Structural Calculations for BALCONY 2 pass-through 1.8m privacy screen system: handrail with & without 58 x 4mm internal steel reinforcing bar: 60 x 60 x 5 SHS posts: alternative base plate options

Our ref: B2SPWB6060300150BP311016RA

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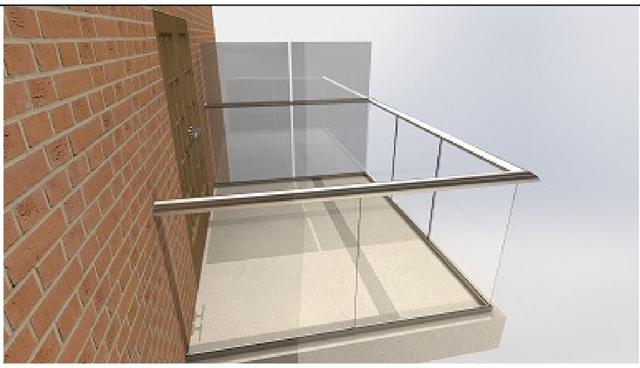
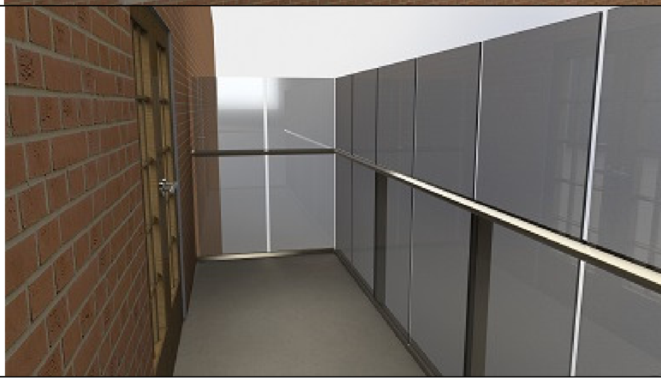
Revised to include additional base plate option: July 2017
Revised with new wind load parameters: September 2018



Balcony 2 pass-through privacy screen fixed on one side only



Balcony 2 pass-through privacy screen on a 3 sided balcony

DESIGN TO EUROCODES & CURRENT BRITISH STANDARDS

Design standards:

EN 1990	Eurocode 0:	Basis of structural design.
EN 1991	Eurocode 1:	Actions on structures.
EN 1991-1-4:2002 + A1 2010 + NA	Eurocode 1:	Actions on structures – wind actions.
EN 1993	Eurocode 3:	Design of steel structures.
EN 1999	Eurocode 9:	Design of aluminium structures.
BS EN 1990:2002 + A1:2005	Eurocode:	UK National annex for Eurocode
BS 6180:2011		British Standard: Barriers in and about buildings.

<u>Design imposed loads:</u>
Occupancy class/es for which this design applies (Table 2: BS6180:2011)

Domestic and residential activities (i) & (ii)
Office and work areas not included elsewhere (iii), (iv) & (v)
Areas without obstacles for moving people and not susceptible to overcrowding (viii) & (ix).

Service load on handrail Q_k = 0.74 kN/m uniformly distributed line load acting 1100mm above finished floor level. (Table 2: BS6180:2011)

Service load applied to the glass infill Q_{k1} = A uniformly distributed load of 1.0 kN/m^2

Point load on glass infill = 0.50 kN applied to any part of the glass infill panels

Balcony 2 pass-through 1.8m privacy screens: ***PAGE 2*** **(B2SPWB6060300150BP311016RA)**

Table 2 Minimum horizontal imposed loads for parapets, barriers and balustrades

Type of occupancy for part of the building or structure	Examples of specific use	Horizontal uniformly distributed line load (kN/m)	Uniformly distributed load applied to the infill (kN/m²)	A point load applied to part of the infill (kN)
Domestic and residential activities	(i) All areas within or serving exclusively one single family dwelling including stairs, landings, etc. but excluding external balconies and edges of roofs	0.36	0.5	0.25
	(ii) Other residential, i.e. houses of multiple occupancy and balconies, including Juliette balconies and edges of roofs in single family dwellings	0.74	1.0	0.5
Offices and work areas not included elsewhere, including storage areas	(iii) Light access stairs and gangways not more than 600 mm wide	0.22	—	—
	(iv) Light pedestrian traffic routes in industrial and storage buildings except designated escape routes	0.36	0.5	0.25
	(v) Areas not susceptile to overcrowding in office and institutional buildings, also industrial and storage buildings except as given above	0.74	1.0	0.5
Areas where people might congregate	(vi) Areas having fixed seating within 530 mm of the barrier, balustrade or parapet	1.5	1.5	1.5
Areas with tables or fixed seatings	(vii) Restaurants and bars	1.5	1.5	1.5
Areas without obstacles for moving people and not susceptible to overcrowding	(viii) Stairs, landings, corridors, ramps	0.74	1.0	0.5
	(ix) External balconies including Juliette balconies and edges of roofs. Footways and pavements within building curtilage adjacent to basement/sunken areas	0.74	1.0	0.5

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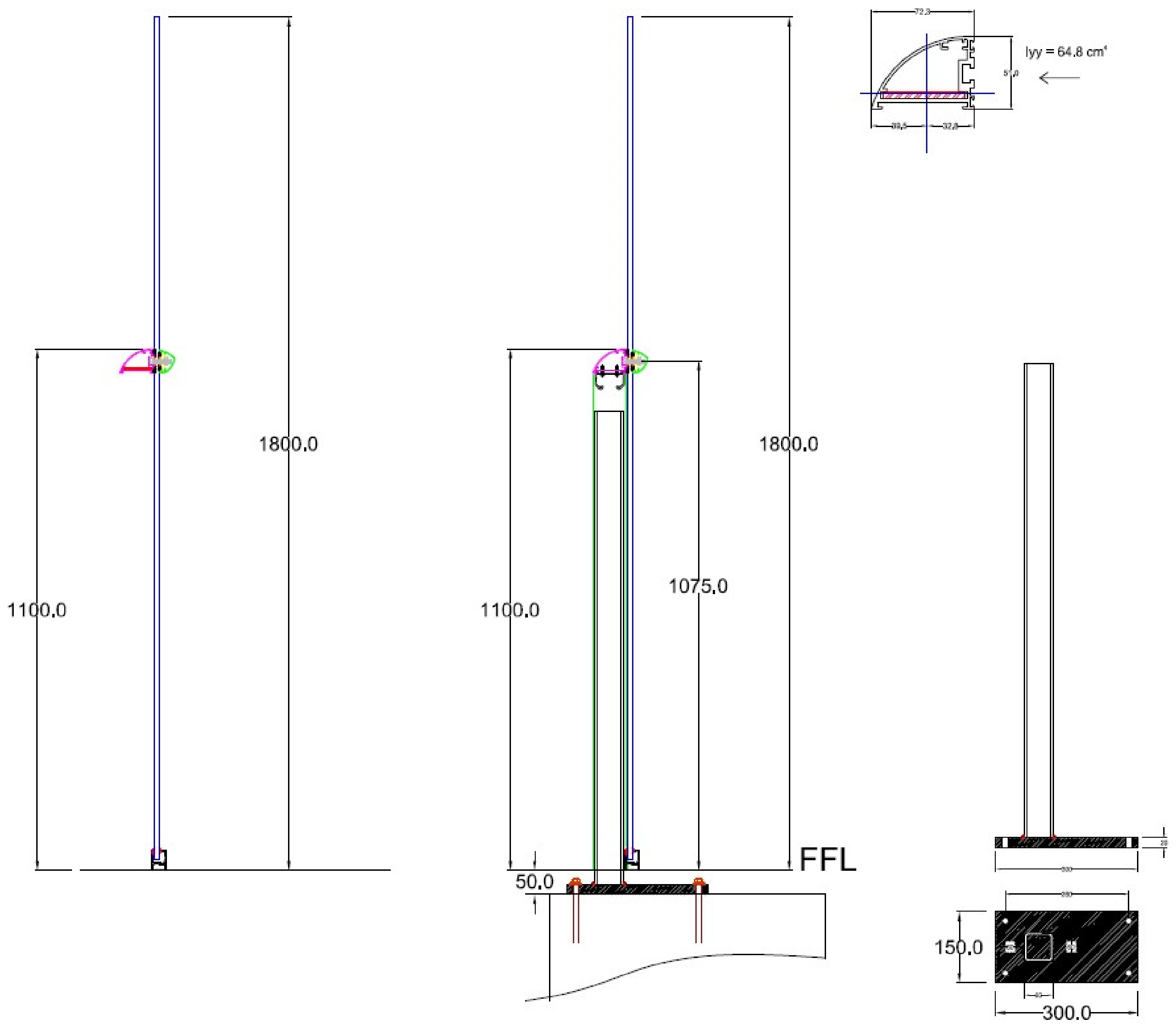
Table 2: BS6180:2011

- These loads are considered as three separate load cases. They are not combined. Wind loading is also considered as a separate design case.
- Factored loads are used for checking the limit state of static strength of a member.
- The service loads are multiplied by a partial factor for variable action $\gamma_{Q,1}$ of 1.5 to give the ultimate design load for leading variable action.

Deflection:

- All structural members deflect to some extent under load. Service loads are used to calculate deflections.
- The total displacement of any point of a barrier from its original unloaded position under the action of service loads is limited to 25mm.

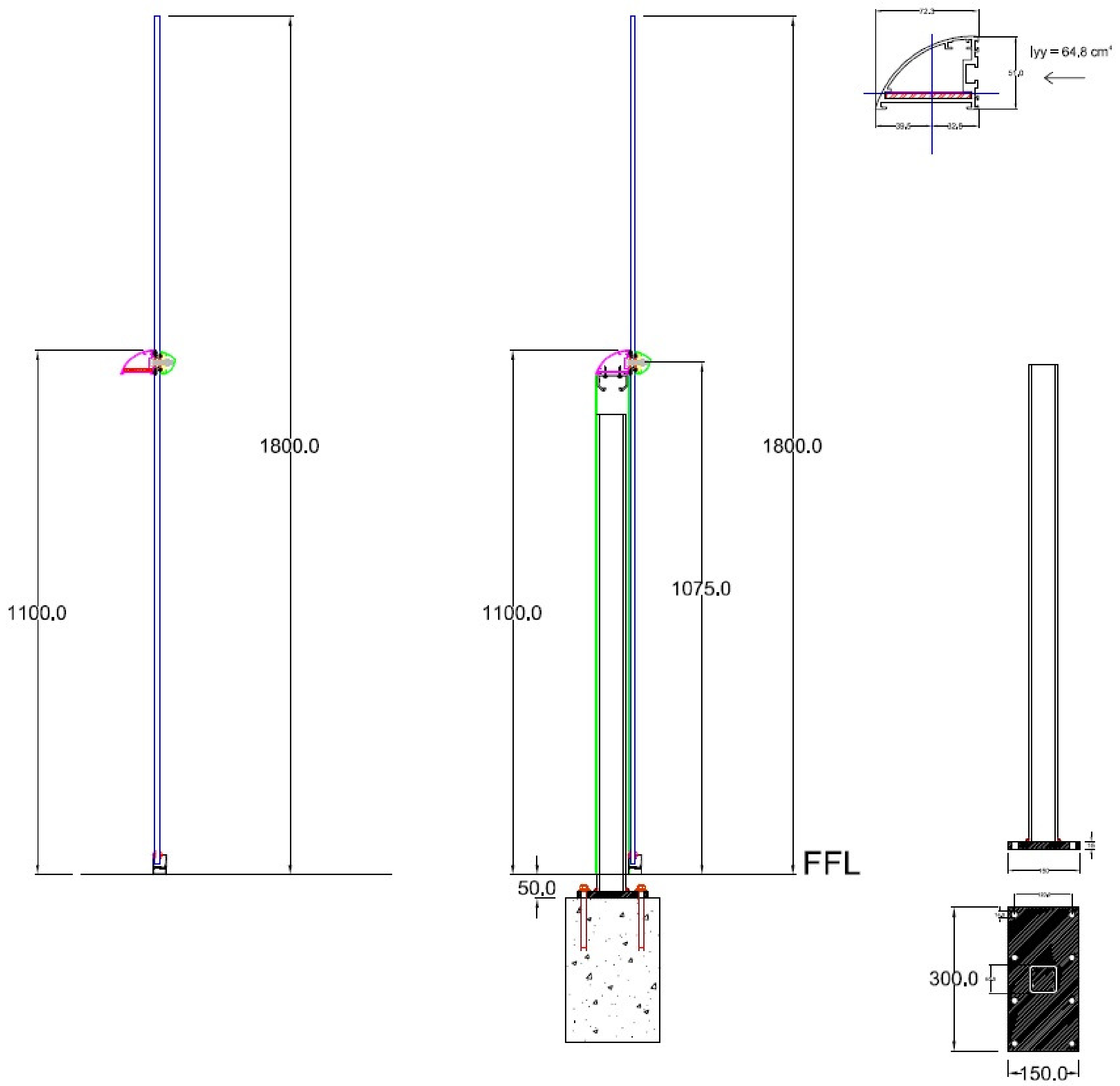
Balcony 2 pass-through 1.8m privacy screens: ***PAGE 3*** **(B2SPWB6060300150BP311016RA)**



Section of Balcony 2 pass-through 1.8m privacy screen system, post and option 1 base plate details.

Balcony 2 pass-through 1.8m privacy screen system: ***PAGE 4*** **(B2SPWB6060300150BP311016RA)**

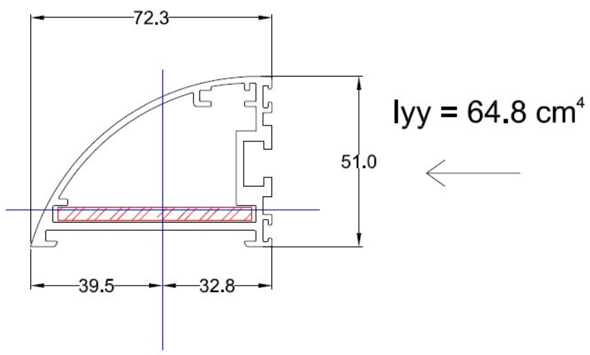
Section properties of handrail:



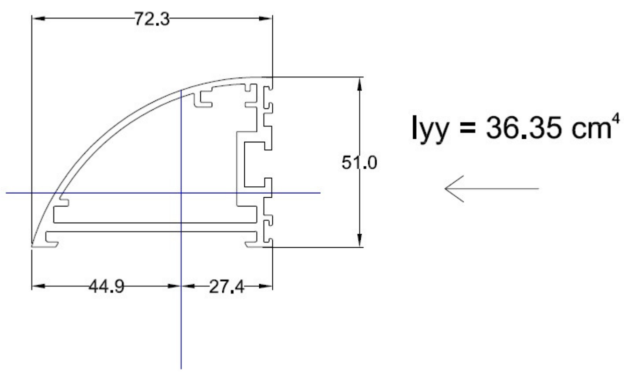
Section of Balcony 2 pass-through 1.8m privacy screen system, post and option 2 base plate details.

Balcony 2 pass-through 1.8m privacy screens:

Section properties with bar



Section properties without bar



Handrail with reinforcing bar:

Material type	=	Extruded aluminium type 6063 T5		
Characteristic 0.2% proof stress	=	f_o	=	130 N/mm^2
Characteristic ultimate tensile strength	=	f_u	=	175 N/mm^2
Modulus of elasticity	=	E	=	70 000 N/mm^2
Shear modulus	=	G	=	27 000 N/mm^2
Moment of inertia about the y-y axis	=	I_{yy}	=	64.8 cm^4
Least elastic modulus about the y-y axis	=	W_{el}	=	16.405 cm^3
Partial factor for material properties	=	γ_{M1}	=	1.10
Value of shape factor	=	α	=	W_{pl}/W_{el}
(conservative value assumed)			=	1.2 say
Design ultimate resistance to bending about the y-y axis	=	M_{Rd}	=	$M_{o,Rd}$

$$= \alpha W_{el} f_o / \gamma_{M1}$$

$$= \frac{1.2 \times 16.405\ cm^3 \times 130\ N/mm^2 \times (10)^{-3}}{1.1}$$

= **2.327 kNm**

Balcony 2 pass-through 1.8m privacy screen system:

Design for imposed loads:

Design ultimate horizontal imposed load on handrail F = $\frac{1.50\ kN/m^2 \times 1.8 \times 0.90}{1.075}$ = 2.26 kN/m

Design horizontal moment on handrail between points of support, assuming simply supported spans (worst case) M = $\frac{F L^2}{8}$

Allowable span between points of support based upon the moment capacity of the handrail L = $\left[\frac{8 \times M_{Rd}}{F}\right]^{0.5}$

= $\left[\frac{8 \times 2.327\ kNm}{2.26}\right]^{0.5}$

= 2.87m say = 2.85m

In terms of bending capacity the handrail can support the design ultimate horizontal imposed load for spans up to 2.85m simply supported between points of support. (ie. A handrail wall fixing, or a handrail corner joint.)

Deflection: Handrail with reinforcing bar:

For a single span simply supported handrail the service load deflection is limited to a maximum of 25mm.

Deflection (Δ) of a simply supported span (L) with an imposed UDL load (F) $\Delta = \frac{5 F L^4}{384 E I}$

Service load on the handrail F = 2.26 / 1.5 = 1.507 kN/m

For a handrail span of 2.85m simply supported $\Delta = \frac{5\ (1507 \times 2.85)\ (2850)^3}{384 \times 70\ 000 \times 64.80 \times (10)^4}$

= 28.54mm > 25mm not OK

Deflection therefore governs the allowable simply supported span of the handrail.

For a handrail span of 2.75m simply supported $\Delta = \frac{5\ (1507 \times 2.75)\ (2750)^3}{384 \times 70\ 000 \times 64.80 \times (10)^4}$

= 24.74mm < 25mm OK

Summary

On single span and corner balconies, the Balcony 2 pass-through handrail has adequate moment capacity to resist the design ultimate imposed loading on spans up to 2.75 metres between points of support (i.e. a wall fixing or a handrail corner joint) without exceeding the service load deflection limit of 25mm.

The allowable span is checked for the design wind loading. See later calculations.

Wind load design:

Design notes:
Design wind loads are influenced by a number of variable factors. These include site location, site altitude above sea level, type of terrain, height of privacy screen above ground level and screen geometry.

These parameters and conditions are defined in BS EN 1991-1-4:2002 + A1: 2010 '*Actions on structures – wind actions' &* UK National Annex to EN 1991-1-4:2002 + A1:2010. We have chosen to prepare a calculation based on certain conditions, resulting in specific coefficients.

The formula applied results in an overall c***haracteristic wind pressure***. The design and calculation will be relevant not only to the conditions specified herein but to any combination of factors that result in a characteristic wind pressure that is equal or less than the one specified in the calculation.

Sites that have a ***characteristic wind pressure*** that exceeds **1.0 kN/m^2** as seen on page 8 below require separate calculation.

a) Sites located geographically within the 23m/sec isopleth in Figure NA1 of the UK National Annex. This covers most of England and the eastern half of Wales.

b) Site altitude 50m maximum above sea level.

c) Top of screen located 10m maximum above ground level.

d) Site located in a coastal area exposed to the open sea, 1.0 km or less from the shoreline, terrain category 0 of BS EN 1991 Table 4.1. This is the most severe exposure category. Smaller wind load coefficients apply to less exposed inland sites, terrain categories 1 to 1V.

e) Sites with no significant orography in relation to wind effects. Increased wind load factors apply to sites near the top of isolated hills, ridges, cliffs or escarpments.

f) Directional, seasonal, and probability factors are all taken as normal, for which the relevant factor is 1.0. This is a slightly conservative approach.

g) Wind loading is considered as a separate design case. It is not considered in combination with other design loads.

Balcony 2 pass-through 1.8m privacy screen system: *PAGE 8* **(B2SPWB6060300150BP311016RA)**

Wind load design:

Basic site wind speed	$V_{b\ map}$	=	23 m/sec	(Figure NA.1)
Site altitude above sea level	A	=	50m	
Height to top of screen above site level	z	=	10m	
Site altitude factor	C_{alt}	=	1.05	(Eqn. NA.2a)
Directional factor	C_{dir}	=	1.0	
Seasonal factor	C_{season}	=	1.0	
Probability factor	C_{prob}	=	1.0	
Site wind speed	V_b	=	$V_{b\ map}(C_{dir} \times C_{season} \times C_{prob})\ (C_{alt})$	
		=	23 m/sec x 1.05	
		=	24.15 m/sec	
Site wind pressure	q_b	=	$0.613\ (V_b)^2$	
		=	$0.613\ (24.15)^2$	
		=	357.5 N/m^2	
Exposure factor	C_e (z)	=	2.80	(Figure NA.7)
Peak velocity pressure **(characteristic wind pressure)**	q_p	=	$q_b \times C_e$ (z)	
		=	0.357 x 2.80	
		=	**1.0 kN/m^2**	
Partial safety factor for leading variable action	γ_{Q1}	=	1.50	
Ultimate design wind pressure		=	1.0 kN/m^2 x 1.5	
		=	1.50 kN/m^2	

Summary:

Characteristic wind pressure = **1.0 kN/m^2**

Ultimate design wind pressure = **1.5 kN/m^2**

Wind loads and the design imposed UDL on the glass (as listed on pages 1 and 2) are therefore the same for the wind parameters adopted in these calculations.

Ultimate cantilever moment due to the design imposed & wind loads on the glass above the handrail fixing point

$$= \frac{1.5\ \text{kN/m}^2 \times (0.725)^2}{2}$$

= 0.394 kNm/m

Balcony 2 pass-through 1.8m privacy screens: ***PAGE 9*** **(B2SPWB6060300150BP311016RA)**

Wind load design:

Glass panels:
10mm thick thermally toughened soda silicate safety glass with smooth float 'as produced' finish and polished edges. Glass panels can be of any length. For design purposes a nominal glass panel width of 1000mm has been used.

Design standard: Institution of Structural Engineers publication 'Structural use of glass in building (second edition) February 2014.'

Characteristic design strength of glass = 120 N/mm^2

Ultimate design stress $f_{g;d} = \dfrac{K_{mod} \times K_{sp} \times K_{g;k}}{\gamma_{M;A}} + \dfrac{k_v (f_{b;k} - f_{g;k})}{\gamma_{M;V}}$

where

K_{mod} = 30 second duration factor
= 0.89 for domestic balustrades

K_{sp} = glass surface profile factor
= 1.0 for float glass 'as produced'

$f_{g;k}$ = characteristic strength of basic annealed glass
= 45 N/mm^2

K_v = manufacturing process strengthening factor
= 1.0 for horizontal toughening

$f_{b;k}$ = characteristic bending strength of prestressed glass
= 120 N/mm^2

$\gamma_{M;A}$ = material partial factor
= 1.6 for basic annealed glass

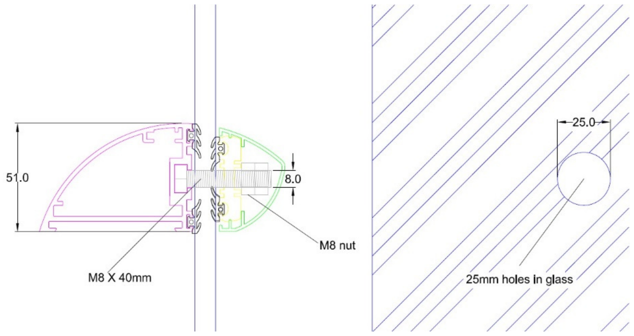
$\gamma_{M;V}$ = material partial factor
= 1.2 for surface prestressed (toughened) glass

Ultimate design stress $f_{g;d} = \dfrac{0.89 \times 1.0 \times 45}{1.6} + \dfrac{1.0\,(120 - 45)}{1.2}$

= **87.53 N/mm^2**

Glass design:

The maximum bending moment on the glass occurs at the handrail fixing point. In this position the glass is connected to the handrail using 3 or 4 M8 bolts per 1000mm wide glass panel. The bolts are pre-set in the handrail channel and pass through 25mm diameter holes in the glass. The minimum effective width of a 1000mm panel at the point of maximum moment is therefore 900mm. The glass is designed to withstand the wind loading calculated on pages 7 and 8 of these calculations. For the wind load parameters chosen, this happens to be equal to the design imposed loads listed on pages 1 and 2.



Inertia of glass panel 10mm thick x 1000mm wide (effectively 900mm) $I_{xx} = \dfrac{900 \times (10)^3}{12} = 75000\ mm^4$

Modulus of glass panel 10mm thick x 1000mm wide (effectively 900mm) $Z_{xx} = \dfrac{900 \times (10)^2}{6} = 15000\ mm^3$

Ultimate moment capacity of glass 10mm thick x 900mm effective width $M_u = f_{g:d} \times Z$

$= 87.53\ N/mm^2 \times 15000 \times (10)^{-6}$

$=$ **1.31 kNm/m**

Cantilever moment due to imposed design ultimate load of 1.5 kN/m^2 $Mu = \dfrac{1.50\ kN/m^2 \times (0.725)^2}{2} = 0.394\ kNm/m$ $= < 1.31\ kNm/m$ OK

Glass deflection – UDL loading:

Consider service load deflection of the glass as a vertical cantilever above the handrail.

Service imposed UDL on cantilever $= 1.0\ kN/m^2$

Service load deflection due of 725mm cantilever due to a UDL of 1.0 kN/m^2 $\Delta = \dfrac{W L^3}{8 E I} = \dfrac{(1000 \times 0.725)(725)^3}{8 \times 70000 \times 75000}$

$= 6.58mm < span/65$ OK

Additional deflection occurs due to the slope of the glass at the handrail.

Slope of glass at the handrail due to the service imposed load of 1.0 kN/m^2 $\phi = \dfrac{M L}{3 E I}$

where M = BM on the cantilever = 0.263 kNm/m

$= 1.0\ kN/m^2 \times (0.725)^2 /2$

L = height of handrail above FFL = 1075mm

$\phi = \dfrac{0.263\ kNm \times (10)^6}{3 \times 70000 \times 75000} = 1.67 \times (10)^{-5}$ radians

Deflection of cantilever due to this slope $= 725mm \times 1.5 \times (10)^{-5}$ radians $= 0.0109mm$

This is very small and can be neglected for practical design purposes.

Glass deflection (cont):

Point load:

Consider service load deflection due to a point load of 0.5 kN acting at the centre of the infill between the handrail and balcony floor.

Conservatively, assume that this point load is carried by a 300mm wide vertical strip of glass.

Inertia of glass 10mm thick x 300mm long = 0.30 x 83333 = 25000 mm^4

Deflection due to point load applied at mid-span

$$\Delta = \frac{P L^3}{48 E I} = \frac{500 \times (1000)^3}{48 \times 70\,000 \times 25000} = 5.95\text{mm} < \text{span}/65 = \text{OK}$$

UDL on glass below handrail:

Consider service load deflection due to a UDL of 1.0 kN/m^2 acting on the glass infill between the handrail and the balcony floor.

BM due to service UDL of 1.0 kN/m^2

$$\Delta = \frac{5 w L^4}{384 E I} = \frac{5 (1000 \times 1.075) (1075)^3}{384 \times 70000 \times 83333} = 2.98 \text{ mm} < \frac{\text{span}}{65} \quad \text{OK}$$

Glass summary:

The 10mm thermally toughened safety glass has adequate moment capacity to resist the ultimate design imposed and wind loads and is within allowable service load deflection limitations.

Handrail (with bar) – single span and corner screens:

Working load imposed force on handrail = $\frac{1.00 \text{ kN/m}^2 \times 1.8 \times 0.90}{1.075}$ = 1.507 kN/m

(moments taken about the underside of the bottom rail)
(ie. where the bottom rail is fixed to the decking)

Ultimate imposed load on handrail = 1.507 x 1.5 = 2.26 kN/m

Ultimate moment capacity of handrail about the y-y axis = 2.327 kNm

Allowable span of handrail between points of support based upon the moment capacity of the handrail = $\left[\frac{8 \times 2.327 \text{ kNm}}{2.26}\right]^{0.5}$ = 2.87m say 2.85m

Service load deflection of handrail for a simply supported span of 2.85m = $\frac{5 \times (1507 \times 2.85) (2850)^3}{384 \times 70000 \times 64.8 \times (10)^4}$ = 28.54mm > 25mm

Reduce span to 2.75m Δ = 24.74 mm < 25mm = OK

Summary: Service load deflection limitations restrict the simply supported span of the handrail to 2.75m between points of support (ie. a wall fixing or a standard handrail corner joint).

Balcony 2 pass-through 1.8m privacy screens: ***PAGE 12*** **(B2SPWB6060300150BP311016RA)**

Longer screens:

On longer screens the handrail (without bar) is used in conjunction with 60 x 60 x 5 SHS steel posts at 1.925m maximum spacing to support the handrail. The handrail without the strengthening bar has a moment of inertia of 36.35 cm^4 about the y-y axis and does not require the strengthening bar where posts are installed at a maximum spacing of 1.925 metres. The overall combined displacement of the handrail + post at any point of the barrier from its original unloaded position is limited to a maximum of 25mm under service load conditions.

Handrail (without bar):

Ultimate design imposed load reaction on handrail on longer screens with posts		=	2.26 kN/m	
Ultimate BM on handrail for a post spacing of 1.925m		=	$\frac{2.26 \times (1.925)^2}{8}$	= 1.05 kNm

Properties of the handrail are similar to the handrail with bar (page 5) except as follows:

Moment of inertia about the y-y axis	I_{yy}	=	36.35 cm^4	
Least elastic modulus about the y-y axis	W_{el}	=	8.10 cm^3	
Design ultimate resistance to bending about the y-y axis	M_{Rd}	=	1.149 kNm	> 1.05 kNm OK
Design service load reaction on handrail on longer screens	F	=	$\frac{2.26\ kN/m}{1.5}$	= 1.507 kN/m
Service load deflection of the handrail for a post spacing of 1.925m assuming a worst case of a simply supported span	Δ	=	$\frac{5\ F\ L^4}{384\ E\ I}$	
		=	$\frac{5\ (1507 \times 1.925)\ (1925)^3}{384 \times 70000 \times 36.35 \times (10)^4}$	
		=	10.59mm	

Vertical posts: 60 x 60 x 5mm SHS: properties of section:

Steel grade	=	S355 H to EN 10210-1		
Nominal value of yield strength	=	f_y	=	355 N/mm^2
Nominal value of ultimate tensile strength	=	f_u	=	510 N/mm^2
Inertia of section	=	I_{xx}	=	50.50 cm^4
Elastic modulus of section	=	W_{el}	=	16.80 cm^3
Plastic modulus of section	=	W_{pl}	=	20.90m cm^3
Partial factor for material properties	=	γ_{M1}	=	1.10
Partial factor for class 1 sections	=	γ_{MO}	=	1.00
Modulus of elasticity	=	E	=	210 000 N/mm^2

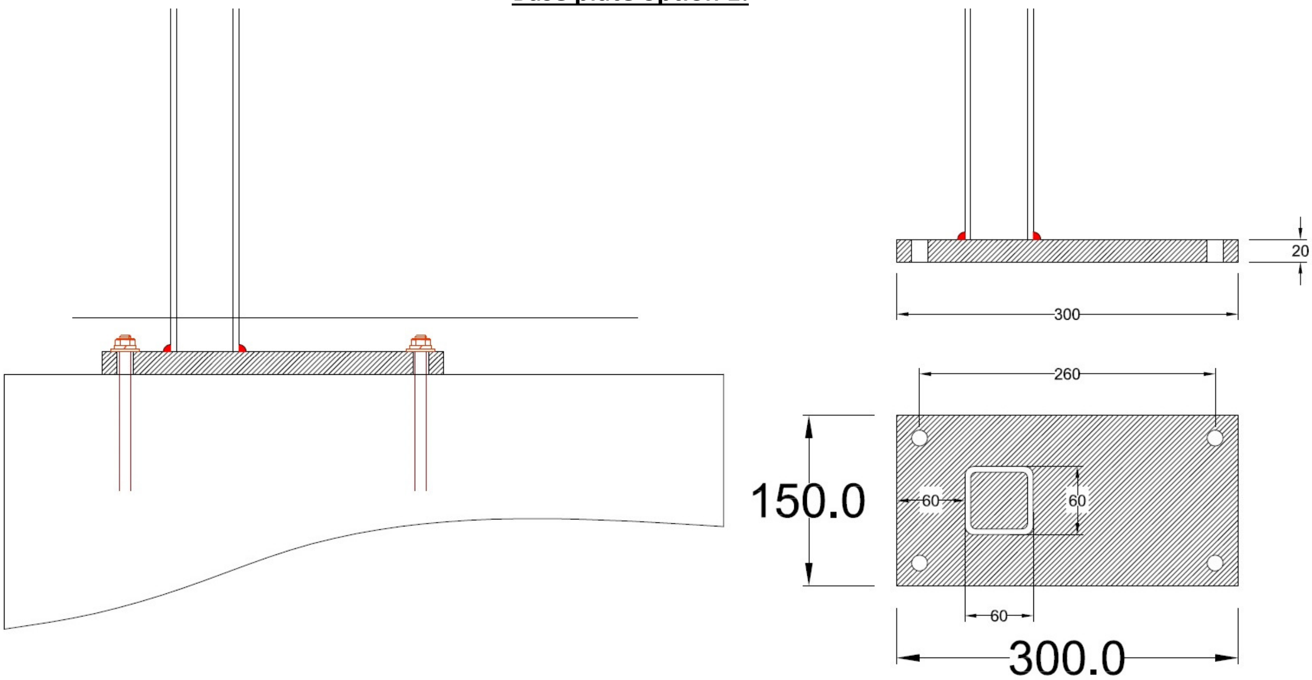
Design ultimate resistance for bending	$M_{pl,Rd}$	=	$\frac{f_y \times W_{pl}}{\gamma_{Mo}}$
		=	$\frac{355\ N/mm^2 \times 20.90\ cm^3 \times (10)^{-3}}{1.0}$
		=	**7.42 kNm**

Balcony 2 pass-through 1.8m privacy screens:

Ultimate moment on posts at 1.925m spacing	M_d	=	2.26 kN/m x 1.125 height x 1.925 c/c
		=	4.894 kNm < 7.42 kNm = OK
Service load deflection of post supporting 1.925m of handrail	Δ	=	$\frac{P\,L^3}{3\,E\,I}$
		=	$\frac{(2260 \times 1.925)\,(1125)^3}{3 \times 210\,000 \times 50.50 \times (10)^4}$ = 19.47mm
Service load deflection of handrail without bar on an end (propped cantilever) span of 1.925 metres	Δ	=	$\frac{W\,L^3}{185\,E\,I}$
		=	$\frac{(1507 \times 1.925)\,(1925)^3}{185 \times 70\,000 \times 36.35 \times (10)^4}$ = 4.40mm
Combined total service load deflection (post + handrail)	Δ	=	19.47 + 4.40 = 23.87mm
		=	< 25mm = OK

SUMMARY: The Balcony 2 pass-through handrail with internal 58 x 4mm steel reinforcing bar is adequate to support the design loading over spans up to 2.75 metres. On longer balustrades where 60 x 60 x 5 SHS posts in steel grade S355 H are installed at a maximum spacing of 1.925 metres the internal reinforcing bar is not required.

Base plate option 1:



Base plate = option 1: 300 x 150 x 20mm

Spacing of posts	=	1.925 m maximum
Design horizontal service load on handrail	=	1.507 kN/m (acts inwards or outwards)

Vertical posts:

Service imposed load moment on posts at 1.925 m c/c	=	1.507 kN/m x 1.925 x 1.125	=	3.26 kNm
Lever arm between the centres of bolts	=	260 mm		
Service load bolt tension on 2 No. bolts	=	$\frac{3.26\ kNm}{2\ No. \times 0.26}$	=	6.27 kN/bolt
Ultimate load bolt tension	=	6.27 x 1.5	=	9.40 kN/bolt

BS 6180:2011, section 6.5, recommends that barrier fixings, attachments and anchorages should be designed to withstand a greater load than the design loading for the barrier generally. This is intended to ensure that under an extreme load condition, barriers show indications of distress by distortion, before there is any possibility of sudden collapse due to failure of the fixings. A 50% increase in the design load on fixings is recommended.

Applying the 50% increase in loads on fixings recommended in BS 6180:2011, the working load bolt tension becomes 9.40 kN/bolt, which should be achievable using M12 drilled resin anchor bolts or similar, or by drilling through and anchoring to the underside of a sound concrete slab.

The nominal tension capacity of M12 (8.8 grade) bolts is 37.80 kN/bolt. Higher bolt forces can therefore be achieved by direct bolting to a suitable steel frame.

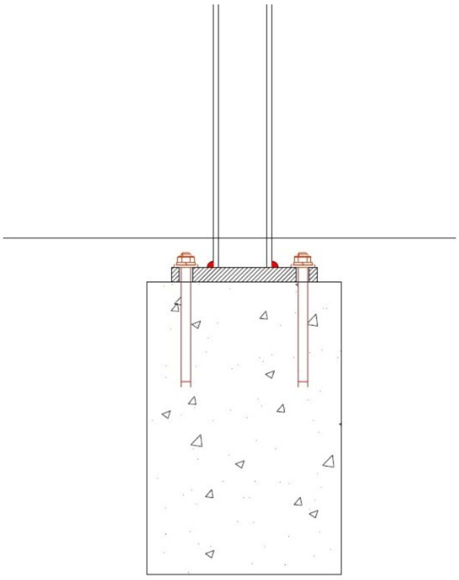
Separate consideration is required where it is proposed to use other types of fixings, or where fixings are to be inserted into weaker materials.

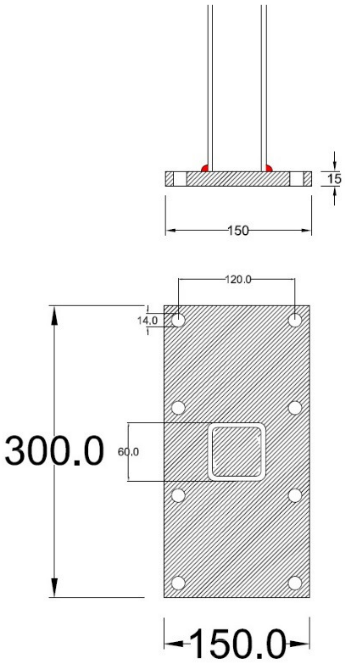
Base plates – option 1: 300mm long x 150mm wide x 20mm thick with 4 M12 bolts:

Ultimate load moment on posts at 1.925m maximum c/c	M_a	=	2.26 kN/m x 1.925 x 1.125	=	4.894 kNm
Plastic modulus of base 150mm wide x 20mm thick	W_{pl}	=	$\frac{150 \times (20)^2}{4}$	=	15000mm^3
Ultimate moment capacity of base in steel grade S275	Mu	=	$\frac{275 \times 15000 \times (10)^{-6}}{1.0}$	=	4.125 kNm
Distance from centre of HD bolts to face of post	d	=	300 – 20 – 60 - 60	=	160mm
Ultimate load bolt tension (not including BS 6180 50% increase)	T	=	9.40 kN		
Ultimate moment on base at face of post	M	=	9.40 kN x 2 No. x 0.16	=	3.0 kNm
(not including BS 6180 50% increase on bolt loads, which only applies to fixings, not other structural elements)		=	< 4.125 kNm	=	OK

Base plates 300 x 150 x 20 in steel grade S275 are adequate.

Base plate – option 2: 300 wide x 150 deep x 15mm thick with 8 M12 bolts.





Tension pull-out loads on fixing bolts:

Ultimate load moment on posts at 1.925m maximum c/c	M_a	=	4.894 kNm		
Distance between bolt centres	d	=	120mm		
Ultimate load bolt tension on 4 No. bolts	T_u	=	$\frac{4.894 \text{ kNm}}{4 \text{ No.} \times 0.12}$	=	10.20 kN/bolt
Working load bolt tension	T_w	=	10.20 / 1.5	=	6.80 kN/bolt

Applying the 50% increase in design loads on fixings recommended in BS 6180: 2011, the working load bolt tension becomes 10.20 kN/bolt, which should be achievable using M12 drilled resin anchor bolts or similar, or by drilling through and anchoring to the underside of a sound concrete slab.

The nominal tension capacity of M12 (8.8 grade) bolts is 37.80 kN. Higher bolt forces can therefore be achieved by direct bolting to a suitable structural steel frame.

Separate consideration is required where it is proposed to use other types of fixings, or where fixings are to be inserted into weaker materials.

Base plates – 300 x 150 x 15mm thick:

Plastic modulus of base 300 wide x 15mm thick	W_{pl}	=	$\frac{300 \times (15)^2}{4}$	=	16875mm^3
Moment capacity of base in steel grade S275	M_c	=	$275 \text{ N/mm}^2 \times 16875 \times (10)^{-6}$		
		=	4.64 kNm		
Distance from centre of bolts to face of SHS post	d	=	60 – 30	=	30mm

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Ultimate load bolt tension (not including the 50% increase as BS 6180)	T_u	=	10.20 kN/bolt		
Ultimate moment on base at face of post (not including BS 6180 50% increase, which applies only to fixings, not other structural elements)	Mu	=	10.20 kN x 4 No. x 0.03	=	1.224 kNm
		=	< 4.64 kNm	=	OK

Base plates 300 wide x 150 deep x 15mm thick in steel grade S275 with 8 M12 bolts are adequate.

Welded connection between post & base plate:

The 60 x 60 x 5mm SHS post is welded to the top of the base by means of a full strength butt and/or fillet weld.

Elastic section modulus of post W_{el} = 16.80 cm^3

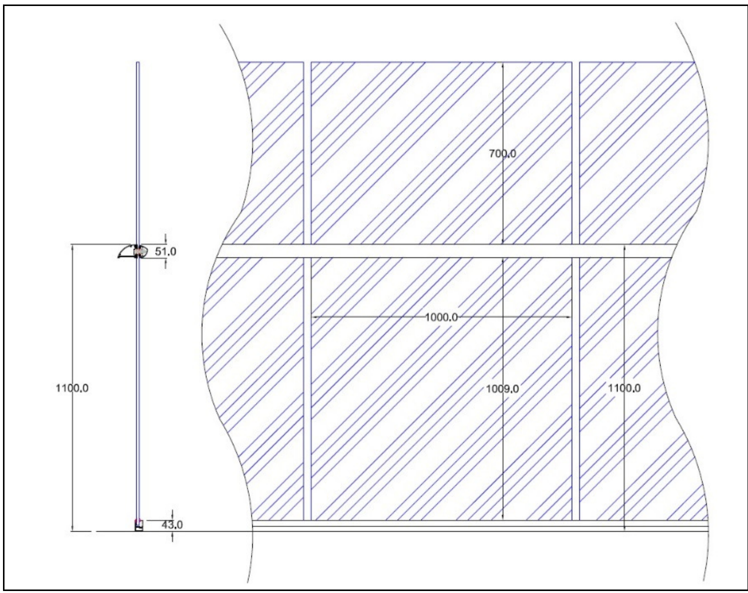
Maximum ultimate elastic bending stress on post $\frac{M_a}{W_{el}} = \frac{4.894 \times (10)^6}{16.80 \times (10)^3} = 291\ N/mm^2$

= 1.455 kN/mm on 5mm thick section

Transverse capacity of 8mm fillet weld = 1.54 kN/mm = OK

A continuous 8mm fillet weld around the perimeter of the post, or a full strength butt weld, are adequate.

Glass infill:



Glass panels can be any length.

For the purposes of design and checking a nominal glass panel width of 1000mm simply supported between the bottom rail and the handrail has been used.

Separate design loading conditions on the infill are considered:

Ultimate design UDL on the infill of 1.50 kN/m²

Ultimate moment on glass due to UDL on span of 1.0m Mu = $\frac{1.50\ kN/m^2 \times (1.0)^2}{8}$ = 0.1875 kNm/m

= < 1.456 kNm = OK

Point load on the infill of 0.5 kN

Point load on the glass	=	0.5 kN point load applied in any position		
Worst case for bending stress on the glass due to point load	=	point load applied at mid-height of glass		

Point load of 0.5 kN on glass infill:

Ultimate moment on glass due to point load = $\frac{0.5 \text{ kN} \times 1.5 \times 1.0\text{m}}{4}$ = 0.1875kNm

Conservatively, it is assumed that this bending moment is carried by a 300mm wide vertical strip of glass.

Moment capacity of 300mm	=	1.456 kNm x 0.3	=	0.437 kNm
strip of glass	=	> 0.1875kNm	=	OK

The glass has adequate strength to support the ultimate design imposed loads and also the design wind loading.

Glass deflection:

Consider service load deflection of the glass due to the design wind UDL:

Inertia of glass 10mm thick x 1000mm long = $\frac{1000 \times (10)^3}{12}$ = 83333 mm^4

Service load deflection due to a UDL of 1.0 kN/m^2 on a simply supported span of 1.0m (floor to handrail)

$$= \frac{5 w L^4}{384 E I}$$

$$= \frac{5 \times (1000 \times 1.0)\ (1000)^3}{384 \times 70\,000 \times 83333}$$

$$= 2.23 \text{ mm} < \frac{span}{65} = OK$$

Conservatively, for deflection calculation purposes consider that the design point load is carried by a 300mm wide vertical strip of glass:

Inertia of glass 10mm thick x 300mm long = 0.3 x 83333 mm^4 = 25 000 mm^4

Service load deflection due to a point load of 0.5 kN applied at mid-span

$$= \frac{P L^3}{48 E I}$$

$$= \frac{500 \times (1000)^3}{48 \times 70\,000 \times 25\,000}$$

$$= 5.95\text{mm} < \frac{span}{65} = OK$$

SUMMARY: The glass is adequate in terms of both bending strength and deflection.

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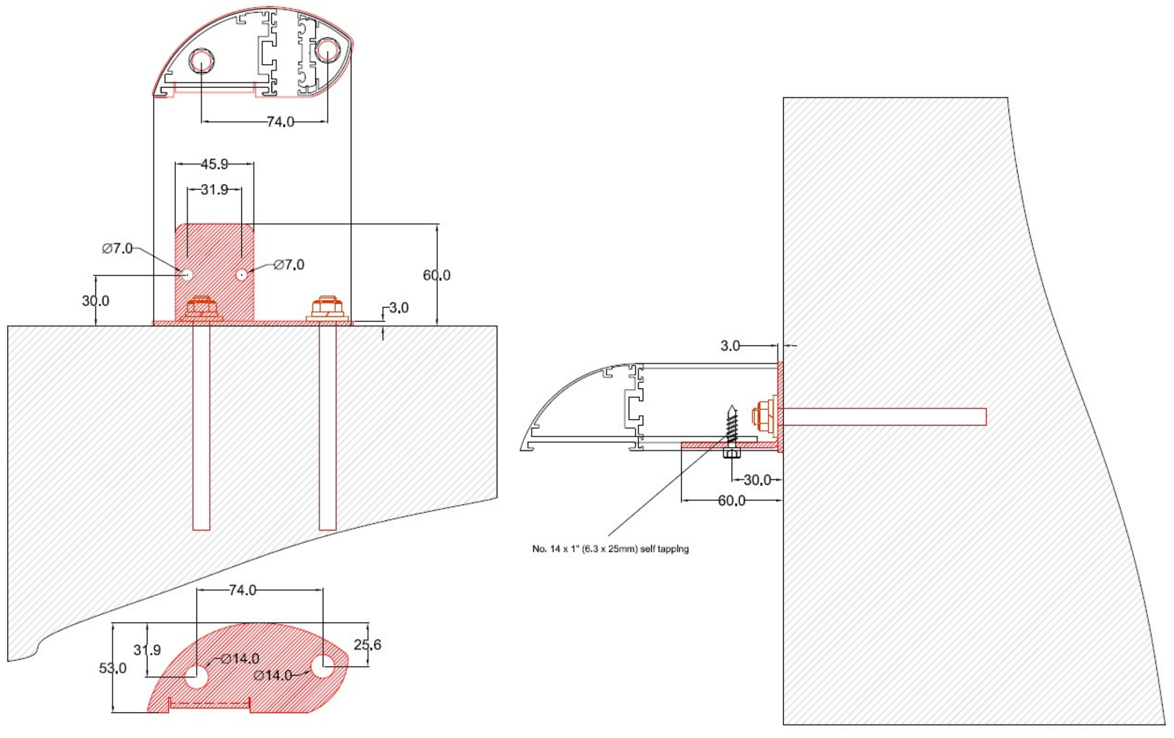
Wall fixings:



The handrail wall fixing consists of a 3mm thick stainless steel angle bolted to the wall with 2 No. M12 stainless steel drilled resin anchor bolts or similar and secured to the handrail with 2 No. 6.3mm diameter stainless steel Phillips self-tapping screws.

The max. simply supported span of the handrail with internal reinforcing bar between points of support is 2.75m.

Service load on the wall fixing for a span of 2.75m	=	1.507 kN/m x 1.375m	=	2.07 kN/fixing

This load is transferred to the angle bracket by means of 2 No. 6.3mm diameter stainless steel Phillips self-tapping screws.

Ultimate shear force on self-tapping screws	=	$\frac{2.07 \times 1.5}{2}$	=	1.55 kN/screw

Applying the 50% increase in loads on fixings recommended in BS 6180:2011, this becomes **2.33 kN/screw.**

The ultimate shear loads on self-tapping screws are taken from the table in Lindab's technical literature.

Thickness of aluminium in the handrail at screw positions	=	2.5mm
Thickness of stainless steel brackets (Nom t mm)	=	3.0 mm

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Wall fixing:

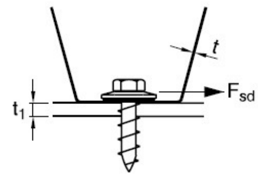
Ultimate shear capacity of 6.3mm diameter screws, safety class 1 = 6.76 kN/screw

For safety classes 2 and 3 this value is divided by 1.1 and 1.2 respectively. Safety class 3 is the highest safety class and has been assumed to apply to balustrades/barriers. The shear capacities given in Lindab's table are based upon material having a tensile yield limit of 350 N/mm^2. The values given in the table have been adjusted to allow for the yield stress of stainless steel type 304 (290 N/mm^2).

The ultimate shear capacity of 6.76 kN/screw has therefore been reduced by 290/350 and divided by 1.2 to represent safety class 3 and 290 N/mm^2 yield stress rather than 350 N/mm^2. The adjusted ultimate shear capacity is then 4.67 kN/screw, compared with the design value of 2.33 kN/screw, and is therefore adequate.

Shearing force, construction screws

Dimensioning value F_{sd} kN/screw. Attention is paid both to failure of the edge of the hole and shearing failure in the screw. Safety class 1.



Nom t mm	When calculating t mm	Tensile yield limit N/mm²	Screw diameter 4.2 mm		Screw diameter 4.8 mm		Screw diameter 5.5 mm		Screw diameter 6.3 mm	
			t_1 =t	t_1 = 2.5 t	t1 =t	t_1 = 2.5 t	t_1 =t	t_1 = 2.5 t	t_1 =t	t_1 = 2.5 t
0.4	0.32	250	0.26	0.54	0.28	0.61	0.30	0.70	0.32	0.81
0.5	0.41	250	0.38	0.69	0.40	0.79	0.43	0.90	0.46	1.03
0.6	0.52	250	0.52	0.86	0.56	0.98	0.60	1.12	0.64	1.29
0.7	0.60	350	0.93	1.41	1.00	1.61	1.07	1.85	1.14	2.12
0.8	0.73	350	1.25	1.72	1.34	1.96	1.43	2.25	1.53	2,58
1.0	0.93	250	1.29	1.56	1.38	1.79	1.47	2.05	1.58	2,34
1,0	0,93	350	1,80	2,19	1,93	2,50	2,06	2,86	2,21	3,28
1,2	1,13	350	2,41	2,66	2,58	3,04	2,76	3,48	2,95	3,99
1,5	1,42	250	2,39	2,39	2,60	2,73	2,78	3,12	2,97	3,58
1,5	1,42	350	3,03*	3,03*	3,63	3,82 3,64	3,89	4,37	4,16	5,01
2.0	1.91	350	3.03*	3.03*	4.16 3.64	4.16 3.64	5.72 5.20	5.72 5.20	6.49	6.74
2.5	2.40	350	3.03*	3.03*	4.16 3.64	4.16 3.64	5.72 5.20	5.72 5.20	7.80 6.76	7.80 6.76

In the area of number pairs in the table and marked *, shearing failure in the screw is decisive.
The value to the left in each number pair relates to carbon steel screws, while the number to the right relates to stainless steel screws.

Shear force on wall fixing bolts

Working load shear force on the 2 No. fixing bolts based upon a handrail span of 2.75m	=	$\frac{2.07\ kN}{2\ No.}$	=	1.035 kN/bolt

Applying the 50% increase on fixing loads recommended in BS 6180:2011, this becomes **1.55 kN/bolt.**

This shear load should be within the working load capacity of M12 drilled resin anchor bolts or similar into good quality concrete or brickwork. Separate consideration is required when drilling into weaker materials or when using other less robust types of fixings.

Pull-out forces on wall fixings

The horizontal load on the handrail is applied to the fixing angles through the Phillips stainless steel self-tapping screws, located 30mm from the back of the angles. The wall fixing bolts are 74mm apart horizontally.

Working load pull-out force on the 2 No. anchor bolts on a span of 2.75m	=	$2.07\ kN \times \frac{30.0}{74}$	=	0.84 kN/bolt
Applying the 50% increase in load on fixings recommended in BS 6180:2011, this			=	**1.26 kN/bolt**

Wall fixing brackets:

Material type	=	stainless steel grade 304
Characteristic ultimate tensile strength	=	621 N/mm^2
Characteristic 0.2% proof stress	=	290 N/mm^2

The horizontal part of the bracket measures 45.9mm wide x 3mm thick.

Plastic modulus of 45.9 x 3mm section for horizontal loads	=	$\frac{3 \times (45.9)^2}{4}$	=	$1580mm^3$
Resistance moment of section	=	$290\ N/mm^2 \times 1580mm^3 \times (10)^{-6}$		
for horizontal loads	=	0.458 kNm		
For a simply supported span of 2.75m: ultimate load on wall bracket	=	2.26 kN/m x 1.375	=	3.11 kN
Ultimate horizontal moment	=	(3.11 kN) (0.03)	=	0.093 kNm
applied to the bracket	=	< 0.458 kNm	=	OK
Shear capacity of section 45.9mm wide x 3mm thick	=	$\frac{A_v (f_y/\sqrt{3})}{\gamma_{M0}}$		
	=	$\frac{(45.9 \times 3)(290/1.732)}{1.1}$	=	20960 N
			=	OK

The wall fixing brackets are adequate.

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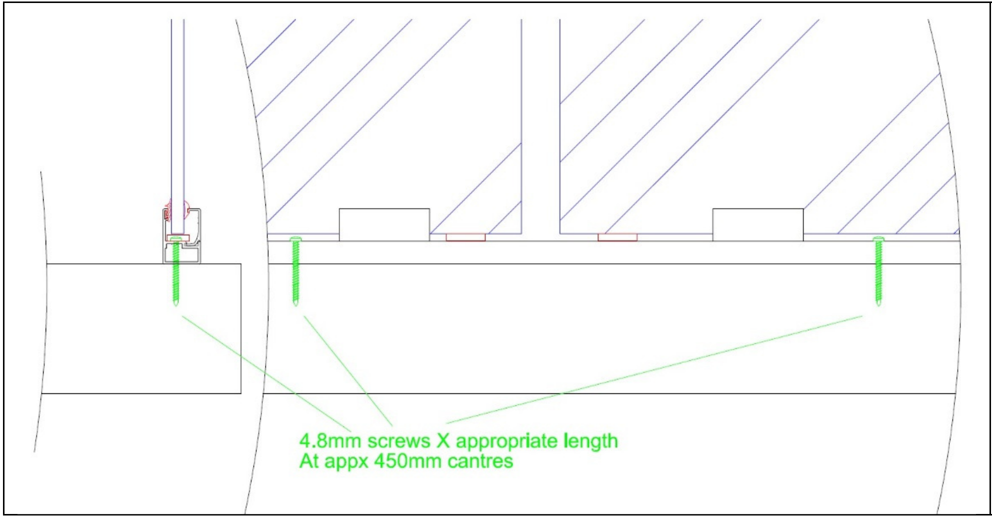
Bottom rail fixing:



The standard bottom rail fixing consists of 4.8mm diameter screws at 450mm centres.

The worst case for design loading on the fixings is when the design service point load of 0.50 kN on the glass acts at the position of a fixing. The fixing screw is then subjected to a working load shear force of 0.50 kN/screw.

The allowable load on the fixing screws varies depending upon the type and thickness of the material into which the screws are inserted.

As an example, fixing to a balcony deck comprising 15mm thick plywood strength class C16, group 1, the basic allowable working load single shear value given in BS 5268 : Part 2 : 1996 for a No. 10 (4.8mm) screw 45mm long is 0.519 kN.

Where a pre-drilled steel component of adequate strength is screwed to a timber member, the basic lateral load of 0.519 kN is multiplied by a modification factor of 1.25, making an allowable shear value of 0.648 kN, which is adequate in relation to the design working shear load shear force of 0.50 kN.

Other values of allowable shear loads on fixings will apply where the deck material is of different strength and/or thickness.

The installers should ensure that where the deck material has a lower strength than 15mm thick plywood, class C16, group 1, the spacing of the fixing screws is reduced accordingly.

SUMMARY

BALCONY 2 pass-through 1.8m privacy screen system: Handrail with 58 x 4mm steel internal reinforcing bar, or without bar using 60 x 60 x 5mm SHS posts at 1.925m maximum spacing, with two base plate options:

1) For sites within the parameters listed on page 7 of these calculations, and/or have a characteristic wind pressure that does not exceed **1.00 kN/m^2**, wind loading and the design imposed loading are the same. Sites that do not come within these parameters require separate consideration.

2) On single span and corner balconies, the system is capable of supporting the ultimate design loads over spans up to **2.75 metres** between points of support. (i.e. a handrail wall fixing, or a handrail corner joint.)

3) On longer balconies where the length of the balustrade exceeds 2.75 metres, vertical posts are installed at a maximum spacing of **1.925m** between post centres. The posts are made from **60 x 60 x 5mm** square hollow steel sections (SHS) in steel grade **S 355 H** with aluminium sleeves.

4) The handrail profile without internal reinforcing bar has a moment of inertia of 36.35 cm^4 about the y–y axis and the internal reinforcing bar is not required where posts are installed at a maximum spacing of 1.925m between post centres.

5) The SHS posts are welded (full strength butt or 8mm fillet welds) to steel base plates. Two options for base plates are considered. Option 1 is 300mm deep x 150mm wide x 20mm thick with 4 No. M12 (8.8 grade) HD bolts. The design working pull-out force on the HD bolts is **9.40 kN/bolt.** Option 2 is 150mm deep x 300mm wide x 15mm thick with 8 M12 (8.8 grade) HD bolts. The design working pull-out load on the holding down bolts is **10.20 kN/bolt.** These values include the 50% increase on calculated loads in accordance with BS 6180:2011.

6) These loads should be achievable using M12 drilled resin anchor bolts or similar into good quality concrete, or by drilling through and anchoring to the underside of a sound concrete slab. However, the installers should satisfy themselves that the fixing bolts chosen are suitable to resist the specified loads, and also that the structure into which they are installed can support these loads.

7) Higher bolt loads could be achieved where fixings are made direct to a substantial structural steel frame. Lower bolt forces would most likely need to be taken where fixings are made into materials having a lower strength than good quality concrete.

8) For the maximum span of **2.75 metres** on single span and corner balconies, the horizontal working load shear force on the wall fixing bolts is **1.55 kN/bolt**, and the working load pull-out force is **1.26 kN/bolt**. Two 14mm diameter holes are provided in wall fixing brackets for M12 drilled anchor bolts or similar. The design loads should be achievable where bolts are installed into good quality concrete or brickwork.

SUMMARY (continued)

9) The 6.3mm diameter self-tapping stainless steel screws connecting the handrail to the stainless steel angle brackets at wall and post fixings are adequate to support the specified design loads. The 3mm thick stainless steel brackets are also adequate to support the design wall fixing loads.

10) The standard bottom rail fixing comprises 4.8mm diameter screws inserted into the balcony deck at 450mm centres. At this spacing the fixings are required to have a working load shear capacity of 0.50 kN/screw. The installers should satisfy themselves that the screws chosen are suitable to resist this load when inserted into the particular deck material present on a specific project. Where the deck material is of reduced strength and/or thickness the spacing of the screws should be reduced accordingly.

11) The 10mm thick thermally toughened safety glass is adequate to support the specified design loads.

Prepared for and on behalf of Balconette by
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