STRUCTURAL DESIGN CALCULATIONS

Project South Kensington Station Stabilisation
Permanent Works to Upper Roof

Project No. 3095 – 003 – RWC - CAL – 0001 – Rev A

Sections Design of Replacement Upper Roof
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<th>Date</th>
<th>Prepared By</th>
<th>Checked By</th>
<th>Approved By</th>
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<td>Mar 14</td>
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**Structural Calculations - Code and Standard Used**

- BS EN 1990 : Basic of Structural Design
- BS EN 1991 : Actions on Structure
- BS EN 1992 : Designs of Concrete Structure
- BS EN1993 : Design of Steel Structure
- BS EN 1994 : Design of Composite Steel and Concrete Structure
- S1050 A7 : Civil Engineering – Common Requirements
- BS5950 : Structural use of Steelwork in Building (for Existing Steel Beam Assessment)
- BS 5628 : Code of Practice for the use of Masonry (for Existing Masonry Assessment)
- S1053 A10 : Civil Engineering – Building and Station Structures
- S1061 A3 : Civil Engineering – Bridges and Structures Assessment Standard
DESIGN OF REPLACEMENT UPPER ROOF

Introduction

As part of the works at South Kensington Station Asset Stabilization Program there is a requirement for a design of replacement upper roof. Refer to page R - 2 for the location of the roof area to be designed. Refer to document KNUK-DES-CDS-189-0003 Rev 04 for Conceptual Design Statement.

Refer to Robert West Drawings no. 3095/003/1010, 1011 and 5010 for proposed replacement roof details

Actions

Permanent Load
- Roof finishes 0.30 kN/m²
- Permanent Formwork F100 Gauge 1.2 0.19 kN/m²
- 350mm thick concrete slab (584+0.05x25) 7.09 kN/m²
- Total permanent load 7.58 kN/m²

Services
- General services 1.00 kN/m²

Variable Load
- Imposed load on roof (UDL) 1.50 kN/m²
- Imposed load on roof (PL) 0.90 kN
- Construction Load 0.75 kN/m²

Snow
- Snow loading in accordance with BS EN 1991–1–3

Wind Loading
- Wind loading in accordance with BS EN 1991-1-4

Max suction load = 61.03 kN

Permanent load
= 7.58 x 18.645 L x 6.725 W x 0.8
= 760 kN

OK
Historic Plan of Roof

Existing Roof Plan Showing Proposed Steel Arrangement and Sizes

Proposed Roof Plan Showing Proposed Steel Arrangement and Sizes

Proposed 203 SFB 100 beam (Typical)
Analysis for a simply supported secondary composite beam under a uniformly distributed loading.

The following distributed loads are applied to the beam.

- self-weight of the beam
- concrete slab
- imposed load

The beam is a 203SFB100 profile in bending about the strong axis. This analysis includes:

- the classification of the cross-section,
- the calculation of bending resistance,
- the calculation of shear resistance,
- the calculation of longitudinal shear resistance of the slab,
- the calculation of deflection at serviceability limit state.

This calculation does not include any shear buckling verification of the web.

Partial factors

- $\gamma_G = 1.35$ (permanent loads)
- $\gamma_Q = 1.50$ (variable loads)
- $\gamma_{M0} = 1.0$
- $\gamma_{M1} = 1.0$
- $\gamma_V = 1.25$
- $\gamma_C = 1.5$

EN 1990

EN 1993-1-1 § 6.1 (1)

EN 1994-1-1 § 6.6.3.1

EN 1992-1-1
Basic data

Design a composite roof beam of a South Kensington station building according to the data given below. The beam is assumed to be no propping required during construction.

The profiled steel sheeting is transverse to the beam.

- Span length : 6.60 m (max)
- Bay width : 2.30 m (max)
- Slab depth : ~ 350mm include metal roof profile
- Imposed load : 1.5 kN/m² or Snow Drift whichever is greater
- Services load : 1.00 kN/m²
- Reinforced Concrete density : 25 kN/m³
- Steel grade : S355

Try Slim Floor Beam 203 x 203 x 100 SFB

<table>
<thead>
<tr>
<th>Mass</th>
<th>100 kg/m</th>
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<tr>
<td>Section area</td>
<td>A = 188.4 cm²</td>
</tr>
<tr>
<td>Second moment of area /yy</td>
<td>Iy = 17457 cm⁴</td>
</tr>
<tr>
<td>Elastic modulus /yy</td>
<td>Wel,y = 1133 cm³</td>
</tr>
<tr>
<td>Modulus of elasticity of steel</td>
<td>Ea = 210000 N/mm²</td>
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BS4 Corus Advance
Profiled steel sheeting

Maximum Span (m) Single or Double span

<table>
<thead>
<tr>
<th>Profile</th>
<th>Steel Thickness (mm)</th>
<th>Profile weight (kN/m²)</th>
<th>Concrete Slab Depth above profile</th>
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<tbody>
<tr>
<td></td>
<td>0mm</td>
<td>100mm</td>
<td>150mm</td>
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Concrete Usage Table

<table>
<thead>
<tr>
<th>Profile</th>
<th>Weight of Concrete (kN/m²)</th>
<th>Slab Depth above profile (mm)</th>
<th>&quot;ED&quot; (mm)</th>
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<tbody>
<tr>
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<td>3.11</td>
<td>4.33</td>
<td>5.55</td>
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<tr>
<td>F100</td>
<td>3.40</td>
<td>4.62</td>
<td>5.84</td>
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To determine concrete usage increase slab depth above profile by "ED" mm.
## Concrete parameters: C 40/50

- **Value of the compressive strength** \( f_{ck} = 40 \text{ N/mm}^2 \)
- **Value of the compressive strength at 28 days** \( f_{cm} = 48 \text{ N/mm}^2 \)
- **Value of the tensile strength at 28 days** \( f_{ctm} = 3.5 \text{ N/mm}^2 \)
- **Secant modulus of elasticity of concrete** \( E_{cm} = 35 \text{ kN/mm}^2 \)

To take into account the troughs of the profiled steel sheeting, the weight of the slab is taken as: 7.58 x 2.3 = 17.434 kN/m

Self weight of the beam: \((100\times 9.81)\times 10^{-3} = 0.98 \text{ kN/m}\)
### Design of Profile Sheeting

#### Permanent Stage

Permanent load: \( G = 7.58 + 1.0 \text{ kN/m}^2 \)

Variable Action:

Imposed load: \( Q = 1.5 \text{ kN/m}^2 \)

Snow drift - average UDL:

\[
Q = (1.35 \times 3.5 \times 0.5 \times 2/6.6) = 0.72 \text{ kN/m}
\]

ULS Combination:

\[
\gamma_G G + \gamma_Q Q = 1.35 \times (7.58 + 1.0) + 1.50 \times 1.5 = 13.833 \text{ kN/m}
\]

Maximum shear force at supports:

\[
V_{z,Ed} = 0.5 \times 13.833 \times 2.30 = 15.91 \text{ kN}
\]

#### Construction Stage

Permanent load:

\( G = 7.58 \text{ kN/m}^2 \)

Variable load:

Imposed load: \( Q = 0.75 \text{ kN/m}^2 \)

ULS Combination:

\[
\gamma_G G + \gamma_Q Q = 1.35 \times 7.58 + 1.50 \times 0.75 = 11.36 \text{ kN/m}
\]

Maximum shear force at supports:

\[
V_{z,Ed} = 0.5 \times 11.36 \times 2.3 = 13.06 \text{ kN}
\]
➢ The Formwork100 gauge 1.2 profiled steel sheeting will be placed over the upper surface of the bottom flange of the beams, spanning between them
➢ Hilti shot fire nails will be installed to restrain the deck

From Hilti Technical Data

Hilti Short Fired Nails X-UP8 capacity

\[ \begin{align*}
N_{rec} &= 1.2 \text{kN} \\
V_{rec} &= 2.6 \text{kN}
\end{align*} \]

Maximum shear force at support \( = 15.91 \text{kN over 1.0m width} \)

Number of Hilti Short Fired Nails \( = 15.91/2.6 \approx 6.2 \) say 7 required

Provide 2No. Hilti Short Fired Nails at every rib (fixed to bottom flange of steel beam).

\( (2 \times 1000/295) + 2 = 8.7 \approx 8 \) no.

OK
Verification of Bending Resistance (Steel Beam Design)

Permanent Stage

Permanent load: \( G = 1.00 + (7.58 + 1.0) \times 2.3 = 20.80 \text{ kN/m} \)

Variable load:

Imposed load \( Q = 1.5 \times 2.3 = 3.50 \text{ kN/m} \)

Snow drift - average UDL

\[ Q = (1.35 \times 3.5 \times 0.5 \times 2/6.6) \times 2.3 = 1.64 \text{ kN/m} \]

ULS Combination:

\[ \gamma_G G + \gamma_Q Q = 1.35 \times 20.80 + 1.50 \times 3.50 = 33.90 \text{ kN/m} \]

Maximum moment at mid span:

\[ M_{y,Ed} = 0.125 \times 33.90 \times 6.60^2 = 184.6 \text{ kNm} \]

Maximum shear force at supports:

\[ V_{z,Ed} = 0.5 \times 33.90 \times 6.60 = 111.87 \text{ kN} \]

Construction Stage

Permanent load:

\[ G = 1.0 + (7.58 \times 2.3) = 18.434 \text{ kN/m} \]

Variable load:

Imposed load \( Q = 0.75 \times 2.3 = 1.73 \text{ kN/m} \)

ULS Combination:

\[ \gamma_G G + \gamma_Q Q = 1.35 \times 18.434 + 1.50 \times 1.73 = 27.48 \text{ kN/m} \]

Maximum moment at mid span:

\[ M_{y,Ed} = 0.125 \times 27.48 \times 6.60^2 = 149.63 \text{ kNm} \]

Maximum shear force at supports:

\[ V_{z,Ed} = 0.5 \times 27.48 \times 6.60 = 90.68 \text{ kN} \]
Yield strength
Steel grade S355. The maximum thickness is 10.9 mm < 40 mm, so \( f_y = 355 \text{ N/mm}^2 \)

With the full lateral restraint provided by the slab and the cross-section of the steel beam in Class 1, the resistance moment of the critical cross-section of the beam \( M_{Rd} \) at mid span is calculated by means of rigid-plastic theory.

The design value of the normal force in the structural steel section is given by:
\[
N_{c,Rd} = f_y \frac{W_{p}}{\gamma_{M0}} = 18840 \times 355 \times 10^{-3} / 1.0 = 6688 \text{ kN}
\]
For Permanent stage applied load 111.87 kN
For Construction stage applied load 90.68 kN

Plastic Resistance Moment at mid span
\[
M_{c,Rd} = f_y \frac{W_{ply}}{\gamma_{M0}} = 355 \times 1133 \times 10^{-3}/1.0 = 402.2 \text{ kN.m}
\]
For Permanent stage applied moment 184.60 kN.m
For Construction stage applied moment 149.63 kN.m

---

EN 1993-1-1
Equation 6.10
Equation 6.13
Shear Resistance

Longitudinal Shear Resistance of the Slab

The plastic longitudinal shear stresses is given by:

$$v_{Ed} = \frac{\Delta F_d}{h_f \Delta x}$$

Where  $$\Delta x = \frac{6.80}{2} = 3.40 \text{ m}$$

The value for  $$\Delta x$$ is half the distance between the section where the moment is zero and the section where the moment is maximum and we have two areas for the shear resistance.

$$\Delta F_d = 111.87 \times 2 = 223.74 \text{ kN}$$

$$h_f = h - h_p = 350 - 100 = 250 \text{ mm}$$

$$v_{Ed} = \frac{\Delta F_d (h_f \cdot \Delta x)}{253.5 \times 1000/(250 \times 3400)} = 0.30 \text{ N/mm}^2$$

To prevent crushing of the compression struts in the concrete flange, the following condition should be satisfied:

$$v_{Ed} < \sqrt{\frac{f_{yd} \sin \theta_f \cos \theta_f}{250}} \text{ with } v = 0.6 \left[1 - \frac{f_{ck}}{250}\right] \text{ and } \theta_f = 45^\circ$$

The following inequality should be satisfied for the transverse reinforcement:

$$A_{sf} f_{yd} / s_t \geq v_{Ed} h_f / \cot \theta_f \text{ where } f_{yd} = 500 / 1.15 = 435 \text{ N/mm}^2$$

Assume the spacing of the bars  $$s_t = 200 \text{ mm}$$ and there is no contribution from the profiled steel sheeting

$$A_{sf} \geq \frac{0.30 \times 180 \times 200}{435 \times 1.0} = 26 \text{ mm}^2$$

Recommended mesh area is a minimum of 0.1% of the cross sectional area of the concrete slab.

Therefore provide A393 mesh (393 mm$^2$/m) extending over the effective concrete breadth with 25mm nominal top cover.

Also 10mm diameter bar will be positioned in every trough at 30mm height (from bottom of the deck) to enhance the crack width and fire design stage performance.
Consider 1000mm width simply supported single span slab with maximum slab depth of 350mm. Actual slab is a continuous slab therefore actual bending moment and shear force within the slab will be less than simply supported single span slab.

Try H10 bar in every trough for sagging moment.

\[ A_s = (1000/295 + 1.0) \times 78.5 = 344 \text{mm}^2/\text{m} \]
\[ D = 350 - 30 - 10/2 = 315 \text{mm} \]

Slab bending moment \( M_{Ed} = 0.125 \times 33.90 \times 2.3 \times 2.3 = 22.42 \text{kN.m} \)

Slab shear force at support \( V_{Ed} = 0.5 \times 33.90 \times 2.3 = 39.0 \text{kN} \)

\[ K = (22.42 \times 10^6)/(40 \times 1000 \times 315^2) = 0.0056 \]

\[ Z = d\{(0.5 + (0.25 - K/0.9)^{0.5}) = 315 \times (0.5 + (0.25 - 0.0056/0.9)^{0.5}) = 313 \text{mm} \} \]

\[ M_{Rd} = 435 \times 344 \times 313 \times 10^{-6} = 46.92 \text{kN.m} > \text{Applied } M_{Ed} = 22.42 \text{kN.m} \]

Total reinforcement in cross section area = Top 393 + bottom 344 = 737 \text{mm}^2/\text{m}

Recommended total reinforcement area is a minimum of 0.13% of the cross sectional area of the concrete slab = 1000x350x0.13% = 455 \text{mm}^2/\text{m}

**OK**

**Serviceability Limit State verification**

**SLS Combination**

\[ G + Q = 23.48 + 3.9 = 27.18 \text{kN/m} \]

\[ \text{Deflection due to } G+Q : w = \frac{5(G+Q)I_y}{384EI_y} \]

Where \( I_y \) depends on the modular ratio \((n)\) depending on the type of loading. By simplification, we can take:

\[ n_0 = E_a / E_{cm} = 210 000 / 31476 = 6.67 \text{ for primary effects (Q)} \]

And \( n = 3E_a / E_{cm} = 20.02 \text{ for permanent loads (G)} \)

\[ I_y \text{ at mid span} = 17457 \text{ cm}^4 \]

\[ w = 5 \times 27.18 \times 6800^4/(384 \times 210000 \times 17457 \times 10^4) \]

\[ = 20.64 \text{mm} = \text{span/329} \text{ < limited span/250 for load combination} \]

**OK**
Check Bearing Stress within pad-stone under the steel beam

Bearing width \( b = \frac{(r + t_f) \cdot \sqrt{(r + \frac{t_w}{2})^2 - r^2}}{r} \)

Where:
- \( r \) is the root radius = 10.2mm
- \( t_w \) is the web thickness = 7.2mm
- \( t_f \) is the flange thickness = 11 mm

\[
\text{Bearing width} = \left\{ (10.2 + 11) \times \left( ((10.2 + 7.2/2)^2 - 10.2^2)^{0.5} \right) / 10.2 \right\} = 193.1\text{mm}
\]

Width of pad-stone = 660mm

Bearing area = 193.1 x 660 = 127446 mm\(^2\)

End reaction due to vertical downward force = 111.87 kN (See Page R-9)

Bearing stress in concrete pad-stone = 111.87 x 1000/127446
= 0.86 N/mm\(^2\)

allowable compressive stress in plain concrete = \( f_{cd,pl} = \frac{a_{cc,p} \times f_{ck}}{\gamma_c} \)

\( f_{ck} = 35 \text{ N/mm}^2 \) for C35/40
\( \gamma_c = 1.5 \)

Allowable compressive stress in plain concrete = 0.6 x 40/1.5
= 14 N/mm\(^2\)

\( \text{OK} \)
Check Bearing Stress under Concrete Pad-stone

Concrete Pad-stone size: 215mm wide x 660mm long x 225mm deep

Maximum End reaction due to vertical downward force = 111.87 kN (See Page R-9)

Bearing stress at underside of pad-stone = 111.87 x 1000/(215 x 660) = 0.78 N/mm²

From LUL 1061 A3: Civil Engineering – Bridges and Structures Assessment Standard,
Clause 3.4.2.1

The characteristic strength for the stock Masonry $f_k = 4.8$ N/mm²

From BS 5628-1 Code of practice for use of masonry

Allowable Bearing Stress $f_{cp} = \left( \gamma_{bear} \times f_k \right) / \gamma_m$

$\gamma_c = 2$ (fig 4(c))

$\gamma_m = 3.5$

$f_{cp} = \left( 2 \times 4.8 \right) / 3.5$

$= 2.74 < \text{Applied stress}$

OK
Replacement Roof Slab Construction Sequence

The construction of each area of new composite slab will be carried out as follow;

1. New non reinforced concrete pad-stones shall be installed at the location where the beam will bear on existing brick wall. Pad-stones will spread the beam applied reactions sufficiently within the bearing capacity wall.
2. The steel beams shall be placed over the concrete pad-stone, once the concrete has achieved design strength.
3. Formwork 100 gauge 1.2 profiled steel sheeting shall be placed over the upper surface of the bottom flange of the beams, all shear connectors connection shall be completed
4. 10mm diameter bar will be positioned in every trough at 30mm height (from bottom of the deck)
5. Laid A393 mesh extending over the concrete breadth with 25mm nominal top cover. The mesh shall be properly supported using chairs to ensure it doesn’t become displaced during concreting.
6. The surface of the decking should be reasonably free of dirt, oil etc. prior to concreting.
7. Concrete shall be poured to complete the construction of the roof slab.
8. The concrete shall be well compacted, particularly near and around any shear connectors.
9. The concrete shall be placed when air temperature is 5 degree or above.
10. Once the slab reached its design strength a low parapet brick wall will be constructed as existing.
11. Waterproofing finishes will be installed, including bond to the perimeter of existing grills directly above the vent shaft.
Assessment of the Existing Brick Wall (At the Rear)

The existing roof structure is solid 340mm concrete slab with asphalt finishes on the top, bear on the existing brick wall.

Self-weight of original concrete slab  = 24 x 0.34  = 8.16 kN/m²

The proposed roof structure will maintain the depth of the existing slab but utilize a profiled composite type deck of thin gauge steel which will reduced the self-weight of the roof. Refer to page R-5 & R9 for roof profile details. Proposed roof self-weight refer to page R-1

Self-weight of proposed roof  = 7.58 kN/m² < Self-weight of original concrete slab

Hence, no additional load occurred on the existing brick wall supporting proposed replacement roof. Therefore no determinably effect on the existing structure under new load due to the installation of the replacement upper roof structure.
Assessment of the Existing Beam supporting Roof Slab (At the Front)

Existing beam size : 12” x 6” x 44lbs

From Historical Structural Steelwork Handbook, Page 21

Self weight = 44 lbs/m = 66kg/m

D = 305 mm

t = 0.40 in = 10.16 mm

A = 12.95in² = 8355 mm²

Zxx = 52.57 in³ = 861.47 mm³

Ixx = 315.4 in⁴ = 131,279,391 mm⁴

Mₐ = 275 x 861.47 x 10⁻³ = 236.9 kN.m

Pₖ = 0.6 x 275 x 305 x 10.16 x 10⁻³ = 511 kN

Refer to Tedds Out-put Page Ex-1 to 2 for loading on existing steel beam supporting roof slab.

Design moment = 235.7 kN.m < Mₐ = 236.9 kN.m

Design Shear = 120.7 kN < Pₖ = 511 kN

Design deflection = 37.4 mm under DL + IL

Design deflection under Imposed Load = 37.4 x 11.39/(65.12 + 11.39)

= 5.56 mm = span /1151 < limited span/360

OK
CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = 1

Material Properties:
- Modulus of elasticity = 205 kN/mm²
- Material density = 7860 kg/m³

Support Conditions:
- Support A: Vertically "Restrained"; Rotationally "Free"
- Support B: Vertically "Restrained"; Rotationally "Free"

Span Definitions:
- Span 1: Length = 6410 mm; Cross-sectional area = 8355 mm²; Moment of inertia = 131.4 x 10⁶ mm⁴

LOADING DETAILS

Beam Loads:
- Load 1: UDL Dead load 0.7 kN/m
- Load 2: Point Dead load 65.1 kN at 1.800 m
- Load 3: Point Imposed load 11.4 kN at 1.800 m
- Load 4: Point Dead load 65.1 kN at 4.120 m
- Load 5: Point Imposed load 11.4 kN at 4.120 m

LOAD COMBINATIONS

Load combination 1 - DL + IL
- Span 1: 1.4 x Dead + 1.6 x Imposed

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary
- Support A: Max react = -120.7 kN; Min react = -120.7 kN; Max mom = 0.0 kNm; Min mom = 0.0 kNm
- Support B: Max react = -104.0 kN; Min react = -104.0 kN; Max mom = 0.0 kNm; Min mom = 0.0 kNm

Beam Max/Min results - Combination Summary
- Maximum shear = 120.7 kN
- Minimum shear Fmin = -104.0 kN
- Maximum moment = 235.7 kNm
- Minimum moment = 0.0 kNm
- Maximum deflection = 37.4 mm
- Minimum deflection = 0.0 mm
Shear Force Envelope

0.0
120.716
-103.991

kN

mm

A

B

120.7

6410

235.717

kNm

mm

A

B

120.7

6410

Deflections - Combination 1 - DL + IL

0.0
37.405

mm

A

B

120.7

6410

Shear Force - Combination 1 - DL + IL

0.0
120.716
-103.991

kN

mm

A

B

120.7

6410

Bending Moment - Combination 1 - DL + IL

0.0
235.717

kNm

mm

A

B

235.7

6410
SNOW LOADING (EN1991-1-3)

In accordance with EN1991-1-3:2003 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex incorporating Corrigendum No.1

TEDDS calculation version 1.0.03

Characteristic ground snow load

Location: London
Site altitude above sea level: A = 20 m
Zone number: Z = 3.0
Density of snow: γ = 2.00 kN/m³

Characteristic ground snow load:

\[ s_k = ((0.15 + (0.1 \times Z + 0.05)) + ((A - 100 m) / 525m)) \times 1kN/m^2 = 0.35 kN/m^2 \]

Exposure coefficient (Normal):

\[ C_e = 1.0 \]

Thermal coefficient:

\[ C_t = 1.0 \]

Building details

Roof type: Flat
Width of roof: b = 19.00 m

Parapet details

Height of parapet LHS: \( h_{pptL} = 0.70 \) m
Height of parapet RHS: \( h_{pptR} = 0.70 \) m

Shape coefficients

Shape coefficient roof (Table 5.2):

\[ \mu_1 = 0.80 \]

Parapet details

Length of snow drift LHS parapet (Annex B4.4):

\[ l_{s, pptL} = \min(5 \times h_{pptL}, b, 15m) = 3.50 \text{ m} \]

Shape coefficient LHS parapet (Annex B4.4):

\[ \mu_{1, pptL} = \min(2kN/m^3 \times h_{pptL} / s_k, 2 \times b / l_{s, pptL}, 8) = 4.03 \]

Length of snow drift RHS parapet (Annex B4.4):

\[ l_{s, pptR} = \min(5 \times h_{pptR}, b, 15m) = 3.50 \text{ m} \]

Shape coefficient RHS parapet (Annex B4.4):

\[ \mu_{1, pptR} = \min(2kN/m^3 \times h_{pptR} / s_k, 2 \times b / l_{s, pptR}, 8) = 4.03 \]

Loadcase 1 Table 5.2

Loading to roof 1:

\[ s_{1,1} = \mu_1 \times C_e \times C_t \times s_k = 0.28 \text{ kN/m²} \]

Parapet Annex B4.4

Loading to roof 1 (LHS):

\[ s_{1,2} = \mu_{1, pptL} \times s_k = 1.40 \text{ kN/m²} \]

Loading to roof 1 (RHS):

\[ s_{1,3} = \mu_{1, pptR} \times s_k = 1.40 \text{ kN/m²} \]
**SNOW LOADING (EN1991-1-3)**

In accordance with EN1991-1-3:2003 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex incorporating Corrigendum No.1

**Characteristic ground snow load**

<table>
<thead>
<tr>
<th>Location</th>
<th>London</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site altitude above sea level</td>
<td>A = 20 m</td>
</tr>
<tr>
<td>Zone number</td>
<td>Z = 3.0</td>
</tr>
<tr>
<td>Density of snow</td>
<td>γ = 2.00 kN/m³</td>
</tr>
</tbody>
</table>

Characteristic ground snow load

\[ s_k = ((0.15 + (0.1 \times Z + 0.05)) + ((A - 100m) / 525m)) \times 1kN/m^2 = 0.35 \text{ kN/m}^2 \]

- Exposure coefficient (Normal): \( C_e = 1.0 \)
- Thermal coefficient: \( C_t = 1.0 \)

**Building details**

<table>
<thead>
<tr>
<th>Roof type</th>
<th>Flat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of roof</td>
<td>b = 6.80 m</td>
</tr>
</tbody>
</table>

**Parapet details**

| Height of parapet LHS | \( h_{pptL} = 0.70 \text{ m} \) |
| Height of parapet RHS | \( h_{pptR} = 0.70 \text{ m} \) |

**Shape coefficients**

| Shape coefficient roof (Table 5.2) | \( \mu_1 = 0.80 \) |

**Parapet details**

| Length of snow drift LHS parapet (Annex B4.4) | \( l_{s,pptL} = \min(5 \times h_{pptL}, b, 15m) = 3.50 \text{ m} \) |
| Shape coefficient LHS parapet (Annex B4.4) | \( \mu_{1,pptL} = \min(2kN/m^3 \times h_{pptL} / s_k, 2 \times b / l_{s,pptL}, 8) = 3.89 \) |
| Length of snow drift RHS parapet (Annex B4.4) | \( l_{s,pptR} = \min(5 \times h_{pptR}, b, 15m) = 3.50 \text{ m} \) |
| Shape coefficient RHS parapet (Annex B4.4) | \( \mu_{1,pptR} = \min(2kN/m^3 \times h_{pptR} / s_k, 2 \times b / l_{s,pptR}, 8) = 3.89 \) |

<table>
<thead>
<tr>
<th>Case (i)</th>
<th>Shape coef</th>
<th>Coef</th>
<th>Loading (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \mu_1 )</td>
<td>0.800</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>( h_{pptL} )</td>
<td>3.886</td>
<td>1.35</td>
<td></td>
</tr>
<tr>
<td>( h_{pptR} )</td>
<td>3.886</td>
<td>1.35</td>
<td></td>
</tr>
</tbody>
</table>

**Loadcase 1 Table 5.2**

| Loading to roof 1 | \( S_{1,1} = \mu_1 \times C_e \times C_t \times s_k = 0.28 \text{ kN/m}^2 \) |

**Parapet Annex B4.4**

| Loading to roof 1 (LHS) | \( S_{1,2} = \mu_{1,pptL} \times s_k = 1.35 \text{ kN/m}^2 \) |
| Loading to roof 1 (RHS) | \( S_{1,3} = \mu_{1,pptR} \times s_k = 1.35 \text{ kN/m}^2 \) |
WIND LOADING (EN1991-1-4)

TEDDS calculation version 3.0.11

Building data
- Type of roof: Flat
- Length of building: L = 19000 mm
- Width of building: W = 6800 mm
- Eaves type: Parapet
- Height of parapet: h_p = 700 mm
- Total height: h = 6300 mm

Basic values
- Location: London
- Wind speed velocity (FigNA.1) \( v_{b,\text{map}} = 21.4 \) m/s
- Altitude above sea level: \( A_{\text{alt}} = 20.0 \) m
- Fund wind speed velocity: \( v_{b,0} = 21.8 \) m/s
- Season factor: \( c_{\text{season}} = 1.00 \)
- Basic wind speed (Exp. 4.1): \( v_b = 21.8 \) m/s

Orography
- Orography factor not signif: \( c_o = 1.0 \)
- Terrain category: Town
- Displacement height: \( H_d = 0 \) mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height \( h \) is less than \( b \) (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 2 parts as the height \( h \) is greater than \( b \) but less than \( 2b \) (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof
- Reference height: \( z = 7000 \) mm
- Exposure factor (Figure NA.7): \( c_o = 2.13 \)
- Exposure correction factor (Figure NA.8): \( c_{o,T} = 0.88 \)
- Peak velocity pressure: \( q_p = 0.55 \) kN/m²

Structural factor
- Structural damping: \( \delta_s = 0.100 \)
- Size factor (Table NA.3): \( c_e = 0.83 \)
- Dynamic factor (Figure NA.9): \( c_d = 1.00 \)
- Structural factor: \( c_{e,d} = 0.832 \)

Peak velocity pressure - windward wall (lower part) - Wind 90 deg
- Reference height: \( z = 6800 \) mm
- Exposure factor (Figure NA.7): \( c_o = 2.11 \)
- Exposure correction factor (Figure NA.8): \( c_{o,T} = 0.87 \)
- Peak velocity pressure: \( q_p = 0.54 \) kN/m²
### Structural factor

- Structural damping \( \delta_s = 0.100 \)
- Size factor (Table NA.3) \( c_s = 0.87 \)
- Dynamic factor (Figure NA.9) \( c_d = 1.00 \)
- Structural factor \( c_sCd = 0.832 \)

### Peak velocity pressure - windward wall (upper part) - Wind 90 deg and roof

- Reference height \( z = 7000 \text{mm} \)
- Exposure factor (Figure NA.7) \( c_e = 2.13 \)
- Exposure correction factor (Figure NA.8) \( c_{e,T} = 0.88 \)
- Peak velocity pressure \( q_p = 0.55 \text{kN/m}^2 \)

### Pressures and forces

- Net pressure \( p = c_sCd \times q_p \times c_{pe} - q_{p,i} \times c_{pi} \)
- Net force \( F_w = pW \times A_{ref} \)

#### Roof load case 1 - Wind 0, \( c_{pi} 0.20, -c_{pe} \)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient ( c_{pe} )</th>
<th>Peak velocity pressure ( q_p ), (kN/m²)</th>
<th>Net pressure ( p ), (kN/m²)</th>
<th>Area ( A_{ref} ), (m²)</th>
<th>Net force ( F_w ), (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F (-ve)</td>
<td>-1.80</td>
<td>0.55</td>
<td>-0.93</td>
<td>7.94</td>
<td>-7.35</td>
</tr>
<tr>
<td>G (-ve)</td>
<td>-1.27</td>
<td>0.55</td>
<td>-0.68</td>
<td>16.00</td>
<td>-10.94</td>
</tr>
<tr>
<td>H (-ve)</td>
<td>-0.70</td>
<td>0.55</td>
<td>-0.43</td>
<td>95.76</td>
<td>-40.84</td>
</tr>
<tr>
<td>I (-ve)</td>
<td>-0.20</td>
<td>0.55</td>
<td>-0.20</td>
<td>9.50</td>
<td>-1.90</td>
</tr>
</tbody>
</table>

Total vertical net force \( F_{w,v} = -61.03 \text{kN} \)
Total horizontal net force \( F_{w,h} = 0.00 \text{kN} \)

#### Walls load case 1 - Wind 0, \( c_{pi} 0.20, -c_{pe} \)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient ( c_{pe} )</th>
<th>Peak velocity pressure ( q_p ), (kN/m²)</th>
<th>Net pressure ( p ), (kN/m²)</th>
<th>Area ( A_{ref} ), (m²)</th>
<th>Net force ( F_w ), (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-1.20</td>
<td>0.55</td>
<td>-0.65</td>
<td>15.88</td>
<td>-10.37</td>
</tr>
<tr>
<td>B</td>
<td>-0.80</td>
<td>0.55</td>
<td>-0.47</td>
<td>26.96</td>
<td>-12.72</td>
</tr>
<tr>
<td>D</td>
<td>0.79</td>
<td>0.55</td>
<td>0.25</td>
<td>119.70</td>
<td>29.84</td>
</tr>
<tr>
<td>E</td>
<td>-0.48</td>
<td>0.55</td>
<td>-0.33</td>
<td>119.70</td>
<td>-39.13</td>
</tr>
</tbody>
</table>

#### Overall loading

- Leeward force overall \( F_l = -39.1 \text{kN} \)
- Windward force overall \( F_w = 29.8 \text{kN} \)
- Lack of correlation (cl.7.2.2(3)) \( f_{corr} = 0.85 \)
- Overall loading overall section \( F_{w,D} = 58.6 \text{kN} \)

#### Roof load case 2 - Wind 0, \( c_{pi} -0.3, +c_{pe} \)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient ( c_{pe} )</th>
<th>Peak velocity pressure ( q_p ), (kN/m²)</th>
<th>Net pressure ( p ), (kN/m²)</th>
<th>Area ( A_{ref} ), (m²)</th>
<th>Net force ( F_w ), (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F (+ve)</td>
<td>-1.80</td>
<td>0.55</td>
<td>-0.65</td>
<td>7.94</td>
<td>-5.18</td>
</tr>
<tr>
<td>Zone</td>
<td>Ext pressure coefficient $c_{pe}$</td>
<td>Peak velocity pressure $q_{p}$ (kN/m$^2$)</td>
<td>Net pressure $p$ (kN/m$^2$)</td>
<td>Area $A_{ref}$ (m$^2$)</td>
<td>Net force $F_w$ (kN)</td>
</tr>
<tr>
<td>------</td>
<td>----------------------------------</td>
<td>------------------------------------------</td>
<td>----------------------------</td>
<td>----------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>F (+ve)</td>
<td>-1.80</td>
<td>0.55</td>
<td>-0.65</td>
<td>7.94</td>
<td>-5.18</td>
</tr>
<tr>
<td>G (+ve)</td>
<td>-1.27</td>
<td>0.55</td>
<td>-0.41</td>
<td>16.00</td>
<td>-6.57</td>
</tr>
<tr>
<td>H (+ve)</td>
<td>-0.70</td>
<td>0.55</td>
<td>-0.15</td>
<td>95.76</td>
<td>-14.73</td>
</tr>
<tr>
<td>I (+ve)</td>
<td>0.20</td>
<td>0.55</td>
<td>0.25</td>
<td>9.50</td>
<td>2.42</td>
</tr>
</tbody>
</table>

Total vertical net force $F_{w,v} = -24.07$ kN  
Total horizontal net force $F_{w,h} = 0.00$ kN

### Walls load case 2 - Wind 0, $c_{pe} -0.3, +c_{pe}$

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient $c_{pe}$</th>
<th>Peak velocity pressure $q_{p}$ (kN/m$^2$)</th>
<th>Net pressure $p$ (kN/m$^2$)</th>
<th>Area $A_{ref}$ (m$^2$)</th>
<th>Net force $F_w$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-1.20</td>
<td>0.55</td>
<td>-0.38</td>
<td>15.88</td>
<td>-6.04</td>
</tr>
<tr>
<td>B</td>
<td>-0.80</td>
<td>0.55</td>
<td>-0.20</td>
<td>26.96</td>
<td>-5.37</td>
</tr>
<tr>
<td>D</td>
<td>0.79</td>
<td>0.55</td>
<td>0.52</td>
<td>119.70</td>
<td>62.48</td>
</tr>
<tr>
<td>E</td>
<td>-0.48</td>
<td>0.55</td>
<td>0.05</td>
<td>119.70</td>
<td>-6.49</td>
</tr>
</tbody>
</table>

### Overall loading

Leeward force overall $F_l = -6.5$ kN  
Windward force overall $F_w = 62.5$ kN

Lack of correlation (cl.7.2.2(3)) $f_{corr} = 0.85$  
Overall loading overall section $F_{w,D} = 58.6$ kN

### Roof load case 3 - Wind 90, $c_{pe}$ 0.20, $-c_{pe}$

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient $c_{pe}$</th>
<th>Peak velocity pressure $q_{p}$ (kN/m$^2$)</th>
<th>Net pressure $p$ (kN/m$^2$)</th>
<th>Area $A_{ref}$ (m$^2$)</th>
<th>Net force $F_w$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F (-ve)</td>
<td>-1.80</td>
<td>0.55</td>
<td>-0.93</td>
<td>2.31</td>
<td>-2.14</td>
</tr>
<tr>
<td>G (-ve)</td>
<td>-1.27</td>
<td>0.55</td>
<td>-0.68</td>
<td>2.31</td>
<td>-1.58</td>
</tr>
<tr>
<td>H (-ve)</td>
<td>-0.70</td>
<td>0.55</td>
<td>-0.43</td>
<td>18.50</td>
<td>-7.89</td>
</tr>
<tr>
<td>I (-ve)</td>
<td>-0.20</td>
<td>0.55</td>
<td>-0.20</td>
<td>106.08</td>
<td>-21.19</td>
</tr>
</tbody>
</table>

Total vertical net force $F_{w,v} = -32.80$ kN  
Total horizontal net force $F_{w,h} = 0.00$ kN

### Walls load case 3 - Wind 90, $c_{pe}$ 0.20, $-c_{pe}$

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient $c_{pe}$</th>
<th>Peak velocity pressure $q_{p}$ (kN/m$^2$)</th>
<th>Net pressure $p$ (kN/m$^2$)</th>
<th>Area $A_{ref}$ (m$^2$)</th>
<th>Net force $F_w$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-1.20</td>
<td>0.55</td>
<td>-0.65</td>
<td>8.57</td>
<td>-5.60</td>
</tr>
<tr>
<td>B</td>
<td>-0.80</td>
<td>0.55</td>
<td>-0.47</td>
<td>34.27</td>
<td>-16.17</td>
</tr>
<tr>
<td>C</td>
<td>-0.50</td>
<td>0.55</td>
<td>-0.34</td>
<td>76.86</td>
<td>-25.81</td>
</tr>
<tr>
<td>Db</td>
<td>0.71</td>
<td>0.54</td>
<td>0.21</td>
<td>46.24</td>
<td>9.68</td>
</tr>
<tr>
<td>Dd</td>
<td>0.71</td>
<td>0.55</td>
<td>0.21</td>
<td>42.84</td>
<td>-0.73</td>
</tr>
<tr>
<td>E</td>
<td>-0.32</td>
<td>0.55</td>
<td>-0.25</td>
<td>42.84</td>
<td>-10.92</td>
</tr>
</tbody>
</table>

### Overall loading

Leeward force upper $F_l = 0.9$ kN  
Windward force upper $F_w = -0.7$ kN

Lack of correlation (cl.7.2.2(3)) $f_{corr} = 0.85$  
Overall loading upper section $F_{w,u} = -1.4$ kN

Leeward force bottom $F_l = -11.8$ kN  
Windward force bottom $F_w = 9.7$ kN

Lack of correlation (cl.7.2.2(3)) $f_{corr} = 0.85$  
Overall loading bottom section $F_{w,b} = 18.3$ kN
Roof load case 4 - Wind 90, $c_p = -0.3, +c_{pe}$

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient $c_{pe}$</th>
<th>Peak velocity pressure $q_{0p}$ (kN/m²)</th>
<th>Net pressure $p$ (kN/m²)</th>
<th>Area $A_{ref}$ (m²)</th>
<th>Net force $F_w$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F (+ve)</td>
<td>-1.80</td>
<td>0.55</td>
<td>-0.65</td>
<td>2.31</td>
<td>-1.51</td>
</tr>
<tr>
<td>G (+ve)</td>
<td>-1.27</td>
<td>0.55</td>
<td>-0.41</td>
<td>2.31</td>
<td>-0.95</td>
</tr>
<tr>
<td>H (+ve)</td>
<td>-0.70</td>
<td>0.55</td>
<td>-0.15</td>
<td>18.50</td>
<td>-2.85</td>
</tr>
<tr>
<td>I (+ve)</td>
<td>0.20</td>
<td>0.55</td>
<td>0.25</td>
<td>106.08</td>
<td>26.98</td>
</tr>
</tbody>
</table>

Total vertical net force $F_{w,v} = 21.67$ kN

Walls load case 4 - Wind 90, $c_p = -0.3, +c_{pe}$

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ext pressure coefficient $c_{pe}$</th>
<th>Peak velocity pressure $q_{0p}$ (kN/m²)</th>
<th>Net pressure $p$ (kN/m²)</th>
<th>Area $A_{ref}$ (m²)</th>
<th>Net force $F_w$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-1.20</td>
<td>0.55</td>
<td>-0.38</td>
<td>8.57</td>
<td>-3.26</td>
</tr>
<tr>
<td>B</td>
<td>-0.80</td>
<td>0.55</td>
<td>-0.20</td>
<td>34.27</td>
<td>-6.83</td>
</tr>
<tr>
<td>C</td>
<td>-0.50</td>
<td>0.55</td>
<td>-0.06</td>
<td>76.86</td>
<td>-4.85</td>
</tr>
<tr>
<td>D_b</td>
<td>0.71</td>
<td>0.54</td>
<td>0.48</td>
<td>46.24</td>
<td>22.29</td>
</tr>
<tr>
<td>D_u</td>
<td>0.71</td>
<td>0.55</td>
<td>0.49</td>
<td>-3.40</td>
<td>-1.65</td>
</tr>
<tr>
<td>E</td>
<td>-0.32</td>
<td>0.55</td>
<td>0.02</td>
<td>42.84</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Overall loading

Leeward force upper $F_l = -0.1$ kN
Lack of correlation (cl.7.2.2(3)) $f_{corr} = 0.85$
Leeward force bottom $F_l = 0.8$ kN
Lack of correlation (cl.7.2.2(3)) $f_{corr} = 0.85$

Windward force upper $F_w = -1.7$ kN
Overall loading upper section $F_{w,u} = -1.4$ kN
Windward force bottom $F_w = 22.3$ kN
Overall loading bottom section $F_{w,b} = 18.3$ kN
Project: South Kensington Station Asset Stabilisation
Job no.: 3095-003

Calcs for: Permanent works to the upper roof
Start page no./Revision: WL - 5

Calcs by: KW
Calcs date: 09/12/2013

Checked by: Checked date: Approved by: Approved date:

A

B

2520

4280

6800

Windward face

C

D

E

6300

19000

F

G

H

I

6800

6800

19000

2720

15600

Wind - 90°

Plan view - Flat roof

1700

3400

1700

3400

6300

19000

Side face

Leeward face
Project: South Kensington Station Asset Stabilisation
Job no.: 3095-003

Calcs for: Permanent works to the upper roof
Start page no./Revision: WL - 6

Calcs by: KW
Calcs date: 09/12/2013
Checked by: A
Checked date: B
Approved by: C
Approved date: D

Calcs:

- Windward face: 6800 x 6300
- Side face: 19000 x 6300
- Leeward face: 6800 x 6300

Dimensions:
- 12200 x 6300
- 5440 x 6300

Units:
- Side face: 1360
- Windward face: 19000
- Leeward face: 6800

Notes:
- Calculations by KW
- Checked by A, Checked date: B
- Approved by C, Approved date: D