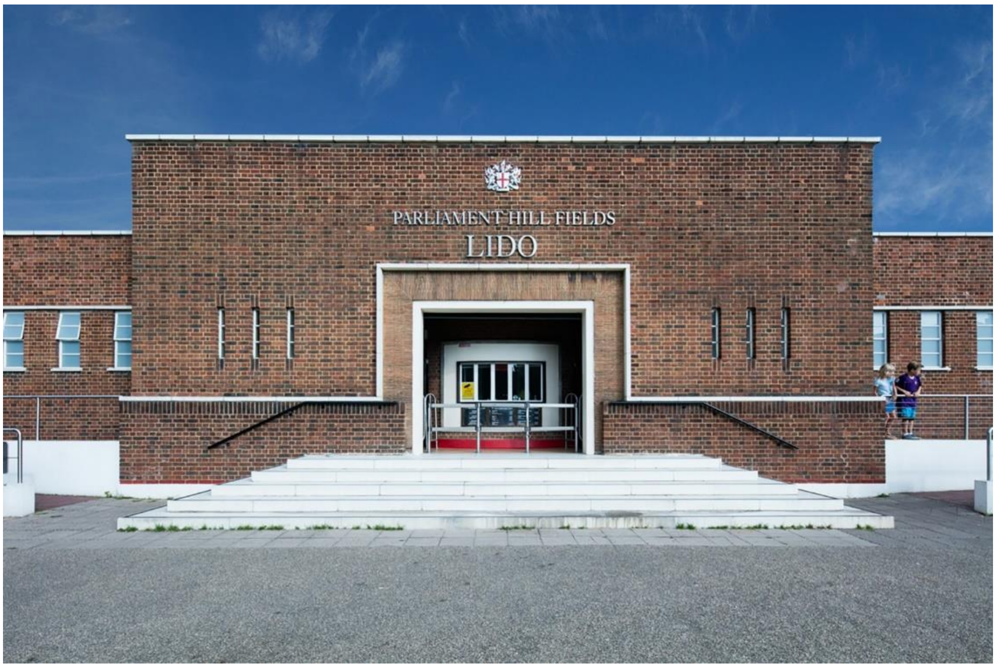
STRUCTRAL ASSESSMENT
Of
PARLIAMENT HILL LIDO
FOR NEW PV PANELS ON THE
MALE CHANGING ROOM ROOF
for
SYKES & SON LIMITED



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Chamberlain consulting LLP
consulting structural and civil engineers

1 Bromley Lane
Chislehurst
Kent
BR7 6LH

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1.0 INSTRUCTIONS & SCOPE

Chamberlain Consulting were instructed by Dave Cossington of Sykes & Son Limited on the Wednesday 29th of November 2023 to assess the existing roof structure to confirm its adequacy for supporting new PV panels. A site visit was undertaken on the 29th of November 2023 by Mark Robinson of Chamberlain Consulting to review the existing structure.

2.0 EXISTING STRUCTURE

The existing structure is constructed from masonry with a lightweight steel/aluminium decking roof spanning between existing steel beams. Existing drawings have been provided and are referenced in this report.

The structure dates to 1938 and appears, although the steelwork is a later addition due to the size of the members. The structure is in a reasonable state of repair, For the purpose of this report the steel beams are assumed to be S275.

The drawings show the modifications undertaken around 1995 that resulted in increasing the roof pitch by adding additional steelwork over the main beams.

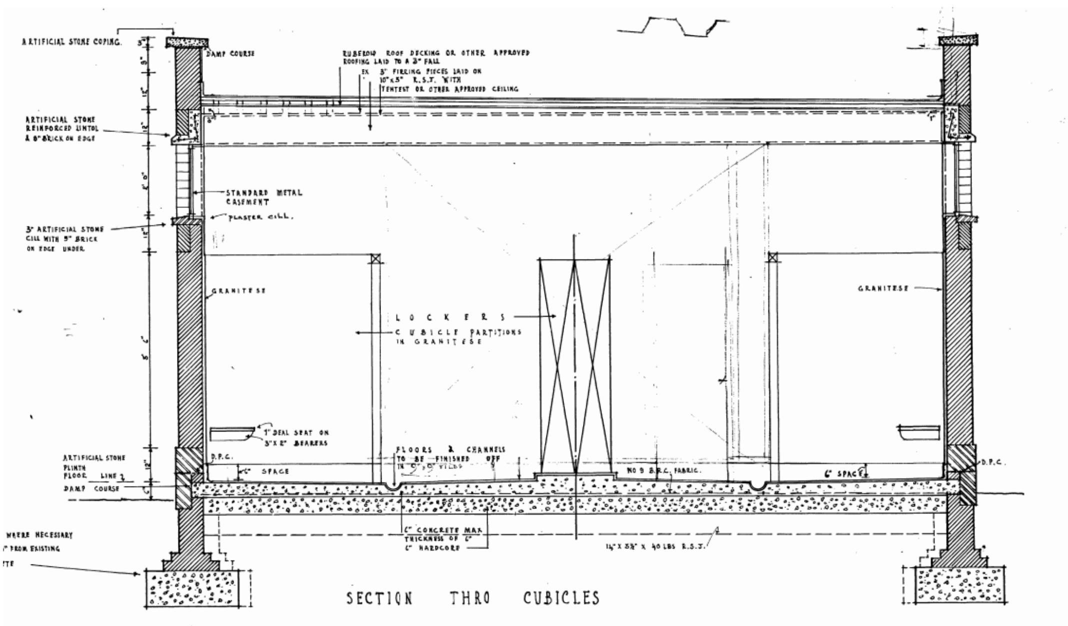


Figure 2.0 – Section Through Original Structure (From Drawing No. 10)

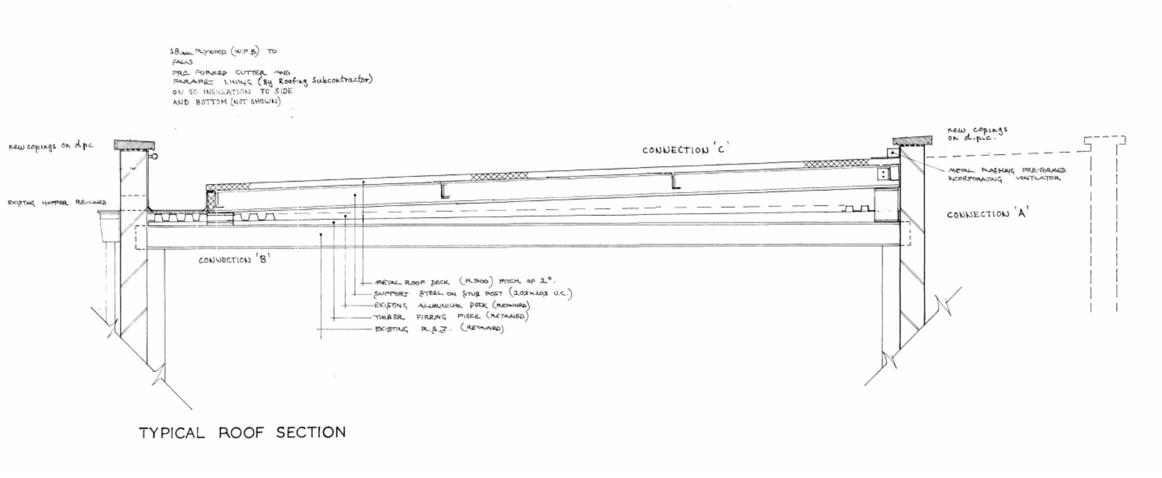


Figure 2.1 – Revised detail from 1995 (Drawing No. C177 R021 96 03)

The modified structure cannot be seen from below, other than the brackets referenced as connection A and B.

Figure 2.2 – Photograph showing the internal layout of the beams

3.0 STRUCTURAL LOAD ASSESSMENT

3.1 PV Panels:

The proposed PV Panels are to be in two sections resting on 6 Redtip swift rails over the span of the roof. The imposed loading on the roof from the rails will and panels is noted as being 0.4 kN/m^2.

The following plan has been provided by the client showing the extent of the additional PV and the loading on the existing roof, the PV is avoiding the roof light at the end.

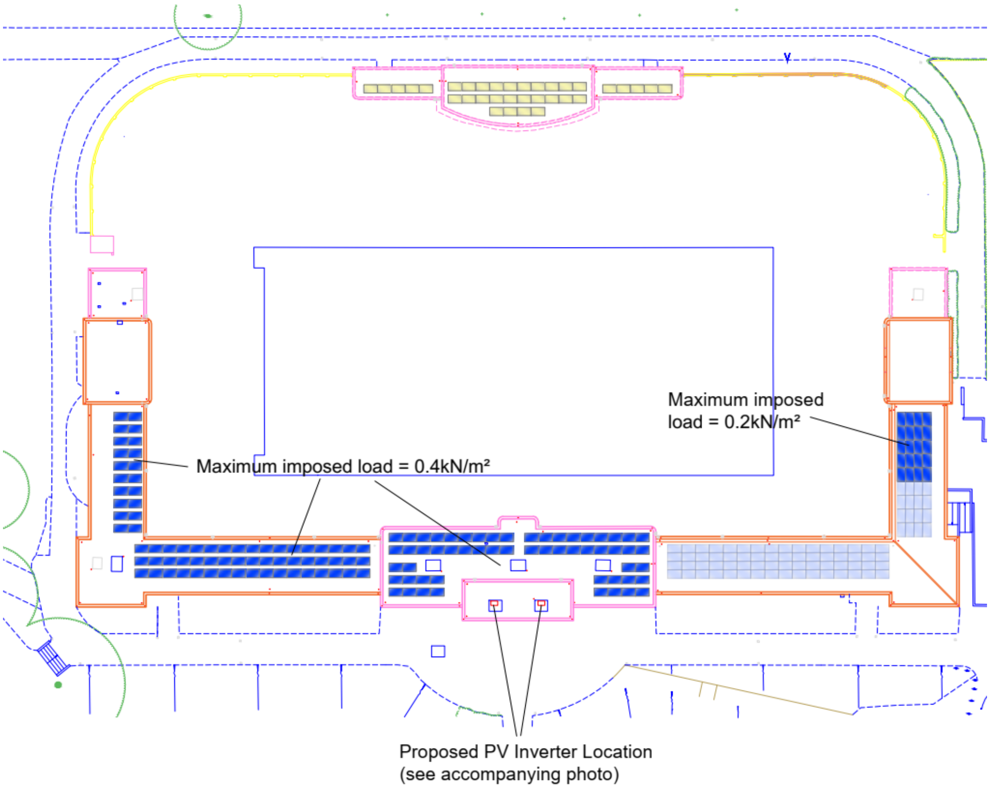


Figure 3.1 – Proposed area of PV.

3.2 Existing Roof Loading:

Imposed loading, or 'live loading', covers items such as allowances for maintenance and snow loading. The minimum level of imposed loading for roofs with 'access for maintenance only' would be 0.75 kN/m^2

As we have no information on the original design loading requirements for the roof as a whole, we believe it would have been designed in accordance with the following loads;

Imposed loading (Roof with access for maintenance only)	=	0.75 kN/m^2
Top level profiled steel sheeting	=	0.15 kN/m^2
Low level profiled aluminium sheeting	=	0.10 kN/m^2
Top level sheeting rails	=	0.10 kN/m^2
Total Loading	**=**	**1.10 kN/m^2**
Additional PV Panel Loading	=	0.40 kN/m^2
Increased Total Loading	**=**	**1.50 kN/m^2**

Consider a typical wind pressure on the roof of 0.3 kN/m^2 and a wind suction of 0.9 kN/m^2

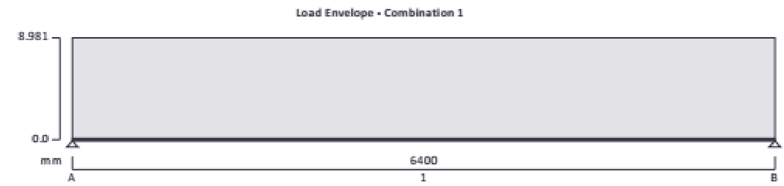
3.3 Steel Beam Design Check

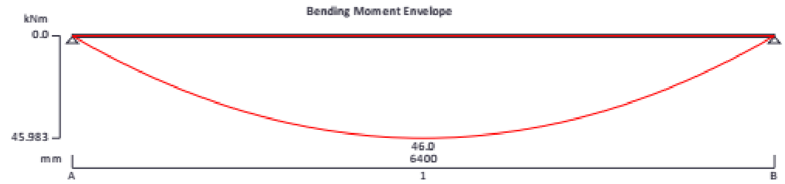
The existing beams are 254mm deep x 125mm wide with a 10mm flange, the closest member is an IPE 240 O which has been used in the following design checks.

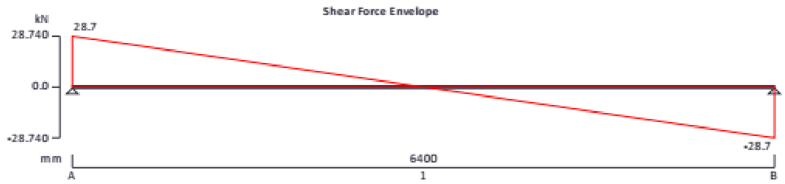
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Exg Dead - Dead full UDL 2.2 kN/m
	PV Loads - Dead full UDL 1.25 kN/m
	Imposed - Imposed full UDL 2.3 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **46** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **28.7** kN	V_{min} = **-28.7** kN
Deflection	δ_{max} = **5.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **28.7** kN	R_{A_min} = **28.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **12.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **7.4** kN	
Maximum reaction at support B	R_{B_max} = **28.7** kN	R_{B_min} = **28.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **12.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **7.4** kN	

Section details

Section type — **IPE 240 O (Arcelor)**

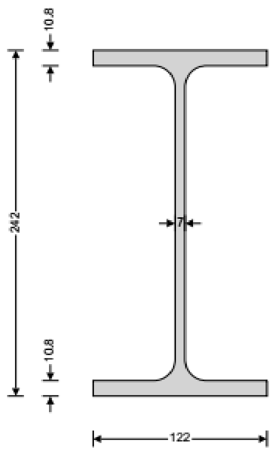
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **10.8** mm

Design strength — p_y = **275** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section — d = **190.4** mm

$d / t = 27.2 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = \mathbf{61}$ mm

$b / T = 5.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min})) = \mathbf{28.7}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D = \mathbf{1694}$ mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v = \mathbf{279.5}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) = \mathbf{46}$ kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{112.8}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{17.778}$ mm

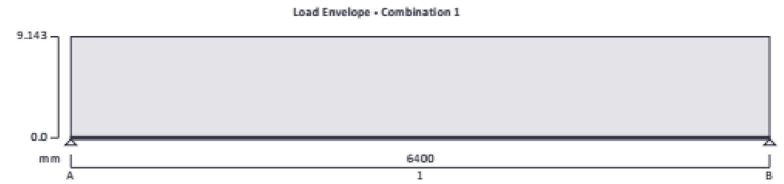
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{5.61}$ mm

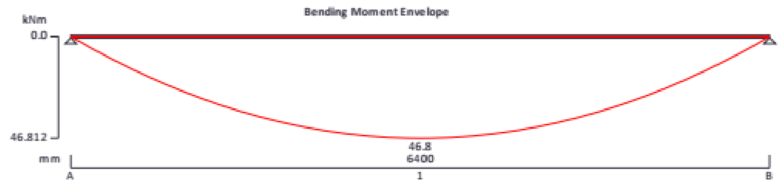
PASS - Maximum deflection does not exceed deflection limit

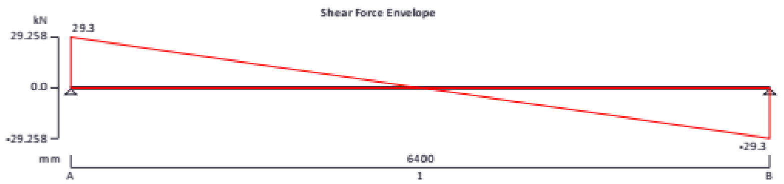
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Exg Dead - Dead full UDL 2.2 kN/m
	PV Loads - Dead full UDL 1.25 kN/m
	Imposed - Imposed full UDL 2.3 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

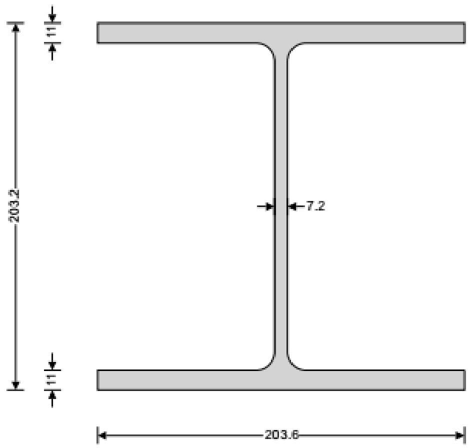
Maximum moment	M_{max} = **46.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **29.3** kN	V_{min} = **-29.3** kN
Deflection	δ_{max} = **5.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **29.3** kN	R_{A_min} = **29.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **12.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **7.4** kN	
Maximum reaction at support B	R_{B_max} = **29.3** kN	R_{B_min} = **29.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **12.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **7.4** kN	

Section details

Section type	**UKC 203x203x46 (Tata Steel Advance)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **11.0** mm
Design strength	p_y = **275** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_x = **1.00**
Effective length factor in minor axis	K_y = **1.00**
Effective length factor for lateral-torsional buckling	$K_{LT.A}$ = **1.00**
	$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section	d = **160.8** mm	
	$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 =$ **101.8** mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$ Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min})) =$ **29.3** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D =$ **1463** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v =$ **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **46.8** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **136.8** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **17.778** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **5.366** mm

PASS - Maximum deflection does not exceed deflection limit

4.0 CONCLUSIONS

In summary of our assessment the existing roof can support the proposed PV Panels based on a maximum loading of $0.2kN/m^2$ for the PV panels or $0.4kN/m^2$ for the PV panels and support system as indicated in figure 3.1, using a continuous rail mounting system, the rails should span from beam to beam.

No PV panels should be installed above roof lights or penetrations in the existing roof, the inverters have no weight provided, they should be less than or equal to $0.4kN/m^2$ and supported in the same way as the PV panels.

The roof should be subject to no additional loads after the proposed PV Panels are installed.

5.0 LIMITATIONS

This structural assessment report has been prepared on the basis of a desktop study only of the information provided as noted in the scope, and is not intended to be exhaustive, but to give a general overview of the roof loading capacity in the specific stated areas. A full structural investigative survey of the building or associated elements was not carried out and therefore, Chamberlain Consulting LLP can accept no liability in respect of defects or issues outside the scope of our appointment.

Inspection and Assessment Report prepared by

Mark Robinson

Mark Robinson

For Chamberlain Consulting LLP

Mark Robinson Meng CEng MIStructE MIMechE