

82 MARINE CRESCENT

Balustrade, & Terrace Sub-Frame Design

Revision 01

Client

Lela & Patrick Clough,

Project Address

82 Marine Crescent,
Worthing,
BN12 4JH

Issue Date

14th September 2026

Our Ref

26-048



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82 MARINE CRESCENT

Balustrade & Terrace Sub-Frame Design



DOCUMENT ISSUE REGISTER

Job	82 Marine Crescent			
Document	Balustrade & Terrace Sub-Frame Design			
Issue	Date	Description	By	Checked
01	12/09/2026	For Client/Contractor Information & Building Control Approval	Philip Wainwright, CEng, MStructE	14/09/2026

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SECTION 1 – INTRODUCTION

Creative Structures were appointed to provide the Engineering input for a new external balustrade steel sub-frame, which also supports the terrace joists.

The terrace is installed on a warm roof, which is an existing roof terrace but has suffered from historical water ingress from poorly fitted balustrade posts in to the timber decking. The roof covering and rooflight are to be removed prior to the new terrace & balustrade works with a new GRP finish.

The proposed arrangement is to comprise a fully independent HDG steel sub-frame, which is hidden below the raised decking on pedestal feet. A glass balustrade is then fixed to the edge steels to resist the torsional moment(s) induced from the cantilever balustrade arrangement.

The image below shows the current roof arrangement. The current roof light is to be replaced with a toughened safety glass walk on rated rooflight. Otherwise, the proposal is to upgrade the balustrade and terrace arrangement to minimize future water ingress by omitting all penetrations through the roof covering.



SECTION 2 – STRUCTURAL ANALYSIS

The balustrade sub-frame analysis is designed to be formed as separate members assembled on site, thus easing logistics and installation on a tight rear garden site.

The frame is to comprise 100x50x5.0 RHS members throughout, which are hot dip galvanised (HDG) to 85m.

The vertical loads are not a concern as the frame is to sit directly on to the warm roof via pedestals at regular centres, which replicated the existing terrace loads.

The balcony edge steels were assessed using a 0.74kN/m line load, however it has been proven that the self-weight of the steel sub-frame + the self-weight of any person(s) imposing the handrail load are sufficient to counter any notional uplift imposed from the balustrade loads and the self-weight of the steel sub-frame, the perimeter glass balustrading, and the internal terrace decking are greater than the lateral wind loads when using favourable and unfavourable factored loads.

The following page(s) confirm the adequacy of the sub-frame and the torsional resistance of the edge 100x50 RHS members, the decking joists and the internal supports.

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Project: 82 MARINE CRESCENT

Part of Structure: TIMBERS.

Project No: 26-048

Date: 14/09/2026

Engineer: P.W.

Sheet No:

DECKING JOISTS

THE Decking joists are support composite decking only, they are not have
A double span with intermediate support on PARASOLS.

All joists to be 50x50. At 300mm c/c

$$\text{loads: } gl = 0.3 \text{ m c/c} \times 0.20 \text{ kN/m}^2 = 0.06 \text{ kN/m}$$

$$\text{max } S_{gl} = 0.3 \text{ m c/c} \times 2.50 \text{ kN/m}^2 = 0.75 \text{ kN/m}$$

$$\text{max double span} = 2 \times 0.7 \text{ m} \quad (1.4 \text{ m})$$

$$\text{max triple span} = 3 \times 0.67 \text{ m} \quad (2.0 \text{ m})$$

USE PROPOSED 2" x 2" JOISTS ARE ADEQUATE.

< 29% utilisation
~ 0.4 m deflection (51% c/c)

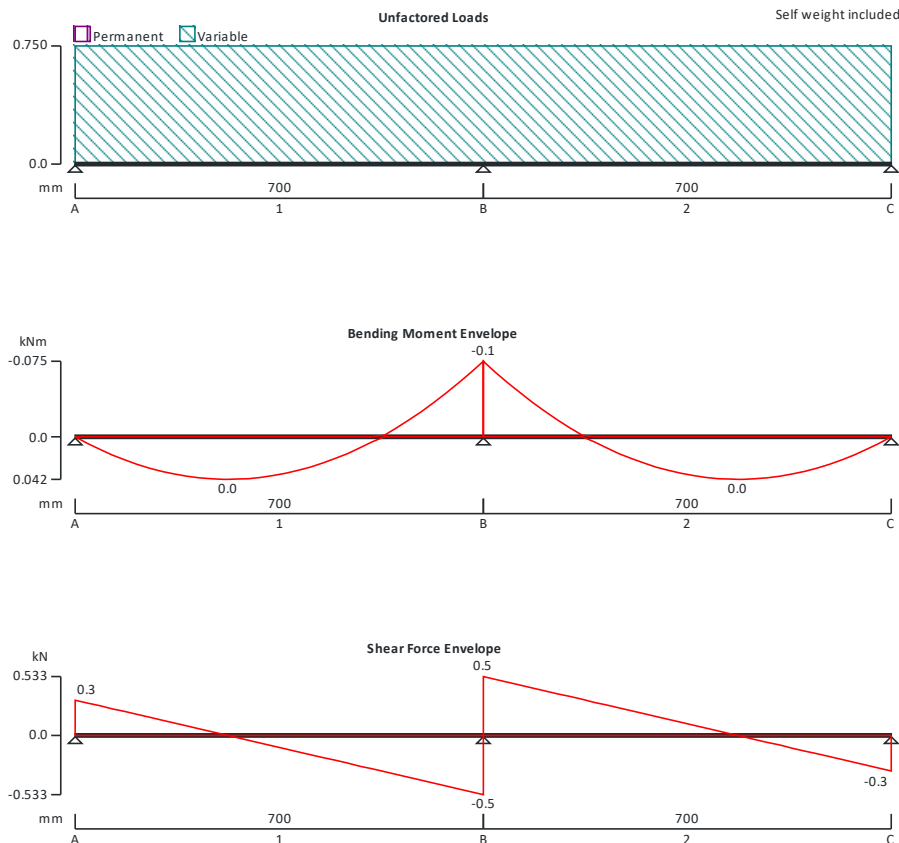
Based on 30 kg/m² live load

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TERRACE JOISTS				1	
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TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

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

Tedds calculation version 1.7.05



Applied loading

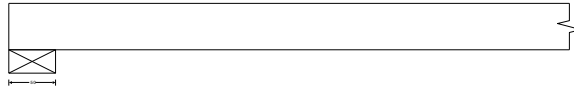
Beam loads

Permanent self weight of beam $\times 1$
 Permanent full UDL 0.060 kN/m
 Variable full UDL 0.750 kN/m

Analysis results

Design moment	$M = 0.075 \text{ kNm}$	Design shear	$F = 0.533 \text{ kN}$
Total load on member	$W_{\text{tot}} = 1.705 \text{ kN}$		
Reactions at support A	$R_{A_{\text{max}}} = 0.320 \text{ kN}$	$R_{A_{\text{min}}} = 0.320 \text{ kN}$	
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 0.018 \text{ kN}$		
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 0.197 \text{ kN}$		
Reactions at support B	$R_{B_{\text{max}}} = 1.065 \text{ kN}$	$R_{B_{\text{min}}} = 1.065 \text{ kN}$	
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 0.060 \text{ kN}$		
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 0.656 \text{ kN}$		
Reactions at support C	$R_{C_{\text{max}}} = 0.320 \text{ kN}$	$R_{C_{\text{min}}} = 0.320 \text{ kN}$	
Unfactored permanent load reaction at support C	$R_{C_{\text{Permanent}}} = 0.018 \text{ kN}$		
Unfactored variable load reaction at support C	$R_{C_{\text{Variable}}} = 0.197 \text{ kN}$		

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Timber section details

Breadth of section	$b = 50 \text{ mm}$	Depth of section	$h = 50 \text{ mm}$
Number of sections	$N = 1$	Breadth of member	$b_b = 50 \text{ mm}$
Timber strength class	C24		

Member details

Service class of timber	3	Load duration	Short-term
Length of span 1	$L_{s1} = 700 \text{ mm}$		
Length of span 2	$L_{s2} = 700 \text{ mm}$		
Length of bearing	$L_b = 50 \text{ mm}$		

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

Compression perpendicular to grain - cl.6.1.4

Design compressive stress	$\sigma_{c,90,d} = 0.426 \text{ N/mm}^2$	Design compressive strength	$f_{c,90,d} = 1.481 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing			

Bending - cl 6.1.6

Design bending stress	$\sigma_{m,d} = 3.580 \text{ N/mm}^2$	Design bending strength	$f_{m,d} = 17.709 \text{ N/mm}^2$
PASS - Design bending strength exceeds design bending stress			

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 0.477 \text{ N/mm}^2$	Permissible shear stress	$f_{v,d} = 2.369 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress			

Deflection - cl.7.2

Deflection limit	$\delta_{lim} = 2.800 \text{ mm}$	Total final deflection	$\delta_{fin} = 0.353 \text{ mm}$
PASS - Total final deflection is less than the deflection limit			

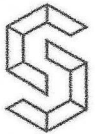
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Project: 82 MARINE CRESCENT

Part of Structure:

STRECKWARK.

Project No: 26-048

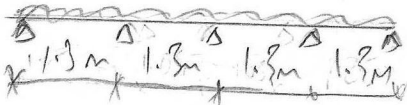
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THREE JOISTS SUPPORT BEAM.

TAKE THE REM IN MEOW THE SUPPL. ROOF. THIS HAS THE LARGEST TRIPLE SPAN
 & SUPPORTS THE LARGEST JOIST SPAN.



$$u_m = g h \pm \frac{0.7m + 0.7m}{2} \times 0.30 \text{ km}^2 \times 0.24 \text{ km}$$

gk. $\frac{0.7m + 0.7m}{2} = 2.50 \text{ km}^2 \cdot 1.40 \text{ km}$

$100 \times 50 \times 5.0$ mm LAM ON EDGE (MINOR AXIS) IS ADEQUATE (17% UTR).

PITCHES 2 TRACTOR axle = 0.8 kW
 TRACTOR model = 2.4 kW. } de oro RHINO pedestals.
 (< 10 kW).

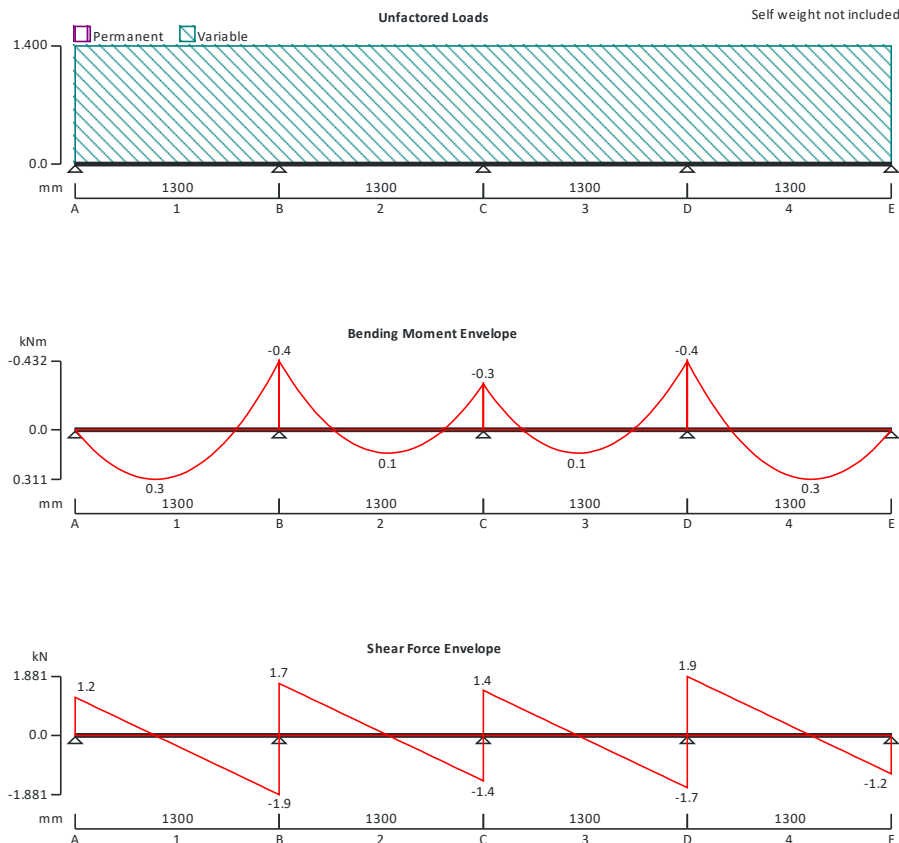
WE SPARE FOR BACK BEAMS. TO GLASS RAUSTRADRE.

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TYPICAL INTERNAL BEAM (WORST CASE LOAD)				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

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically restrained Rotationally free
Support D	Vertically restrained Rotationally free
Support E	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent full UDL 0.21 kN/m Variable full UDL 1.4 kN/m
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Analysis results

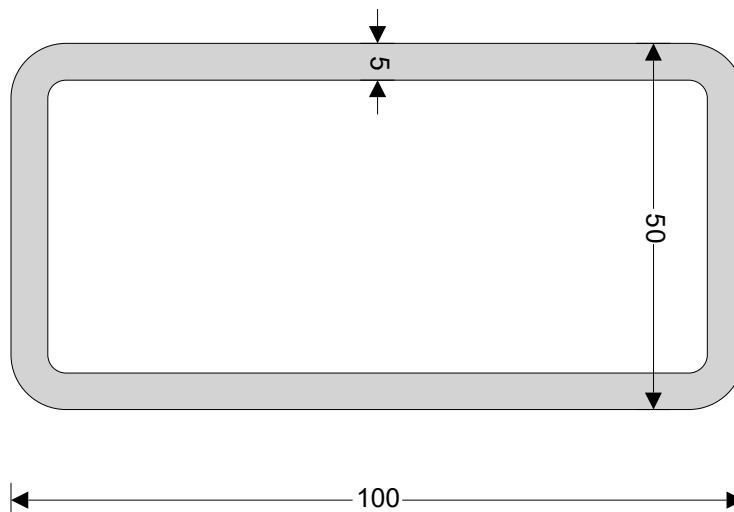
Maximum moment	$M_{\max} = 0.3 \text{ kNm}$	$M_{\min} = -0.4 \text{ kNm}$
Maximum moment span 1	$M_{s1_{\max}} = 0.3 \text{ kNm}$	$M_{s1_{\min}} = -0.4 \text{ kNm}$

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Maximum moment span 2	$M_{s2_max} = 0.1 \text{ kNm}$	$M_{s2_min} = -0.4 \text{ kNm}$
Maximum moment span 3	$M_{s3_max} = 0.1 \text{ kNm}$	$M_{s3_min} = -0.4 \text{ kNm}$
Maximum moment span 4	$M_{s4_max} = 0.3 \text{ kNm}$	$M_{s4_min} = -0.4 \text{ kNm}$
Maximum shear	$V_{max} = 1.9 \text{ kN}$	$V_{min} = -1.9 \text{ kN}$
Maximum shear span 1	$V_{s1_max} = 1.2 \text{ kN}$	$V_{s1_min} = -1.9 \text{ kN}$
Maximum shear span 2	$V_{s2_max} = 1.7 \text{ kN}$	$V_{s2_min} = -1.4 \text{ kN}$
Maximum shear span 3	$V_{s3_max} = 1.4 \text{ kN}$	$V_{s3_min} = -1.7 \text{ kN}$
Maximum shear span 4	$V_{s4_max} = 1.9 \text{ kN}$	$V_{s4_min} = -1.2 \text{ kN}$
Deflection	$\delta_{max} = 0.2 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Deflection span 1	$\delta_{s1_max} = 0.2 \text{ mm}$	$\delta_{s1_min} = 0 \text{ mm}$
Deflection span 2	$\delta_{s2_max} = 0.1 \text{ mm}$	$\delta_{s2_min} = 0 \text{ mm}$
Deflection span 3	$\delta_{s3_max} = 0.1 \text{ mm}$	$\delta_{s3_min} = 0 \text{ mm}$
Deflection span 4	$\delta_{s4_max} = 0.2 \text{ mm}$	$\delta_{s4_min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 1.2 \text{ kN}$	$R_{A_min} = 1.2 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 0.1 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 0.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 3.5 \text{ kN}$	$R_{B_min} = 3.5 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 0.3 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 2.1 \text{ kN}$	
Maximum reaction at support C	$R_{C_max} = 2.9 \text{ kN}$	$R_{C_min} = 2.9 \text{ kN}$
Unfactored permanent load reaction at support C	$R_{C_Permanent} = 0.3 \text{ kN}$	
Unfactored variable load reaction at support C	$R_{C_Variable} = 1.7 \text{ kN}$	
Maximum reaction at support D	$R_{D_max} = 3.5 \text{ kN}$	$R_{D_min} = 3.5 \text{ kN}$
Unfactored permanent load reaction at support D	$R_{D_Permanent} = 0.3 \text{ kN}$	
Unfactored variable load reaction at support D	$R_{D_Variable} = 2.1 \text{ kN}$	
Maximum reaction at support E	$R_{E_max} = 1.2 \text{ kN}$	$R_{E_min} = 1.2 \text{ kN}$
Unfactored permanent load reaction at support E	$R_{E_Permanent} = 0.1 \text{ kN}$	
Unfactored variable load reaction at support E	$R_{E_Variable} = 0.7 \text{ kN}$	

Section details

Section type **RHS 100x50x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))** Steel grade **S355H**



Section classification **Class 1**

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Check shear - Section 6.2.6

Design shear force $V_{Ed} = 2 \text{ kN}$

Design shear resistance $V_{c,Rd} = 93.8 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{Ed} = 0.4 \text{ kNm}$

Des.bending resist.moment $M_{c,Rd} = 9.2 \text{ kNm}$

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = 3.6 \text{ mm}$

Maximum deflection $\delta = 0.227 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

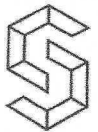
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Project: 82 MARINE CRESCENT

Part of Structure:

STAIRWELL

Project No: 26-048

Date:

14/09/2026

Engineer: P.W.

Sheet No:

BAULSTAKE SUPPORT TO EDGE BEAM - TORSION

THE BAULSTAKE CONTRIBUTES VERTICALLY FROM THE EDGE STEEL. THE
MAXIMUM SPAN BETWEEN LATERAL SUPPORTS IS 7.30m.

torsional capacity.

Scenario 1 - HANDRAIL = 0.74 kN/m APPLIED AT 1.1m ABOVE FFL KNEE
= $0.74 \text{ kN/m} \times 1.1 \text{ m} = 0.814 \text{ kNm (SLS)}$

Scenario 2 = WIND = $\frac{-wL^2}{8} = \frac{1.20 \text{ kN/m}^2 \times 1.1 \text{ m}^2}{2} = 0.73 \text{ kNm}$

CHECK SCENARIO 1: 0.814 kNm

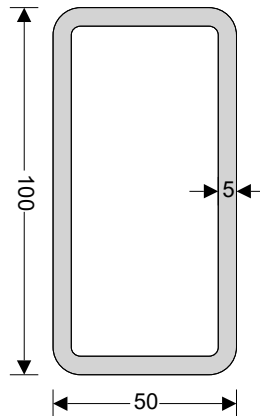
A 100 x 50 x 5.0 RHS IS ADEQUATE < 7% UTILISATION

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BALUSTRADE EDGE BEAM CHECK				1	
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STEEL BEAM TORSION DESIGN

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 2.0.04



Section details

Section type	RHS 100x50x5.0	Steel grade	S355
Design strength	$p_{yw} = p_y = 355 \text{ N/mm}^2$	Constant	$\varepsilon = 0.880$

Calculated torsional properties

Height of web	$d_x = b_y = D - 3 \times t = 85.0 \text{ mm}$
Width of flange	$b_x = d_y = B - 3 \times t = 35.0 \text{ mm}$
Shear area – y	$A_{vy} = A \times D / (D + B) = 9.2 \text{ cm}^2$
Shear area – x	$A_{vx} = A \times B / (D + B) = 4.6 \text{ cm}^2$
	$d_{vx} = A_{vx} / (2 \times t) = 45.8 \text{ mm}$
	$d_{vy} = A_{vy} / (2 \times t) = 91.5 \text{ mm}$
Plastic modulus of web	$S_{vx} = 2 \times t \times d_{vy}^2 / 4 = 21.0 \text{ cm}^3$
Plastic modulus of flange	$S_{vy} = 2 \times t \times d_{vx}^2 / 4 = 5.2 \text{ cm}^3$
Statical moment at mid-depth	$Q_w = S_x / 2 = 21 \text{ cm}^3$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span	$L = 1300 \text{ mm}$		
Length of segment LTB	$L_{LT} = 2000 \text{ mm}$	Effective length for LTB	$L_{E_LT} = 2200 \text{ mm}$

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for ults design

Applied shear force	$F_{vy} = 0.1 \text{ kN}$	Maximum bending moment	$M_{LT} = M_x = 0.20 \text{ kNm}$
Applied torque	$T_q = 0.81 \text{ kNm}$	Minor axis bending moment	$M_y = 0 \text{ kNm}$
Compression force	$F_c = 0 \text{ kN}$		

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) $m_{LT} = 1.000$

Torsional deflection analysis

Beam is torsion fixed at each end. (as defined in SCI-P-057 section 2.1.6)

Max torque (at supports)	$T_o = 0.41 \text{ kNm}$	Avg torque support & CL	$T_{av} = 0.20 \text{ kNm}$
Max. angle of twist (midspan)	$\phi = 0.001 \text{ rads}$		

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Section classification

$$\begin{aligned}
 b_x / t &= 7.0 & d_x / t &= 17.0 \\
 b_y / t &= 17.0 & d_y / t &= 7.0 \\
 r_{1sx} &= 0.000 & r_{1sy} &= 0.000 \\
 r_{2s} &= 0.000
 \end{aligned}$$

Section classification is plastic

Shear capacity (parallel to y-axis)

$$\text{Design shear force} \quad F_{vy} = 0.1 \text{ kN} \quad \text{Design shear resist (cl. 4.2.3)} \quad P_{vy} = 195.0 \text{ kN}$$

Pass - Shear

Moment capacity (x-axis)

$$\text{Design bending moment} \quad M_x = 0.2 \text{ kNm} \quad \text{Mnt cap low shear (cl. 4.2.5.1)} \quad M_{cx} = 14.2 \text{ kNm}$$

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

$$\text{Effective length for LTB} \quad L_{E_LT} = 2200 \text{ mm}$$

$$\text{Slenderness ratio - cl 4.3.6.5} \quad \lambda = 111$$

$$D / B = 2.0$$

LTB check not required

$$\text{Buckling resistance mnt} \quad M_b = M_{cx} = 14.2 \text{ kNm}$$

Buckling under combined bending & torsion - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

$$\text{Max angle of twist} \quad \phi = 0.001 \text{ rads} \quad \text{Induced minor axis moment} \quad M_{yt} = 0.00 \text{ kNm}$$

$$\text{Norm stress corner due to } M_{yt} \quad \sigma_{byt} = 0 \text{ N/mm}^2 \quad \text{Interaction index} \quad i_b = 0.01$$

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

$$\text{Max. direct stress due to } M_x \quad \sigma_{bx} = M_x / Z_x = 6 \text{ N/mm}^2$$

$$\text{Combined stress - eqn 2.22} \quad \sigma_{bx} + \sigma_{byt} = 6 \text{ N/mm}^2 \quad \text{Design strength} \quad p_y = 355 \text{ N/mm}^2$$

Pass - Local capacity

Combined shear stresses SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

$$\text{Max. shear stress bending} \quad \tau_{bw} = 0 \text{ N/mm}^2 \quad \text{Max. shear stresses torsion} \quad \tau_t = 11 \text{ N/mm}^2$$

$$\text{Amplified shear stress torsion} \quad \tau_{vt} = 11 \text{ N/mm}^2 \quad \text{Combined shear bend & tors} \quad \tau = 11 \text{ N/mm}^2$$

$$\text{Shear strength} \quad p_v = 213 \text{ N/mm}^2$$

Pass - Combined shear stresses

Twist check

$$\text{Total applied torque} \quad T_{qu} = 0.81 \text{ kNm}$$

$$\text{Max twist under sls loading} \quad \phi_{sls} = 0.07 \text{ deg}$$

$$\text{Twist limit} \quad \phi_{lim} = 1.00 \text{ deg}$$

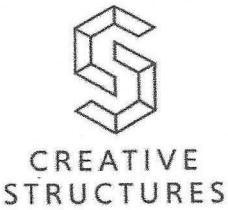
Pass - Twist

Deflection

$$\text{Maximum y-axis deflection} \quad \delta_{y_max} = 0.1 \text{ mm} \quad \text{Deflection limit - cl. 2.5.2} \quad \delta_{lim} = 3.6 \text{ mm}$$

Pass - Deflection within specified limit

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Project: 82 MARINE CRESCENT		Part of Structure: STEELWORK.	
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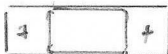
CONNECTION DETAILS TO SUB-STRUCTURE

①

TORSION RESISTANT OF INNER TIES TO PERPENDICULAR BEAMS.

$$TORSION = \frac{1.3m + 1.3m}{2} \times 0.814 \text{ kNm} = 1.06 \text{ kNm (SLS)}$$

$$1.5 > 1.06 \text{ kNm} = 1.59 \text{ kNm ULS}$$

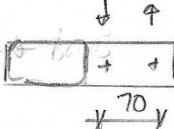
25# 
$$\frac{1.59 \text{ kNm}}{0.025m} = \frac{63.6 \text{ kN}}{2 \text{ BOLTS}} = 31.8 \text{ kN PER BOLT}$$

M12 G8.8.
TENSILE RESISTANCE PER BOLT (F_{ED}) = 48.6 kN ∴ OK

② - OUTER/EDGE BEAM CONNECTION

$$TORSION = \frac{1.3m}{2} \times 0.814 \text{ kNm} = 0.529 \text{ kNm (SLS)}$$

$$1.5 > 0.529 \text{ kNm} = 0.794 \text{ kNm (ULS)}$$

20# 
$$ASH \text{ RM} = \frac{0.794 \text{ kNm}}{0.07m} = 11.34 \text{ kN}$$

SHARZ CAPACITY OF M12 G8.8 BOLT = 27.5 kN. ∴ OK

CALCULATION SHEET

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Project: 82 MARINE CRESCENT		Part of Structure: WIND LADING	
Project No: 26-048	Date: 14/09/2026	Engineer: P.W.	Sheet No:

WIND LADING

(conservative)

$$\text{ON LONG EDGE} = \sim 13\text{m} \times 1.1\text{m} \times 1.20\text{kg/m}^2 = 17.16\text{ kW (SLS)}$$

$$\text{WEIGHT OF SUBSTRATE ONLY} = 116\text{m lengths} \times 10.8\text{ kg/m} = 1252.8\text{ kg} = 12.2\text{ kW}$$

$$\text{WEIGHT OF GLASS (21.5m)} = 26\text{m} \times 1.1\text{m} \times 68\text{ kg/m}^2 = 1944\text{ kg} = 19\text{ kW}$$

$$\text{WEIGHT OF DECKING + JOINTS} = 55\text{m}^2 \times 30\text{ kg/m}^2 = 1650\text{ kg} = 16.18\text{ kW}$$

$$\Sigma = 47.38\text{ kW}$$

FACTORED APPROACH

$$1.5 \times \text{WIND} : 0.9 \times \text{DECK (SLS)}$$

$$1.5 \times 17.16\text{ kW} : 0.9 \times 47.38\text{ kW}$$

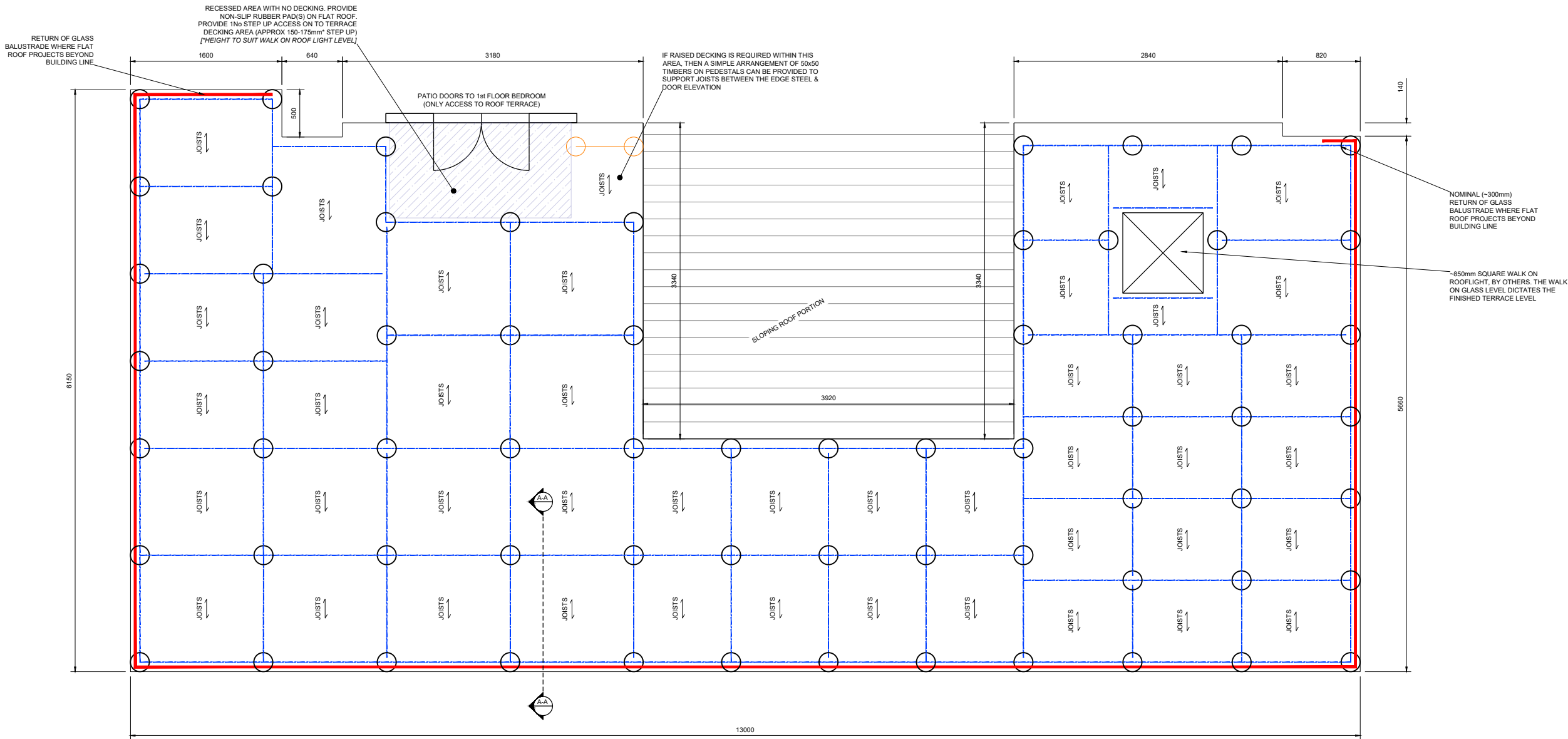
$$25.74\text{ kW} : 42.64\text{ kW}$$

$$\text{FOS} = 1.65 \quad \therefore \underline{\text{OK}}$$

THE DECKING & GLASS WEIGHTS HAVE BEEN CONFIRMED AS THE SPEC!

SECTION 3 – DRAWING(S)

The following drawing shows the arrangement of the steel sub-frame, a typical section and the key steel connection details.



PLAN VIEW ON TERRACE
SUB-FRAME & JOIST LAYOUT
(1:25)

BLUE LINES DENOTES 100x50x5.0 RHS STEEL FRAME HDG TO 85m. ALL BEAMS ON EDGE (50mm DEEP)

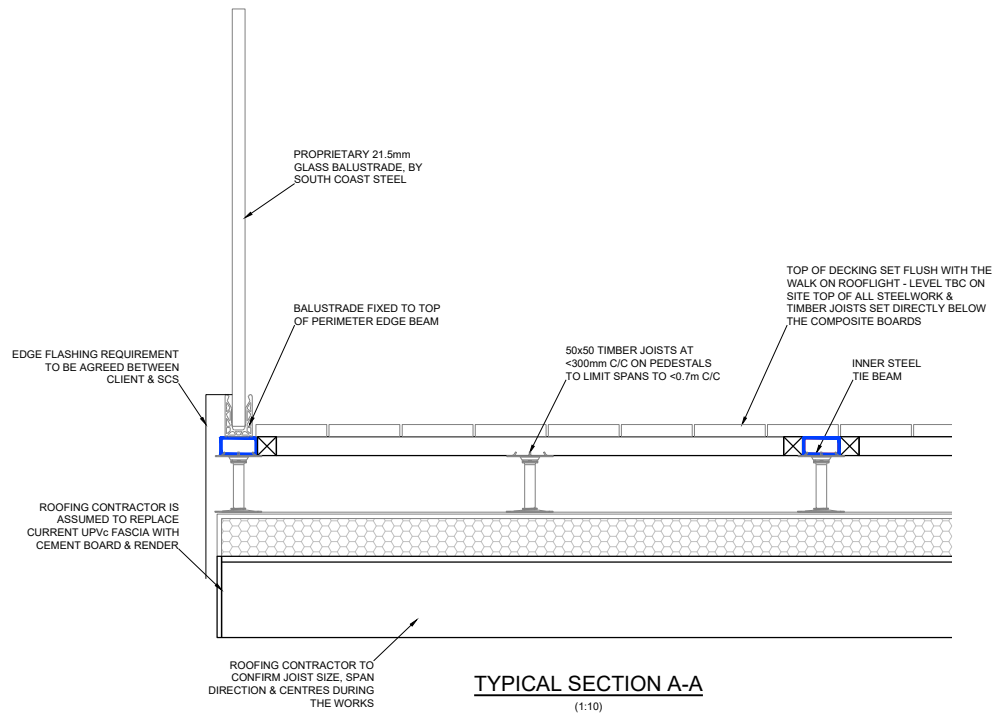
ORANGE LINES DENOTES 50x50 EDGE TIMBER TRIMMER, IF REQUIRED TO SMALL AREA OF INFILL

RED LINES DENOTES PERIMETER 21.5mm GLASS BALUSTRADE CANTILEVERING UP FROM PERIMETER FRAME

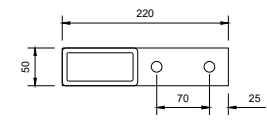
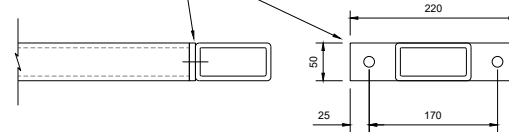
JOISTS 50x50 C24 JOISTS AT 300mm C/C
JOISTS TO BE SUPPORTED AT MID-SPAN ON ADJUSTABLE PEDESTALS WITH MAX CLEAR SPAN OF 0.7m C/C
COMPOSITE DECKING BOARDS PERPENDICULAR

ADJUSTABLE PEDESTAL SUPPORTS TO THE STEEL SUB-FRAME - PEDESTALS SAT ON RUBBER PADS

ALL DIMENSIONS SHOWN ARE INDICATIVE BASED ON TAPE MEASURE SURVEY 10/09/2026. ALL FINAL DIMENSIONS ARE TBC ON SITE BY SOUTH COAST STEEL



ALL TERRACE STEELWORK TO BE 100x50 RHSs HOT DIP GALVANISED TO 85 MICRONS. ALL CONNECTIONS TO BE MADE USING 10mm THICK ENDPLATE (AS SHOWN ADJACENT) FIXED WITH SCREWED & TAPPED M12 BOLTS



STEEL SUB-FRAME
CONNECTION DETAIL(S)
(1:5)

- NOTES:-
- THIS DRAWING IS NOT TO BE SCALED, WORK FROM GIVEN DIMENSIONS ONLY.
 - THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS, SERVICE ENGINEERS AND SPECIALISTS DRAWINGS.
 - ANY DISCREPANCIES OR VARIATIONS ARE TO BE REPORTED IMMEDIATELY TO ENGINEER FOR CLARIFICATION BEFORE THE AFFECTED WORK COMMENCES.
 - ALL DIMENSIONS ARE TO BE CHECKED ON SITE. ANY DISCREPANCIES TO BE REPORTED TO ENGINEER IMMEDIATELY BEFORE THE AFFECTED WORK COMMENCES.
 - ALL PROPRIETARY PRODUCTS ARE TO BE USED IN ACCORDANCE WITH THE MANUFACTURERS DETAILS AND SPECIFICATIONS.
 - THE CONTRACTOR IS TO ENSURE THAT TEMPORARY LOADS PLACED ONTO THE NEW STRUCTURES SHALL BE LESS THAN THE LOADS FOR WHICH IT HAS BEEN DESIGNED.
 - THE CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY WORKS AND FOR THE STABILITY OF THE WORKS IN PROGRESS.
 - THESE DRAWINGS CONSTITUTE A FULL DESIGN FOR BUILDING CONTROL APPROVAL. THEY ARE NOT CONSTRUCTION STAGE DRAWINGS UNTIL BUILDING CONTROL HAVE APPROVED THE DESIGN(S).

WARNING IDENTIFICATION

PARTICULAR ATTENTION TO BE PAID WHERE THE HAZARD SYMBOL IS PLACED ON A DRAWING. THIS SYMBOL HIGHLIGHTS A PARTICULAR CONCERN IN RELATION TO THE DESIGN, SITE WORKING OR WHERE THE CONTRACTORS ATTENTION IS REQUIRED.

MEASURES TO MITIGATE RISK SHOULD BE CONSIDERED AT ALL STAGES OF DESIGN DEVELOPMENT AND CONSTRUCTION.

COORDINATION HIGHLIGHT

WHERE THE BLUE HIGHLIGHT SYMBOL IS PLACED ON A DRAWING PARTICULAR ATTENTION IS REQUIRED WITH REGARD TO COORDINATION.

STABILITY REQUIREMENT HIGHLIGHT

WHERE THE GREEN HIGHLIGHT SYMBOL IS PLACED ON A DRAWING THIS REFERS TO THIS ELEMENT OF THE DESIGN BEING CRITICAL TO THE STABILITY STRATEGY

REV	DESCRIPTION	DATE	BY
B1	BUILDING REGULATIONS	14.09.2026	PW

BUILDING REGULATIONS



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Client
LEAH & PATRICK CLOUGH

Project
82 MARINE CRESCENT,
WORTHING, BN12 4JH

Title
BALUSTRADE & TERRACE
SUB-FRAME DESIGN

Date 14.09.2026 Checked 14.09.2026

Drawing Sheet Size A1

Scale 1:25, 1:10, 1:5 Engineer PW

Status BUILDING REGULATIONS

Project Number	Drawing Number	Revision
26-048	01	B1