Ed} / M _{c,z,I}					
					ign resistance e	exceeds the de	esign momer	
Combi	ned bending and axial	force (cl. 6.2.9)	1	-	-		-	
	esign axial to design pla			d) / N _{pl,Rd} = 0.06	35			
	eb area to gross area			5, (A - $2 \times b \times$				
	ange area to gross area		$a_f = min(0.5, (A - 2 \times h \times t) / A) = 0.255$					
				, (<u>-</u>	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,			
	ng - Minor axis (cl. 6.2.9	9.1)	M = mo	(aba(M =) a	$b_{0}(M =) = 22.0$			
-	bending moment			$r_{pl.z} \times f_y / \gamma_{M0} = 8$	bs(M _{z,Ed2})) = 23.0	UKINITI		
	design resistance					a)) = 00 0 kbla	_	
woame	d design resistance				1 - n) / (1 - 0.5 ×	$a_{\rm f})) = 80.9 {\rm kinn}$	n	
		DASS	M _{z,Ed} / M _{N,z,} - Bending resis		onco of avial lo	ad avcoade de	nsian momo	
		FASS	- Denuing resis	stance in pres	ence of axial to		sign momen	
	ng resistance (cl. 6.3)	t	£ - 255 N/	2				
	trength for buckling resis		f _y = 355 N/r	nm-				
	al buckling - Major axis	5		_				
	critical buckling force			$E \times I_y / L_{cr_y^2} =$				
	mensional slenderness		, (f _y / N _{cr,y}) = 0.5	74			
	g curve (Table 6.2)		а					
-	ection factor (Table 6.1)		α _y = 0.21		= -	_		
Parame					$(0.2) + \overline{\lambda}_{y}^{2} = 0.7$			
	ion factor				$(y_y^2 - \overline{\lambda}_y^2)]) = 0.90$	0		
Design	buckling resistance			$\times A \times f_y / \gamma_{M1} =$	2141.8 kN			
			N _{Ed} / N _{b,y,Rd}					
			PASS - The fle	exural bucklin	g resistance ex	ceeds the des	ign axial loa	
Flexur	al buckling - Minor axis	6						
Elastic	critical buckling force			$E \times I_z / L_{cr_z^2} =$				
Non-dir	mensional slenderness		$\overline{\lambda}_z = \sqrt{A \times A}$	f _y / N _{cr,z}) = 1.0	15			
Bucklin	g curve (Table 6.2)		а					
Imperfe	ection factor (Table 6.1)		α _z = 0.21					
					$(0.2) + \overline{\lambda}_z^2 = 1.1$			

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Symmetrys		G-GSC01				4		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da		
Reduction factor		χ _z = min(1.	0, 1 / [Φ_{z} + $\sqrt{4}$	$(\Phi_z^2 - \overline{\lambda}_z^2)]) = 0.65$	5			
Design buckling resistance		$N_{b,z,Rd} = \chi_z$	\times A \times f _y / γ_{M1} =	= 1560.2 kN				
		N_{Ed} / $N_{b,z,Rd}$	= 0.099					
		PASS - The fle	exural bucklir	ng resistance ex	ceeds the des	sign axial lo		
Minimum buckling resistance	9							
Minimum buckling resistance		$N_{b,Rd} = min$	$(N_{b,y,Rd}, N_{b,z,Rd})$	= 1560.2 kN				
		N _{Ed} / N _{b,Rd} =	N _{Ed} / N _{b,Rd} = 0.099					
	I	PASS - The axia	l load bucklir	ng resistance ex	ceeds the des	sign axial lo		
Combined bending and axial	compression (cl. 6.3.3)						
Characteristic resistance to nor	mal force	$N_{Rk} = A \times f_{y}$, = 2381 kN					
Characteristic moment resistan	ce - Major axis	$M_{y,Rk} = W_{pl.}$	$M_{y,Rk} = W_{pl,y} \times f_y =$ 144.9 kNm					
Characteristic moment resistan	ce - Minor axis	$M_{z,Rk} = W_{pl.z} \times f_y = 86.9 \text{ kNm}$						
$\psi_y = if(abs(M_{y,Ed1}) \le abs(M_{y,Ed2})$, M _{y,Ed1} / if(M _{y,Ed2}	>=0 kNm,max(M	_{y,Ed2} ,0.0001 kN	Im),M _{y,Ed2}), M _{y,Ed2}	/ if(M _{y,Ed1} >=0			
kNm,max(M _{y,Ed1} ,0.0001 kNm),M	/l _{y,Ed1})) = 0.000							
Moment distribution factor - Ma	ijor axis	$\psi_y = M_{y,Ed1}$	$/ M_{y,Ed2} = 0.00$	0				
Moment factor - Major axis		C _{my} = max($C_{my} = max(0.4, 0.6 + 0.4 \times \psi_y) = 0.600$					
Moment distribution factor - Min	nor axis	$\psi_z = M_{z,Ed2}$	/ M _{z,Ed1} = 0.08	37				
Moment factor - Minor axis		C _{mz} = max(C_{mz} = max(0.4, 0.6 + 0.4 × ψ_z) = 0.635					
Moment distribution factor for L	TB		$\psi_{LT} = M_{y,Ed1} / M_{y,Ed2} = 0.000$					
Moment factor for LTB			$C_{mLT} = max(0.4, 0.6 + 0.4 \times \psi_{LT}) = 0.600$					
Interaction factor kyy				$\overline{\lambda}_{y}$ - 0.2) $ imes$ N _{Ed} / ()				
Interaction factor k _{zy}		2	$k_{zy} = 1 - min(0.1, 0.1 \times \overline{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times (\chi_z \times N_{Rk} / \gamma_{M1})) = 0.972$					
Interaction factor kzz				$\overline{\lambda}_z$ - 0.2) $ imes$ N _{Ed} / ()	(z × NRk / γм1)] =	= 0.685		
Interaction factor kyz		$k_{yz} = 0.6 \times$	k _{zz} = 0.411					
Section utilisation		-		$(M_{M1}) + k_{yz} \times M_{z,Ed} /$				
		$UR_{B_2} = N_E$		m ₁) + k _{zz} × M _{z,Ed} / PASS - The buck				

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	STRUCTURAL / CIVIL ENGINEERS Symmetrys	Calcs for	1F-1	Start page no./Revision 1			
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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



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 330mm Masonry. 6.5KN/m2 x 5.8m - Permanent full UDL 38 kN/m Roof. 1.3KN/m2 x 7.8m/2 - Permanent full UDL 5 kN/m Roof. 0.6KN/m2 x 7.8m/2 - Variable full UDL 2.3 kN/m 2nd floor. 1KN/m2 x 5m/2 - Permanent full UDL 2.5 kN/m 2nd floor + partitions. 2.7KN/m2 x 5m/2 - Variable full UDL 6.7 kN/m 1st floor. 1KN/m2 x 7.8m/2 - Permanent full UDL 3.9 kN/m 1st floor + partitions. 2.7KN/m2 x 7.8m/2 - Variable full UDL 10.5 kN/m Load combinations Load combination 1 Support A Permanent \times 1.35

SYMMETRYS	Project 43a RE	21141					
STRUCTURAL / CIVIL ENGINEERS	Calcs for 1F-1SB01			Start page n		o./Revision 2	
Jnit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da	
					ble $ imes$ 1.50 anent $ imes$ 1.35		
		Support B			ble $ imes$ 1.50 anent $ imes$ 1.35		
Analysis results				Variab	1.50 ble \times 1.50		
Maximum moment		M _{max} = 81.0	6 kNm	M _{min} =	0 kNm		
Maximum shear		V _{max} = 125	.5 kN	V _{min} =	-125.5 kN		
Deflection		δ _{max} = 1.2 r	nm	δ _{min} =	0 mm		
Maximum reaction at support A		RA_max = 12	2 5.5 kN	R _{A_min}	= 125.5 kN		
Unfactored permanent load reac	tion at support A	A RA_Permanent	= 64.8 kN				
Unfactored variable load reaction	n at support A	R _{A_Variable} =	25.4 kN				
Maximum reaction at support B		R _{B_max} = 12	2 5.5 kN	$R_{B_{min}}$	= 125.5 kN		
Unfactored permanent load reac	tion at support E	B RB_Permanent	= 64.8 kN				
Unfactored variable load reaction	n at support B	R _{B_Variable} =	25.4 kN				
Section details							
Section type		UC 203x20)3x46 (BS4-1)				
Steel grade		S355					
EN 10025-2:2004 - Hot rolled p	roducts of stru	ctural steels					
Nominal thickness of element		t = max(t _f , t	t _w) = 11.0 mm				
Nominal yield strength		f _y = 355 N/	mm²				
Nominal ultimate tensile strength	ı	f _u = 470 N/	mm²				
Modulus of elasticity		E = 21000	0 N/mm²				
	- <u>∓</u> <u>∓</u>						
203.2			-7.2				
	- <u>+</u>						
		203.6					
Partial factors - Section 6.1							
Resistance of cross-sections		γ _{M0} = 1.00					
Resistance of members to instat	oility	γ _{M1} = 1.00					
Resistance of tensile members to	o fracture	γ _{M2} = 1.10					
Lateral restraint							

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Symmetrys		1F-1	1F-1SB01			3
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da
Effective length factors						
Effective length factor in major		K _y = 1.000				
Effective length factor in minor		K _z = 1.000				
Effective length factor for torsio	n	K _{LT.A} = 1.00				
		K _{LT.B} = 1.00	00			
Classification of cross sectio	ns - Section					
		ε = √[235 Ν	J/mm ² / f _y] = 0.8	1		
Internal compression parts s	ubject to ben	ding - Table 5.2 (sheet 1 of 3)			
Width of section		c = d = 160).8 mm			
		c / t _w = 27.4	$4 \times \varepsilon \le 72 \times \varepsilon$	Class	1	
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section		c = (b - t _w -	2×r) / 2 = 88	mm		
		c / t _f = 9.8 :	×ε<=10×ε	Class	2	
					Sec	tion is clas
Check shear - Section 6.2.6						
Height of web		h _w = h - 2 >	< t _f = 181.2 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	×ε/η			
				Shear buckling	g resistance c	an be igno
Design shear force		V _{Ed} = max(abs(V _{max}), abs(V _{min})) = 125.5 kN	N	
Shear area - cl 6.2.6(3)		$A_v = max(A)$	$-2 \times b \times t_{\rm f}$ + (t	w + 2 × r) × t _f , η :	× h _w × t _w) = 169	98 mm²
Design shear resistance - cl 6.2	2.6(2)	$V_{c,Rd} = V_{pl,R}$	$A_{v} = A_{v} \times (f_{y} / \sqrt{3})$	3]) / γ _{M0} = 347.9 k	٨N	
		PAS	SS - Design sh	ear resistance	exceeds desi	gn shear fo
Check bending moment majo	or (y-y) axis -	Section 6.2.5				
Design bending moment		M _{Ed} = max	(abs(M _{s1_max}), a	bs(M _{s1_min})) = 81	.6 kNm	
Design bending resistance mor	nent - eq 6.13	$M_{c,Rd} = M_{pl}$	$_{\rm Rd}$ = W _{pl.y} × f _y / γ	_{′M0} = 176.6 kNm		
Slenderness ratio for lateral t	orsional buc	kling				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (l₂	z / l _y)] = 0.813			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 >	< (1 + v)] = 807	69 N/mm ²		
Unrestrained length		L = 1.0 × L	_{s1} = 2600 mm			
Elastic critical buckling moment		$M_{cr} = C_1 \times C_1$	$\pi^2 \times E \times I_z / (L^2)$	× g) × $\sqrt{[I_w / I_z + I_z]}$	$L^2 \times G \times I_t / (\pi^2)$	$\times E \times I_z)] =$
		753.5 kNm				
Slenderness ratio for lateral tors	sional buckling		$_{\text{pl.y}} \times f_{\text{y}} / M_{\text{cr}}) = 0$.484		
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$				
			$\overline{\lambda_{LT}} > \overline{\lambda_{LT,0}} - La$	teral torsional	buckling canı	not be igno
Design resistance for bucklin	g - Section 6	.3.2.1				
Buckling curve - Table 6.5		b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled sect	ons	β = 0.75				
	otor	ф.т.= 0.5 ×	$[1 + \alpha_{1T} \times (\overline{\lambda}_{1T})]$	- $\overline{\lambda}_{LT,0}$) + $\beta \times \overline{\lambda}_{L}$	⁻²] = 0.602	
LTB reduction determination factor				$\beta \times \overline{\lambda}_{LT^2}$], 1, 1/	-	

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		Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

Modification factor	f = min(1 - 0.5 × (1 - k _c)× [1 - 2 × ($\overline{\lambda}_{LT}$ - 0.8) ²], 1) = 0.976
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = min(\chi_{LT} / f, 1) = 0.990$
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y \ / \ \gamma_{M1} = \textbf{174.9} \ kNm$
PASS - I	Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads Limiting deflection Maximum deflection span 1

 $\delta_{\text{lim}} = L_{s1} / 360 = 7.2 \text{ mm}$

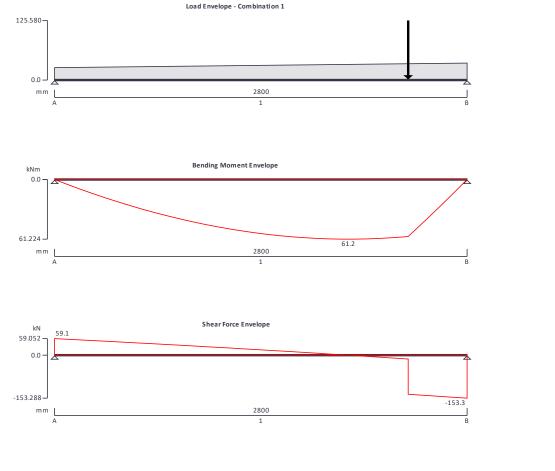
 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 1.21 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

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Symmetrys Unit 6 The Courtyard, Lynton Road London N8 8SL		Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

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



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 225mm Masonry. 4.5KN/m2 x 4.2m to 5.8m - Permanent full VDL 18.9 kN/m to 26.1 kN/m 1F-1SB01 - Permanent point load 64.8 kN at 2400 mm 1F-1SB01 - Variable point load 25.4 kN at 2400 mm Load combinations Support A Load combination 1 $Permanent \times 1.35$ Variable \times 1.50 $Permanent \times 1.35$ Variable \times 1.50

TEDDS calculation version 3.0.14

SYMMETRYS	Project 43a RE	Job no. 21141				
STRUCTURAL / CIVIL ENGINEERS	Calcs for					Revision
Symmetrys Unit 6 The Courtyard, Lynton Road	1F-1SB02					2
London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da
		Support B		Perma	anent $ imes$ 1.35	
				Variab	le imes 1.50	
Analysis results					.	
Maximum moment		$M_{max} = 61.2$			0 kNm	
Maximum shear		V _{max} = 59.1			-153.3 kN	
Deflection		δ _{max} = 0.5 m		δ _{min} = (
Maximum reaction at support A		R _{A_max} = 59		$R_{A_{min}}$	= 59.1 kN	
Unfactored permanent load read						
Unfactored variable load reactio	n at support A	R _{A_Variable} = 3			450.01.01	
Maximum reaction at support B	tion of every and D	$R_{B_{max}} = 15$		$R_{B_{min}}$	= 153.3 kN	
Unfactored permanent load read		-				
Unfactored variable load reactio	n at support B	$R_{B_{Variable}} = 2$	21.8 KN			
Section details		110 202220	2×46 (DC4 4)			
Section type		S355	3x46 (BS4-1)			
Steel grade	raduata of atru					
EN 10025-2:2004 - Hot rolled p Nominal thickness of element	broducts of stru		") = 11.0 mm			
Nominal yield strength		•	•			
Nominal ultimate tensile strength	h	f _y = 355 N/mm ² f _u = 470 N/mm ²				
Modulus of elasticity	1	E = 210000				
modulus of clasticity		L - 210000				
Ť						
203.2			7.2			
~	<u>,</u> <u>↓</u>					
<u>.</u>	<u> </u>					
	◀────	203.6-		▶		
Partial factors - Section 6.1						
Resistance of cross-sections		γ _{M0} = 1.00				
Resistance of members to instal	bility	γ _{M1} = 1.00				
Resistance of tensile members t	-	γ _{M2} = 1.10				
Lateral restraint						
		Span 1 has	lateral restrain	t at supports on	ly	
Effective length factors						
Effective length factors Effective length factor in major a	axis	K _y = 1.000				
		K _y = 1.000 K _z = 1.000				

SYMMETRYS	43a	REDINGTON RO	AD, LONDON	NW3 7RA	2	1141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./			
Symmetrys		1F-	1F-1SB02			3		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved d		
		K _{LT.B} = 1.0	00					
Classification of cross sectio	ns - Section							
I. 4		-	$N/mm^2 / f_y] = 0.$	81				
Internal compression parts so Width of section	ubject to ben	c = d = 16	-					
Width of Section			4×ε<= 72×ε	Class	1			
Outstand flanges - Table 5.2 ((about 2 of 2)	0, w <u>1</u> .						
Width of section	Sheet 2 OF 5)	c = (b - t	- 2 × r) / 2 = 88	mm				
		,	×ε<= 10×ε	Class	2			
		C7 4 - 9.0	× e <= 10 × e	Class		tion is clas		
Check shear - Section 6.2.6					000			
Height of web		h -h-2	≺ t _f = 181.2 mm					
Shear area factor		η = 1.000	∧ ų – 101.2 mm					
		η – 1.000 h _w / t _w < 72) v e / n					
		11w7 tw < 12	. ~ 67 1	Shear buckling	ı resistance c	an be igno		
Design shear force		V _{Ed} = max	(abs(V _{max}), abs	(V _{min})) = 153.3 kN		u 20 .g		
Shear area - cl 6.2.6(3)			,	(t _w + 2 × r) × t _f , η >		98 mm ²		
Design shear resistance - cl 6.2	2.6(2)			3]) / γ _{M0} = 347.9 k				
•	. ,			hear resistance of		gn shear fo		
Check bending moment majo	or (y-y) axis - S	Section 6.2.5						
Design bending moment		M _{Ed} = max	(abs(M _{s1_max}), a	abs(M _{s1_min})) = 61	. 2 kNm			
Design bending resistance mor	nent - eq 6.13	$M_{c,Rd} = M_{pl}$	$_{,Rd}$ = W _{pl.y} × f _y /	γ _{M0} = 176.6 kNm				
Slenderness ratio for lateral t	orsional bucl	kling						
Correction factor - Table 6.6		k _c = 0.94						
		$C_1 = 1 / k_c^2$						
Curvature factor		. .	z / ly)] = 0.813					
Poissons ratio		v = 0.3						
Shear modulus		-	× (1 + v)] = 807	769 N/mm²				
Unrestrained length			. _{s1} = 2800 mm					
Elastic critical buckling moment				$(\times g) \times \sqrt{[I_w / I_z + L]}$	$L^2 \times G \times I_t / (\pi^2)$	$\times E \times I_z$] =		
Claudoweas notic for lateral tow		664.6 kNm		0 545				
Slenderness ratio for lateral tors	sional buckling		$f_{pl.y} \times f_y / M_{cr}) =$	0.515				
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$		ateral torsional	buckling canr	not be igno		
Design resistance for bucklin	g - Section 6	.3.2.1						
Buckling curve - Table 6.5		b						
Imperfection factor - Table 6.3		α _{LT} = 0.34						
Correction factor for rolled section	ons	β = 0.75						
LTB reduction determination fac	ctor	•		$T - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT,0}$	-			
LTB reduction factor - eq 6.57		$\chi_{LT} = \min(2)$	1 / [φ _{LT} + √(φ _{LT} ²	- $\beta \times \overline{\lambda}_{LT}^2$)], 1, 1 /	$\overline{\lambda}_{LT}^2$) = 0.954			
Modification factor		f = min(1 -	0.5 imes (1 - k _c)×	$[1 - 2 \times (\overline{\lambda}_{LT} - 0.8)]$	3) ²], 1) = 0.975			
Madification TD and action for the second	ea 6 58	$\gamma_{\rm IT mod} = m$	in(χ _{L⊤} / f, 1) = ().978				
Modified LTB reduction factor -	09 0.00	λ-1,1100	(<u>//</u> , ., .)					

8	SYMMETRYS STRUCTURAL / CIVIL ENGINEERS	Project 43a RE	DINGTON ROA	Job no. 21141			
	STRUCTURAL / CIVIL ENGINEERS	Calcs for 1F-1SB02				Start page no./Revision 4	
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection

Maximum deflection span 1

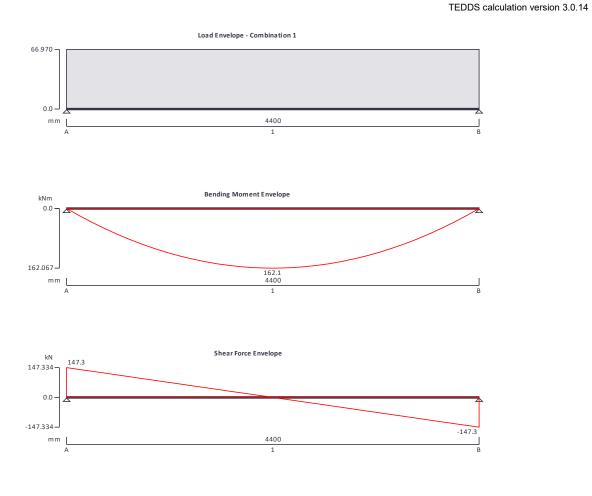
 $\delta_{\text{lim}} = L_{s1} / 360 = 7.8 \text{ mm}$

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 0.516 \text{ mm}$

	SYMMETRYS	Project 43a R	EDINGTON ROA	Job no. 21141			
\sim	STRUCTURAL / CIVIL ENGINEERS	Calcs for		,	-	Start page no./R	evision
	Symmetrys		1F-1	1			
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

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



Support conditions Support A

Support B

Applied loading

Beam loads

Vertically restrained Rotationally free Vertically restrained Rotationally free

Permanent self weight of beam × 1 102.5mm Masonry. 2.2KN/m2 x 7m - Permanent full UDL 15.4 kN/m 2nd floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m 2nd floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 kN/m 1st floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m 1st floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 kN/m

Load combinations

Load combination 1

Support A

Permanent \times 1.35 Variable \times 1.50 Permanent \times 1.35

SYMMETRYS	Project 43a RI	EDINGTON RO	AD, LONDON I	NW3 7RA	Job no. 21141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./Revision		
Symmetrys		1F-	1SB03			2	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved	
				Variab	le imes 1.50		
		Support B		Perma	nent $ imes$ 1.35		
				Variab	le imes 1.50		
Analysis results							
Maximum moment		M _{max} = 162			0 kNm		
Maximum shear		V _{max} = 147			-147.3 kN		
Deflection		δ _{max} = 14.4		$\delta_{\min} = 0$			
Maximum reaction at support A		R _{A_max} = 1 4		$R_{A_{min}}$	= 147.3 kN		
Unfactored permanent load rea		-					
Unfactored variable load reaction		$R_{A_{Variable}} =$		_			
Maximum reaction at support B		R _{B_max} = 14		R _{B_min}	= 147.3 kN		
Unfactored permanent load rea		-					
Unfactored variable load reaction	on at support B	$R_{B_Variable} =$	49.1 KIN				
Section details			00-74 (00 1 4)				
Section type			03x71 (BS4-1)				
Steel grade EN 10025-2:2004 - Hot rolled	producto of ot-	S355					
Nominal thickness of element	products of str		t _w) = 17.3 mm				
Nominal yield strength		f _y = 345 N/					
Nominal ultimate tensile strengt	th	f _u = 470 N/					
Modulus of elasticity		E = 21000					
,	Ļ						
	Ť						
	215.8-	-	⊢ 10				
	Ĩ						
	↓ 1 <u>1</u> .3 ↓						
	<u>▼</u> <u></u>						
		206.4	I	→			
Partial factors - Section 6.1							
Fartial lactors - Section 6.1		γ _{M0} = 1.00					
Resistance of cross-sections		γ _{M1} = 1.00					
	ability	1.00					
Resistance of cross-sections		γ _{M2} = 1.10					
Resistance of cross-sections Resistance of members to insta		•					
Resistance of cross-sections Resistance of members to insta Resistance of tensile members		γ _{M2} = 1.10	s lateral restrair	nt at supports on	ly		
Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint		γ _{M2} = 1.10	s lateral restrair	nt at supports on	ly		
Resistance of cross-sections Resistance of members to insta Resistance of tensile members	to fracture	γ _{M2} = 1.10		nt at supports on	ly		

SYMMETRYS	Project 43a F	REDINGTON RO	AD, LONDON I	NW3 7RA	Job no. 2	1141
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./	Revision
Symmetrys		1F-1	ISB03			3
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da
Effective length factor for torsio	n	K _{LT.A} = 1.0				
		K _{LT.B} = 1.00	00			
Classification of cross sectio	ns - Section 5					
		-	J/mm ² / f _y] = 0.8	33		
Internal compression parts so Width of section	ubject to benc	:) c = d = 16	-			
Width of Section			5×ε<= 72×ε	Class	1	
Outstand flanges Table 5.2	(about 2 of 2)	0, t _w 10.		01000	•	
Outstand flanges - Table 5.2 (Width of section	Sheet 2 of 3)	c = (b - t _w -	2×r)/2=88	mm		
			×ε<=9×ε	Class	1	
						tion is clas
Check shear - Section 6.2.6						
Height of web		h _w = h - 2 >	< t _f = 181.2 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	×ε/η			
				Shear buckling	resistance c	an be ignor
Design shear force				(V _{min})) = 147.3 kN		
Shear area - cl 6.2.6(3)				$t_w + 2 \times r) \times t_f, \eta >$	-	27 mm ²
Design shear resistance - cl 6.2	2.6(2)			3]) / γ _{M0} = 483.5 k		un abaau fa
	<i>.</i>		55 - Design sn	ear resistance e	exceeds desig	in snear to
Check bending moment major Design bending moment	or (y-y) axis - S		(abc/M) a	bc(M + 1) = 16		
Design bending resistance mor	nent - eg 6 13			lbs(M _{s1_min})) = 16 2 γ _{M0} = 275.6 kNm	2.1 KINIII	
6 6	•		Ru VVpi.y XVy7			
Slenderness ratio for lateral t Correction factor - Table 6.6	orsional buck	kc = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (l₂	z / l _y)] = 0.817			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 >	< (1 + v)] = 807	69 N/mm ²		
Unrestrained length			_{s1} = 4400 mm		_	
Elastic critical buckling moment				\times g) $\times \sqrt{[I_w / I_z + L]}$	$L^2 \times G \times I_t / (\pi^2)$	$\times E \times I_z)] =$
Plandarnoos ratio for lateral tor	nional husblir -	691.2 kNm		1 624		
Slenderness ratio for lateral tors Limiting slenderness ratio	Sonal Duckling	$\lambda_{LT} = \sqrt{VV}$ $\overline{\lambda}_{LT,0} = 0.4$	$_{\text{pl.y}} \times f_{\text{y}} / M_{\text{cr}} = 0$	1.001		
Limiting Schoeniess 1810				ateral torsional l	buckling can	not be iano
Design resistance for bucklin	a - Section 6		·w_i · /w_i,U − ∟ C			
Buckling curve - Table 6.5	9 - Section 6.	b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled section	ons	β = 0.75				
LTB reduction determination fac	ctor	ϕ_{LT} = 0.5 $ imes$	[1 + $\alpha_{LT} \times (\overline{\lambda}_{LT})$	- $\overline{\lambda}_{LT,0}$) + $\beta \times \overline{\lambda}_{L}$	²] = 0.689	
LTB reduction factor - eq 6.57		χ _{LT} = min(1	/ [ϕ _{LT} + √(ϕ _{LT} ² ·	- $\beta \times \overline{\lambda}_{LT}^2$)], 1, 1 /	$\overline{\lambda}_{LT}^2$) = 0.903	
Modification factor		f = min(1 -	0.5 × (1 − k _c)× [$1 - 2 \times (\overline{\lambda}_{LT} - 0.8)$	3) ²], 1) = 0.972	
Modified LTB reduction factor -	eq 6.58	$\chi_{\text{LT,mod}} = m^2$	in(χ _{LT} / f, 1) = 0	.929		
Design buckling resistance mor	nent - ea 6 55	$M_{h,Rd} = \gamma_{I,T}$	mod × Wnly × fy /	γ _{M1} = 256 kNm		

\$ SYMMETRYS STRUCTURAL/CIVIL ENGINEERS	Project 43a R	EDINGTON ROA	Job no. 21141			
STRUCTURAL / CIVIL ENGINEERS Symmetrys		Calcs for 1F-1SB03				evision 4
e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

Maximum deflection span 1

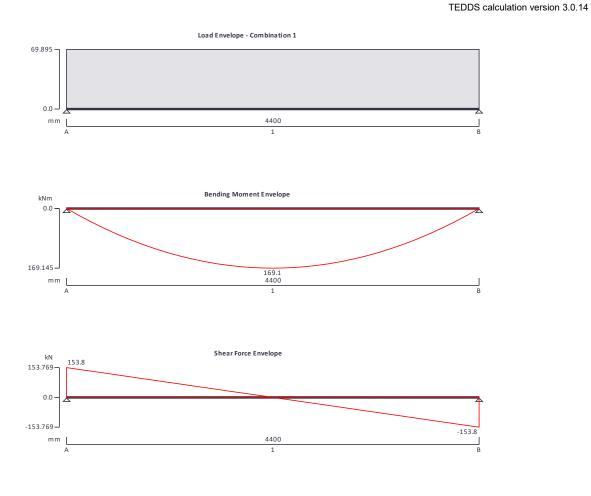
 $\delta_{\text{lim}} = L_{s1} / 250 = 17.6 \text{ mm}$

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **14.367** mm

	SYMMETRYS	Project 43a R	EDINGTON ROA	Job no. 21141			
V	STRUCTURAL / CIVIL ENGINEERS		Calcs for 1F-1SB04				evision 1
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

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



Support conditions Support A

Support B

Applied loading

Beam loads

Vertically restrained Rotationally free Vertically restrained Rotationally free

Permanent self weight of beam × 1 102.5mm Masonry. 2.2KN/m2 x 7m - Permanent full UDL 15.4 kN/m 2nd floor. 1KN/m2 x 9.5m/2 - Permanent full UDL 4.7 kN/m 2nd floor + partitions. 2.7KN/m2 x 9.5m/2 - Variable full UDL 12.8 kN/m 1st floor. 1KN/m2 x 8.4m/2 - Permanent full UDL 4.2 kN/m 1st floor + partitions. 2.7KN/m2 x 8.4m/2 - Variable full UDL 11.3 kN/m

Load combinations

Load combination 1

Support A

Permanent \times 1.35 Variable \times 1.50 Permanent \times 1.35

SYMMETRYS	Project 43a R	EDINGTON RO	AD, LONDON	NW3 7RA	Job no. 21141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for						
Symmetrys		1F-1	1SB04		2		
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved d	
				Variab	le × 1.50		
		Support B		Perma	anent $ imes$ 1.35		
				Variab	le imes 1.50		
Analysis results							
Maximum moment		M _{max} = 169	. 1 kNm	M _{min} =	0 kNm		
Maximum shear		V _{max} = 153	.8 kN	V _{min} =	-153.8 kN		
Deflection		δ _{max} = 15 m	nm	δ _{min} =	0 mm		
Maximum reaction at support A	L Contraction of the second se	R _{A_max} = 15	5 3.8 kN	R_{A_min}	= 153.8 kN		
Unfactored permanent load rea		_					
Unfactored variable load reaction		R _{A_Variable} =					
Maximum reaction at support B		R _{B_max} = 15		R _{B_min}	= 153.8 kN		
Unfactored permanent load rea		—					
Unfactored variable load reaction	on at support B	$R_{B_{Variable}} =$	53 kN				
Section details							
Section type			03x71 (BS4-1)				
Steel grade		S355					
EN 10025-2:2004 - Hot rolled	products of str						
Nominal thickness of element		•	t _w) = 17.3 mm				
Nominal yield strength		f _y = 345 N/					
Nominal ultimate tensile strengt	th	f _u = 470 N/					
Modulus of elasticity		E = 21000	U N/MM²				
		206.4	- 10				
Partial factors - Section 6.1							
Resistance of cross-sections		γ _{M0} = 1.00					
Resistance of members to insta	ability	γ _{M0} = 1.00 γ _{M1} = 1.00					
Resistance of tensile members		γ _{M1} = 1.00 γ _{M2} = 1.10					
		γm2 – 1.10					
I adamal us admaind		0 11	. 1. 4		h.,		
Lateral restraint		Span 1 has	s lateral restrai	nt at supports on	Iy		
Lateral restraint							
Effective length factors							
		K _y = 1.000 K _z = 1.000					

SYMMETRYS	Project 43a R	EDINGTON RO	AD, LONDON N	IW3 7RA	Job no. 2 ⁻	1141
STRUCTURAL / CIVIL ENGINEERS	Calcs for				Start page no./ł	Revision
Symmetrys		1F- 1	SB04			3
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved da
Effective length factor for torsio	n	K _{LT.A} = 1.00				
		K _{LT.B} = 1.00	0			
Classification of cross sectio	ns - Section 5.			_		
		-	l/mm ² / f _y] = 0.8 3	3		
Internal compression parts so Width of section	ubject to bend	ing - Table 5.2 (s c = d = 160	-			
			5×ε<= 72×ε	Class ²	1	
Outstand flanges Table 5.2	(abaat 0 af 0)	07 tw = 13.) ~ C ~ 12 ~ C	01033	I	
Outstand flanges - Table 5.2 (Width of section	(Sheet 2 OF 3)	c = (b - t	2 × r) / 2 = 88 n	nm		
			<ε<=9×ε	Class ²	1	
		0,1,012,		01000		tion is clas
Check shear - Section 6.2.6						
Height of web		h _w = h - 2 >	: t _f = 181.2 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	×ε/η			
				Shear buckling	resistance c	an be ignoi
Design shear force		V _{Ed} = max(abs(V _{max}), abs(\	/ _{min})) = 153.8 kN	l	
Shear area - cl 6.2.6(3)				$_v$ + 2 × r) × t _f , η >	,	2 7 mm²
Design shear resistance - cl 6.2	2.6(2)]) / γ _{M0} = 483.5 k		
			SS - Design she	ear resistance e	exceeds desig	n shear fo
Check bending moment majo	or (y-y) axis - S					
Design bending moment				$s(M_{s1_{min}})) = 169$	9.1 kNm	
Design bending resistance mor	·		$r_{d} = VV_{pl.y} \times T_y / \gamma_{f}$	_{M0} = 275.6 kNm		
Slenderness ratio for lateral t	orsional buckl	-				
Correction factor - Table 6.6		k _c = 0.94 C ₁ = 1 / k _c ²	= 1 132			
Curvature factor			/ l _v)] = 0.817			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 >	< (1 + ν)] = 8076	39 N/mm ²		
Unrestrained length		-	₃1 = 4400 mm			
Elastic critical buckling moment	:	$M_{cr} = C_1 \times c_1$	$\tau^2 \times E \times I_z / (L^2 \times I_z)$	$\langle g \rangle \times \sqrt{[I_w / I_z + L]}$	$L^2 imes G imes I_t / (\pi^2)$	$\times E \times I_z)$] =
		691.2 kNm				
Slenderness ratio for lateral tors	sional buckling		$f_{y} \times f_{y} / M_{cr}$ = 0.	.631		
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$				
			$\lambda_{LT} > \lambda_{LT,0} - Lat$	teral torsional l	buckling cann	ot be ignoi
Design resistance for bucklin	ig - Section 6.3					
		b				
Buckling curve - Table 6.5		α _{LT} = 0.34				
Imperfection factor - Table 6.3		· ·				
Imperfection factor - Table 6.3 Correction factor for rolled section		$\beta = 0.75$	[4]	$\overline{1}$) \cdot 0 $\overline{1}$	21 - 0 000	
Imperfection factor - Table 6.3 Correction factor for rolled section LTB reduction determination factor		φ _{LT} = 0.5 ×		$- \overline{\lambda}_{LT,0} + \beta \times \overline{\lambda}_{L}$		
Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57		$\phi_{LT} = 0.5 \times \chi_{LT} = min(1)$	/ [ϕ_{LT} + $\sqrt{(\phi_{LT}^2 -$	$\beta imes \overline{\lambda}_{LT}^2$)], 1, 1 /	$\overline{\lambda}_{\text{LT}}{}^2) = \textbf{0.903}$	
Imperfection factor - Table 6.3 Correction factor for rolled section LTB reduction determination factor	ctor	φ _{LT} = 0.5 × χ _{LT} = min(1 f = min(1 -	/ [ϕ_{LT} + $\sqrt{(\phi_{LT}^2 -$	$eta imes \overline{\lambda}_{LT}^2)], 1, 1 / - 2 imes (\overline{\lambda}_{LT} - 0.8)$	$\overline{\lambda}_{\text{LT}}{}^2) = \textbf{0.903}$	

8	SYMMETRYS STRUCTURAL/CIVIL ENGINEERS	Project 43a R	EDINGTON ROA	Job no. 21141			
	STRUCTURAL / CIVIL ENGINEERS Symmetrys		Calcs for 1F-1SB04				evision 4
Unit 6 Th	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 27/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

Maximum deflection span 1

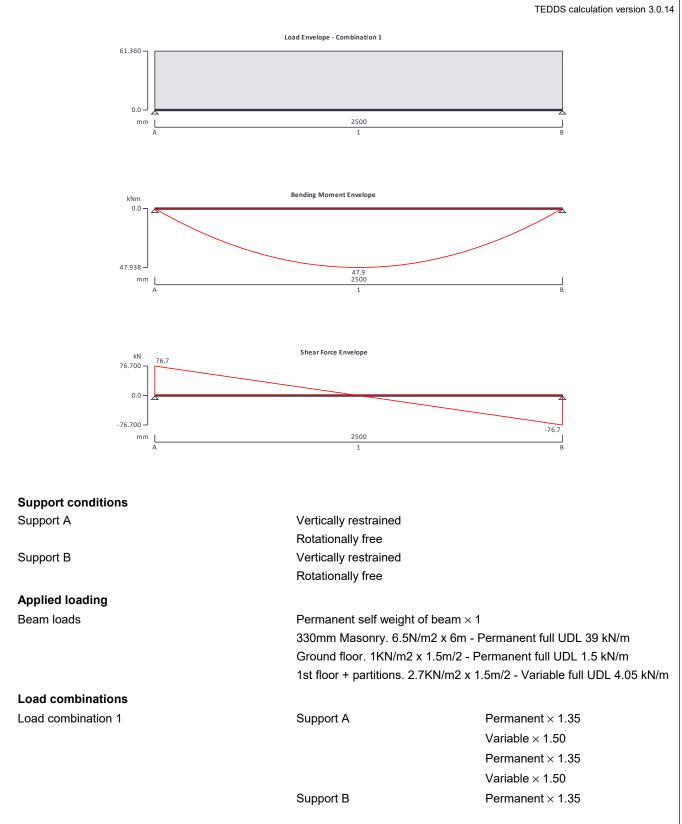
 $\delta_{\text{lim}} = L_{s1} / 250 = 17.6 \text{ mm}$

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **14.977** mm

8	SYMMETRYS STRUCTURAL / CIVIL ENGINEERS	Project 43a R	EDINGTON ROA	Job no. 21141			
	Structural / civil engineers Symmetrys		Calcs for 1F-1SB05				evision 1
	e Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 31/08/2021	Checked by DS	Checked date	Approved by DS	Approved date

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



SYMMETRYS	43a REDINGTON ROAD, LONDON NW3 7RA				21141		
STRUCTURAL / CIVIL ENGINEERS	Calcs for		0005		Start page no./Revision		
Symmetrys			SB05			2	
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 31/08/2021	Checked by DS	Checked date	Approved by DS	Approved da	
				Variab	le × 1.50		
Analysis results					0 1		
Maximum moment Maximum shear		M _{max} = 47.9 V _{max} = 76.7			0 kNm -76.7 kN		
Deflection		ν _{max} = 78.7 δ _{max} = 2.4 n		$\delta_{min} = 0$			
Maximum reaction at support A		R _{A max} = 76			= 76.7 kN		
Unfactored permanent load read	tion at support A	—					
Unfactored variable load reactio		R _{A_Variable} =					
Maximum reaction at support B		R _{B_max} = 76	.7 kN	$R_{B_{min}}$	= 76.7 kN		
Unfactored permanent load read		—					
Unfactored variable load reaction	n at support B	$R_{B_{Variable}} =$	5.1 kN				
Section details							
Section type			3x46 (BS4-1)				
Steel grade		S355					
EN 10025-2:2004 - Hot rolled p	products of stru		\ 44 0				
Nominal thickness of element		t = max(t _f , t f _y = 355 N/r	w) = 11.0 mm				
Nominal yield strength Nominal ultimate tensile strengt	n	$f_v = 355 \text{N/r}$ $f_u = 470 \text{N/r}$					
Modulus of elasticity	•	E = 210000					
	<u>∓</u>						
			7.2				
	•	203.6-					
Partial factors - Section 6.1							
Resistance of cross-sections		γ _{M0} = 1.00					
Resistance of members to insta	-	γ _{M1} = 1.00					
Resistance of tensile members	o tracture	γ _{M2} = 1.10					
Lateral restraint		Span 1 has	lateral restrair	nt at supports on	ly		
Effective length factors							
Effective length factor in major a	ixis	K _y = 1.000					
Effective length factor in minor a		K _z = 1.000					
Effective length factor for torsion	ı	K _{LT.A} = 1.00	0				
		K _{LT.B} = 1.00	0				

SYMMETRYS	Project 43a R	EDINGTON RO	AD, LONDON	NW3 7RA	Job no. 2	1141					
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Symmetrys											
Unit 6 The Courtyard, Lynton Road London N8 8SL	Calcs by SB	Calcs date 31/08/2021	Checked by DS	Checked date	Approved by DS	Approved d					
Classification of cross sectio	ns - Section 5.		$J/mm^2 / f_y] = 0.3$	81							
Internal compression parts s	ubject to bend	-									
Width of section		c = d = 160	-								
		c / t _w = 27.4	4×ε<= 72×ε	Class ²	1						
Outstand flanges - Table 5.2 ((sheet 2 of 3)										
Width of section	,	c = (b - t _w -	2×r) / 2 = 88	mm							
			, ×ε<=10×ε	Class 2	2						
		·			Sec	tion is clas					
Check shear - Section 6.2.6											
Height of web		h _w = h - 2 >	< t _f = 181.2 mm	1							
Shear area factor		η = 1.000									
		h _w / t _w < 72	×ε/n								
				Shear buckling	resistance c	an be iqno					
Design shear force	V _{Ed} = max(V_{Ed} = max(abs(V_{max}), abs(V_{min})) = 76.7 kN									
Shear area - cl 6.2.6(3)		A _v = max(A	$A_{v} = \max(A - 2 \times b \times t_{f} + (t_{w} + 2 \times r) \times t_{f}, \eta \times h_{w} \times t_{w}) = 1698 \text{ mm}^{2}$								
Design shear resistance - cl 6.2	2.6(2)	$V_{c,Rd} = V_{pl,F}$	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 347.9 \text{ kN}$								
		PAS	SS - Design sl	hear resistance e	exceeds desig	yn shear fo					
Check bending moment majo	or (y-y) axis - S	ection 6.2.5									
Design bending moment			(abs(M _{s1_max}), a	abs(M _{s1_min})) = 47 .	9 kNm						
Design bending resistance mor	nent - eq 6.13	$M_{c,Rd} = M_{pl}$	$_{\rm Rd}$ = W _{pl.y} × f _y /	γ _{M0} = 176.6 kNm							
Slenderness ratio for lateral t	orsional buck	ina									
Correction factor - Table 6.6		k _c = 0.94									
		$C_1 = 1 / k_c^2$	= 1.132		$g = \sqrt{[1 - (I_z / I_y)]} = 0.813$						
Curvature factor											
Curvature factor Poissons ratio											
		g = √[1 - (l _i v = 0.3		769 N/mm²							
Poissons ratio		g = √[1 - (l; v = 0.3 G = E / [2 :	z / ly)] = 0.813	7 69 N/mm²							
Poissons ratio Shear modulus		g = √[1 - (l: v = 0.3 G = E / [2 × L = 1.0 × L	z / Iy)] = 0.813 ≺ (1 + ∨)] = 807 ₅1 = 2500 mm	769 N/mm² ×g)×√[I _w / I _z + L	$L^2 \times G \times I_t / (\pi^2)$	$\times E \times I_z)] =$					
Poissons ratio Shear modulus Unrestrained length	:	g = √[1 - (l: v = 0.3 G = E / [2 × L = 1.0 × L	z / Iy)] = 0.813 ≺ (1 + ∨)] = 807 ₅1 = 2500 mm		$L^2 \times G \times I_t / (\pi^2)$	$\times E \times I_z)] =$					
Poissons ratio Shear modulus Unrestrained length		g = √[1 - (l; v = 0.3 G = E / [2 : L = $1.0 \times L$ M _{or} = C ₁ × 806 kNm	z / Iy)] = 0.813 ≺ (1 + ∨)] = 807 ₅1 = 2500 mm	$ imes$ g) $ imes \sqrt{[I_w / I_z + L]}$	$L^2 imes G imes I_t / (\pi^2)$	$\times E \times I_z)] =$					
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment		g = √[1 - (l; v = 0.3 G = E / [2 : L = $1.0 \times L$ M _{or} = C ₁ × 806 kNm	(1 + v) = 0.813 (1 + v)] = 807 $\pi^2 \times E \times I_z / (L^2)$ ply $\times f_y / M_{cr} = 0$	$ imes$ g) $ imes \sqrt{[I_w / I_z + L]}$	$L^2 imes G imes I_t / (\pi^2)$	× E × Iz)] =					
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors		$g = \sqrt{[1 - (l_{2} + v_{1})]}$ $G = E / [2 + v_{2}]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times$ 806 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ $= p_{l,y} \times f_y / M_{cr} = 0$	$ imes$ g) $ imes \sqrt{[I_w / I_z + L]}$							
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors	sional buckling	$g = \sqrt{[1 - (leven v = 0.3]]}$ $G = E / [2 \times 2]$ $L = 1.0 \times L$ $M_{cr} = C_1 \times $ 806 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ $= p_{l,y} \times f_y / M_{cr} = 0$	×g)×√[I _w / I _z + L 0.468							
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio	sional buckling	$g = \sqrt{[1 - (leven v = 0.3]]}$ $G = E / [2 \times 2]$ $L = 1.0 \times L$ $M_{cr} = C_1 \times $ 806 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ $= p_{l,y} \times f_y / M_{cr} = 0$	×g)×√[I _w / I _z + L 0.468							
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin	sional buckling	$g = \sqrt{[1 - (l_{x}^{2} + c_{y}^{2})]^{2}}$ $G = E / [2 + c_{y}^{2} + c_{y}^{2}]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times $ 806 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ 3.2.1	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ $= p_{l,y} \times f_y / M_{cr} = 0$	×g)×√[I _w / I _z + L 0.468							
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5	sional buckling ng - Section 6.3	$g = \sqrt{[1 - (l]_{v}]}$ $v = 0.3$ $G = E / [2 \times L]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times $ 806 kNm $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ 6.2.1 b	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ $= p_{l,y} \times f_y / M_{cr} = 0$	×g)×√[I _w / I _z + L 0.468							
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3	sional buckling n g - Section 6.3 ions	$g = \sqrt{[1 - (l]_{v}]}$ $v = 0.3$ $G = E / [2 + 1]_{v}$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times 806 \text{ kNm}$ $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$	(1 + v) = 0.813 (1 + v) = 807 = 2500 mm $\pi^2 \times E \times I_z / (L^2)$ = 1000000000000000000000000000000000000	×g)×√[I _w / I _z + L 0.468	buckling cann						
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled section	sional buckling n g - Section 6.3 ions	g = [1 - (leven with the set of the set	$[2 / l_y)] = 0.813$ (1 + v)] = 807 $s_1 = 2500 \text{ mm}$ $\pi^2 \times E \times l_z / (L^2)$ $p_{l,y} \times f_y / M_{cr}) = 0$ $\overline{\lambda} \overline{\mu} \tau > \overline{\lambda} \overline{\mu} \tau, o - L^2$ $[1 + \alpha_{LT} \times (\overline{\lambda} L^2)]$	×g)×√[l _w /l _z + L 0.468 ateral torsional L	buckling cann 1 ²] = 0.594						
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination factor	sional buckling n g - Section 6.3 ions	$g = \sqrt{[1 - (l_{x} + v_{z})]^{2}}$ $G = E / [2 + v_{z}]^{2}$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times \frac{806 \text{ kNm}}{\lambda_{LT}} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = \min(1)$	[2 / y] = 0.813 (1 + v)] = 807 (1 + v)] = 807 $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $\mu_{y} \times f_{y} / M_{cr}) = 0$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $[1 + \alpha_{LT} \times (\overline{\lambda}_{L})]$	× g) × $\sqrt{[I_w / I_z + L]}$ 0.468 ateral torsional L T - $\overline{\lambda}_{LT,0}$) + $\beta \times \overline{\lambda}_{L}$	buckling cannot $r^2] = 0.594$ $\overline{\lambda}_{LT^2}) = 0.973$						
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac	sional buckling n g - Section 6.3 ions ctor	$g = [1 - (leven with methods]{1 - (leven w$	[2 / y] = 0.813 (1 + v)] = 807 (1 + v)] = 807 $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $\mu_{y} \times f_{y} / M_{cr}) = 1$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $[1 + \alpha_{LT} \times (\overline{\lambda}_{L})]$	× g) × $\sqrt{[I_w / I_z + L]}$ 0.468 ateral torsional L $T - \overline{\lambda}_{LT,0}$ + $\beta \times \overline{\lambda}_{L}$ - $\beta \times \overline{\lambda}_{L}T^2$], 1, 1 / [1 - 2 × ($\overline{\lambda}_{LT}$ - 0.8	buckling cannot $r^2] = 0.594$ $\overline{\lambda}_{LT^2}) = 0.973$						
Poissons ratio Shear modulus Unrestrained length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Design resistance for bucklin Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor	sional buckling ng - Section 6.3 tions ctor eq 6.58	g = [1 - (leven with the set of the set	(1 + v) = 0.813 (1 + v) = 807 (1 + v) = 807 $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times E \times I_{z} / (L^{2})$ $\pi^{2} \times F_{y} / M_{cr} = 1$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $(1 + \alpha_{LT} \times (\overline{\lambda}_{L})^{2})$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$ $\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L^{2}$	× g) × $\sqrt{[I_w / I_z + L]}$ 0.468 ateral torsional L $T - \overline{\lambda}_{LT,0}$ + $\beta \times \overline{\lambda}_{L}$ - $\beta \times \overline{\lambda}_{L}T^2$], 1, 1 / [1 - 2 × ($\overline{\lambda}_{LT}$ - 0.8	buckling cannot $r^2] = 0.594$ $\overline{\lambda}_{LT^2}) = 0.973$						

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Check vertical deflection - Section 7.2.1 Consider deflection due to permanent and variable loads Limiting deflection $\delta_{\text{lim}} = L_{\text{s1}} \ / \ 250 = \textbf{10} \ \text{mm}$ Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 2.386 \text{ mm}$

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