BURO HAPPOLD

Barbican Installation - Ibrahim Mahama

Structural Engineering Report

BAR-BHE-RP-231122

P059932

22 November 2023

Revision P01

Revision	Description	Issued by	Date	Checked
P01	For Information	Maxime C.	22/11/2023	Anton S.

Report Disclaimer

author	Maxime Chollet
date	22/11/2023
approved	Anton Sawicki
signature	[redacted]
date	22/11/2023

Contents

1 Design Criteria

This Design Note is to cover the Structural Basis of Design and Calculations of the temporary artwork installation by Ibrahim Mahama Artwork. Th artwork is to be installed on the elevation of the Barbican facing Lakeside. This note should be read in conjunction with the other documents part of the Special Structure Application to be submitted to Building Control.

The following design criteria are considered when developing the structural proposals.

1.1 Design Standards & Guidance

The following design standards and their relevant national application documents have been used to develop the design.

ID	Reference	Description
[1]	BS EN 1990:2002 + A1:2005	Eurocode 0 - Basis of Design
[2]	BS EN 1991-1-1:2002	Eurocode 1 – Actions on structures Part 1-1: General actions – Densities, self-weight, imposed loads for buildings
[3]	BS EN 1991-1-3:2003	Eurocode 1 – Actions on structures Part 1-3: General actions – Snow actions
[4]	BS EN 1991-1-4:2005 + A1 2010	Eurocode 1 – Actions on structures Part 1-4: General actions – Wind actions
[5]	BS EN 1991-1-6:2005	Eurocode 1 – Actions on structures Part 1-7: General actions – Actions during execution
[6]	BS EN 1991-1-7:2006	Eurocode 1 – Actions on structures Part 1-7: General actions – Accidental actions
[7]	BS EN 1993-1-1:2005	Eurocode 3 – Design of steel structures Part 1-1: General rules and rules for buildings
[8]	-	Temporary demountable structures Guidance of procurement, design and use – Fourth Edition Institution of Structural Engineers, 2017
[9]	-	Met Office climate online database 30-year averages (1991-2020) https://www.metoffice.gov.uk/research/climate/maps-and-data/uk-climate-averages/gcpsvg3nc

1.2 Loading

1.2.1 Self-Weight

Element	Description	Loading
Fabric + Mesh	Dry Wet (fully saturated)	6.0 kg/m^2 11.0 kg/m^2
Steel Frame	-	7850.0 kg/m^3

Wind Loading

The wind loading to be applied in the analysis of permanent structures in the UK is determined by BS EN 1991-1-4 [4]. This part of Eurocode 1 is also ideally suited to the design of temporary structures for specific locations and periods of use.

However, due to the nature of our structure, i.e. not resisting any loads (especially live) excepted its own self-weight and wind pressure and being in place for a period of 4 months only, it is considered appropriate to allow an alternative design approach (7.6.2 [8]). This approach consists of:

1) the definition of design wind speed thresholds based on the Beauford Scale and local conditions agreed and the Client and Building Control.

2) a comprehensive wind management plan defining the actions to be taken if those thresholds are exceeded.

The approach and strategy has been discussed and reviewed with Building Control. The proposed wind speed to be considered for the design as presented below.

Figure 1: Beaufort wind force scale

Threshold	Description	Wind speed [m/s] / [mph]	Beaufort Scale Level
Design Wind Speed (DWS)	Ultimate wind speed the temporary structure will be designed to withstand	25.0 / 56.0	10
Operational Wind Speed (OWS)Ca	Maximum wind speed at which the fabric can be safely deposed by the Specialist Contractor. OR the temporary structure does not pose a hazard to the public	19.0 / 43.0	8
Max Mean Wind Speed (MWS)	Maximum monthly mean wind speed at 10m over the period January to June [9] (Station: Heathrow weather station)	4.2 / 9.4	3

Design Wind Speed - ULS

Design Wind Speed $v_{c,g\ b} = 25.0\ {}^{n}/_{t}$

Basic velocity pressure $r_{c,g\ b} = \frac{1}{2}\rho_{b\ i} v_{s}^{2} = 0.391\ l\ \ Q\ \ b$

Force Coefficient, [3] Table 7.15 (fixed flag) $d_g = 1.80$

Characteristic Design Wind Pressure $\boldsymbol{r_{q\ U\ M} = 1.8\ \ 1 l 4 P\ \ a}$

Design Wind Speed - SLS

Design Wind Speed	$v_{c,tu\ s} = 4.20\ ^{n}/_{t}$
Basic velocity pressure	$r_{c,t\ u\ s} = \frac{1}{2}\rho_{b\ i}\, v_{s}^{2} = 0.011\ l\ \ Q\ \ b$
Force Coefficient, [3] Table 7.15 (fixed flag)	$d_{g} = 1.80$
Characteristic Design Wind Pressure	$\boldsymbol{r_{q\ IT\ M\ T} = 1.1\ 3\ \ P\ \ a}$

The wind speed above is considered as peak velocity pressure according to BS EN 1991-1-4 [4] and partial load coefficient will still be applied. Therefore, the following wind pressure are going to be used to verify the design.

1.2.2 Snow

Considering the setting out of the fabric, its location and period of installation (February to May), it is considered very unlikely snow would be able to accumulate.

All fabric panels are vertical, except the stage 2B models which are at a 66° angle which is more than the maximum 60° inclination considered by BS EN 1991-1-3 [3] for snow accumulation.

As a precaution, in the event of an extreme event, the Barbican management team will monitor the weather conditions for snow falls.

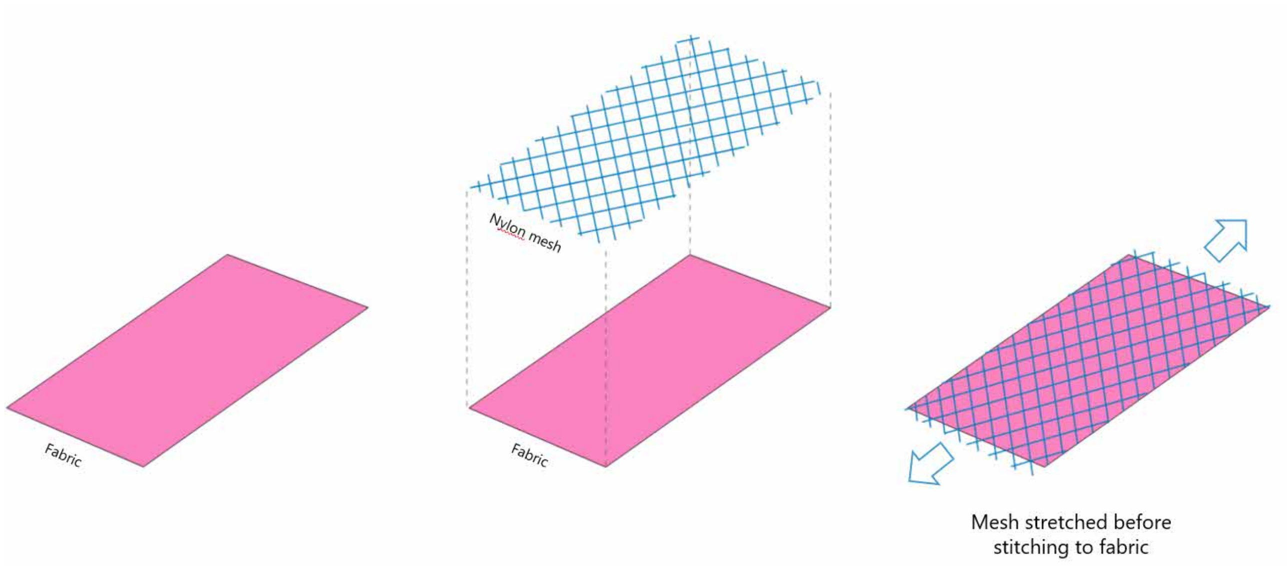
1.3 Materials

1.3.1 Fabric and mesh

The Fabric is made up of a coloured cotton material that has been weaved in Ghana.

The strips of material will be stitched together to the cutting panels and then stitched to the nylon mesh that will be acting as the tensile support to the fabric and will be attached to the steel frame and wire ropes.

The characteristics of the composite fabric and mesh construction will be confirmed after sample testing is completed.



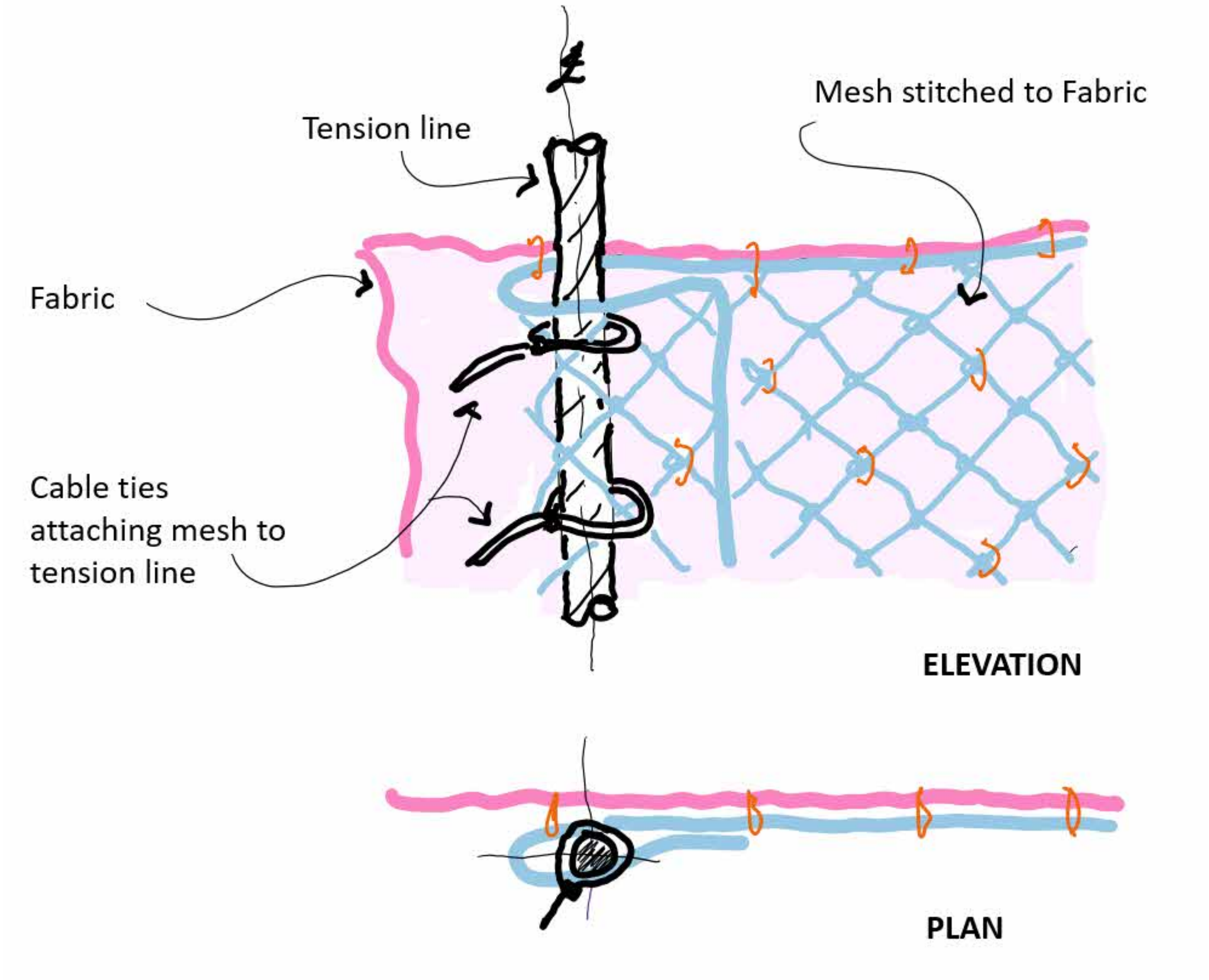


Figure 2: Fabric build-up & fastening to tension line

1.3.2 Supporting Structure

The supporting is proposed to be made of made of either steel or aluminium frame. It could be either traditional scaffolding products or rental aluminium frame (Prolyte of similar). To avoid any damage to the structure, the supporting structure will not be physically fixed to the existing structure. Diffusion of resulting forces will achieve either by direct contact or friction.

Figure 3: Surface protection (Correx or similar)


Figure 4: Supporting Frame (traditional scaffold or Prolyte or similar)


Figure 5: Kentledge (water tank, sandbags, steel plates or similar)

1.3.3 Wire Rope

For the calculations, steel wire rope assumed to be either galvanised or stainless steel with $f_u = 1770\ N/mm^2$ with nominal diameter ranging from 2 to 26mm.

2 Performance Criteria

2.1 Design Life

The intention is that the artwork will be installed for 4 months during February and May 2024.

Structure considered as Category 1 in accordance with EC0 [1] applicable to temporary structure.

2.2 Durability

All structural components assumed to be graded for external use.

Steel elements to be mild steel with a pint application or stainless steel.

2.3 Fire Protection

Fabric to be protected with fire retardant to comply with Class B S3 D2. Or similar approved.

Fabric will be set at least 4.0m above terrace level to prevent accidental ignition by the public.

2.4 Deflections

If and where needed, steel wire ropes will be pre-tensioned to limit displacements below tolerances values considered.

2.5 Tolerances

A positional tolerance of 25mm has been considered between the existing frame and the artwork supporting frame.

2.6 Noise consideration from wind

The installation is a taut cotton fabric, with low permeability to air.

It will be stretched and stitched onto the nylon mesh backing across the building and between trusses and strand stays. The proposed strategy will give comfort that the risk of wind induced noise is low. The low induced noise will be heard with the ambient noise levels of the gusting wind within the area which will be the more prominent noise level.

3 Proposed Structural System

The proposed structural aims to fulfil the following objectives:

Objective	Design Principle
Integration to the artwork	Structural system enabling the realisation of the artist vision
Preserve the existing structure	All existing surface and edge in contact with temporary structure protected (Correx or similar). No direct physical connection to the existing structure. Diffusion of resulting forces done either by direct contact with structure or through friction.
Easy, safe and quick installation	Clear and comprehensive method statement
Minimize the need for heavy lifting equipment and special equipment	Top to bottom installation method favoured whenever possible.

To achieve these objectives, the structural system consists of a combination of steel frame and galvanised steel strand anchored via water-filled kentledge or through direct contact with existing structure where possible. The proposed system has been discussed and reviewed with specialist subcontractors their feasibility and compliance to the design requirements mentioned above. The proposed structural system is briefly described below. For more details, please refer to the Structural Sketches Set in appendix of this report.

For structural design purpose, the artwork has been split into 3 sections:

1. The Towers
2. Roof to Balcony
3. Balcony to Lakeside terrace

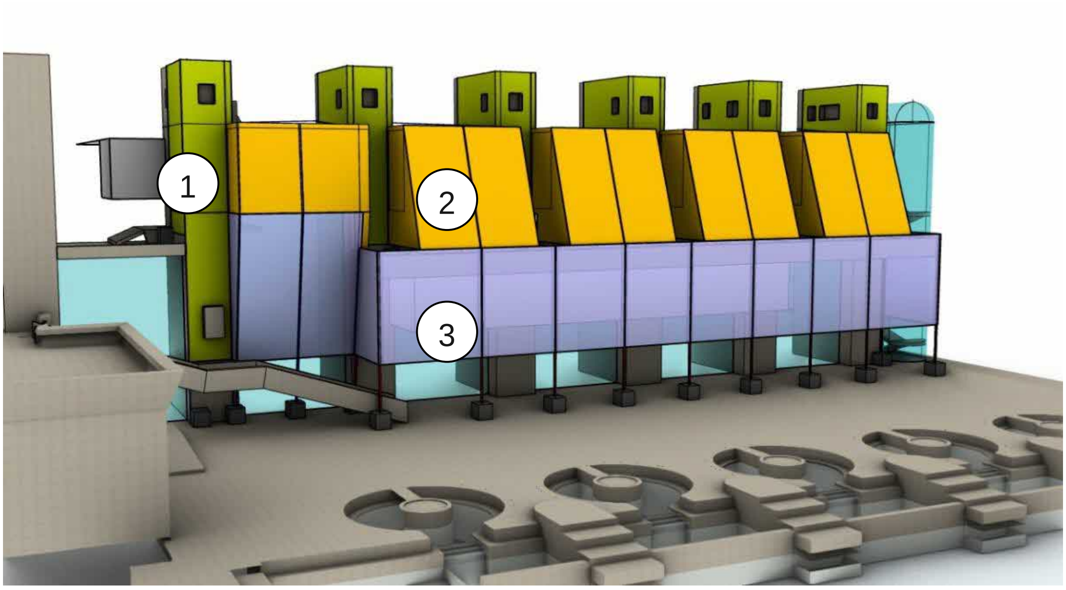


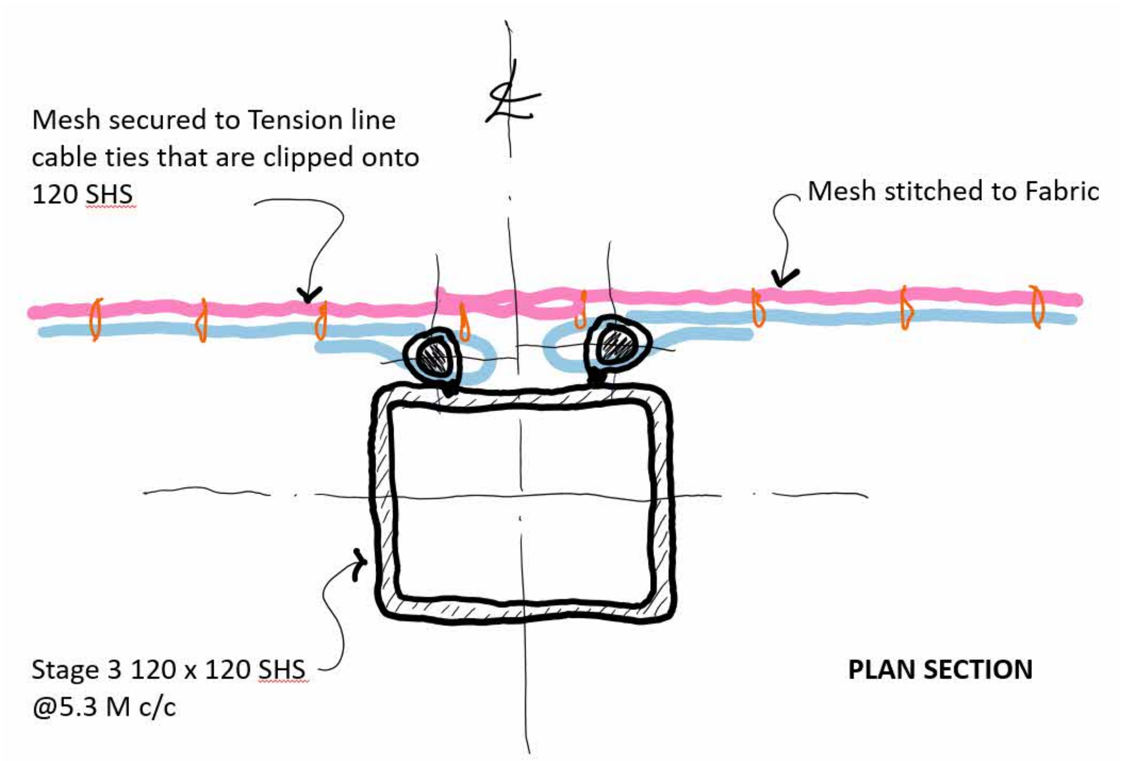
Figure 6: The 3 sections: 1 – the Towers (green), 2 – Roof to balcony (orange), 3 – Balcony to Lakeside Terrace (purple)

The section 1 consists of fabric panels applied right against the tower faces. The fabric is hung to the top of the tower. A light steel frame will be installed behind it to provide fixing points for the fabric and strands. The steel frame is maintained in position using adjustable steel props spread transversally. At the bottom, the fabric edge is secured using strands connected to kentledges on the balcony, first and terrace floors.

The section 2 comprises a network of steel strands positioned along the folding and main stitching lines and tied to 4-chords trusses on roof and balcony levels. At roof level, these edge trusses are then stabilised using horizontal cross-bracing wrapped around the towers. The reason for this choice of system is that, upon our initial site inspection, we observed the detachment of the waterproofing on the flat roof. There is a notable risk that any further weight imposed on the roof deck could potentially lead to damage to the waterproofing membrane. Therefore, to mitigate this risk, we have modified our structural design to prevent the transmission of any load through the primary roof deck hence the use of cross-bracing. While some access will still be necessary for assembly and disassembly, surface protection is considered. Nonetheless, it is crucial to exercise caution when working at this level. At balcony level, the trusses are secured by adjustable steel props extended across the planters taking advantage of the existing concrete parapets.

In addition, the system has been conceived so the fabric can be retracted and case the wind velocity should exceed those agreed in the design requirement. For more information on that topic please refer to the Wind Management plan below and the Structural Sketches Set.

The section 3, due to its span and its exposure to wind, consists of vertical 120 SHS @5.30m c/c spanning from the Lakeside terrace to the balcony, restrained at their base by water-filled kentledges. Smaller CHS's are laid alongside the bottom edge of the fabric, 4.0m above the terrace finish level. As for the stage 2, all panels in the section are either fully or partly retractable.



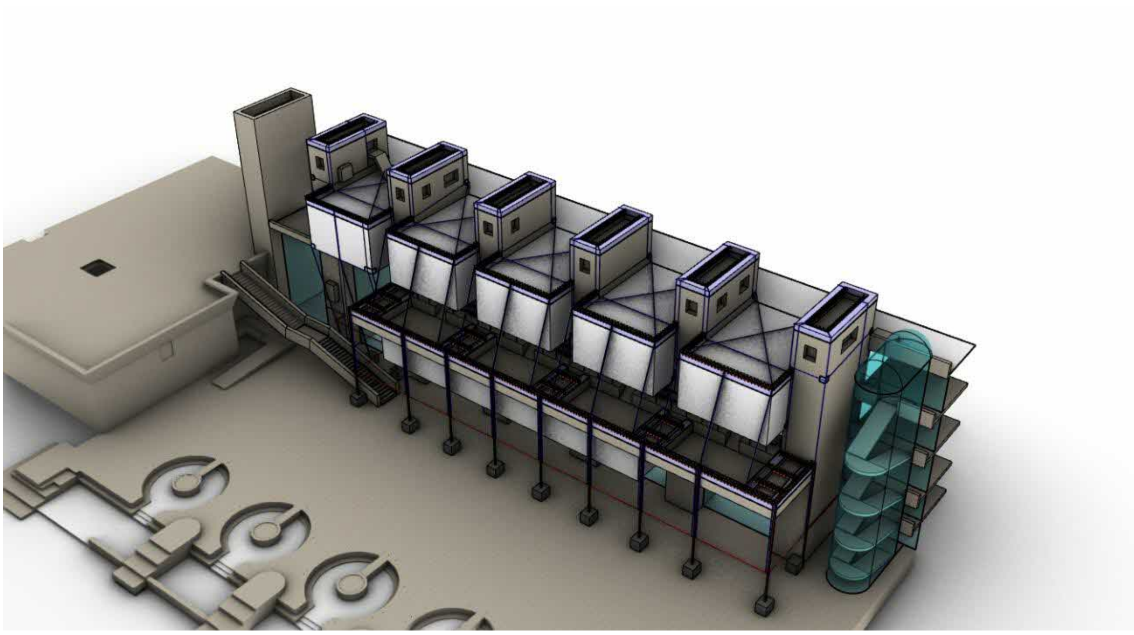


Figure 7: Proposed Structural System, S-E View

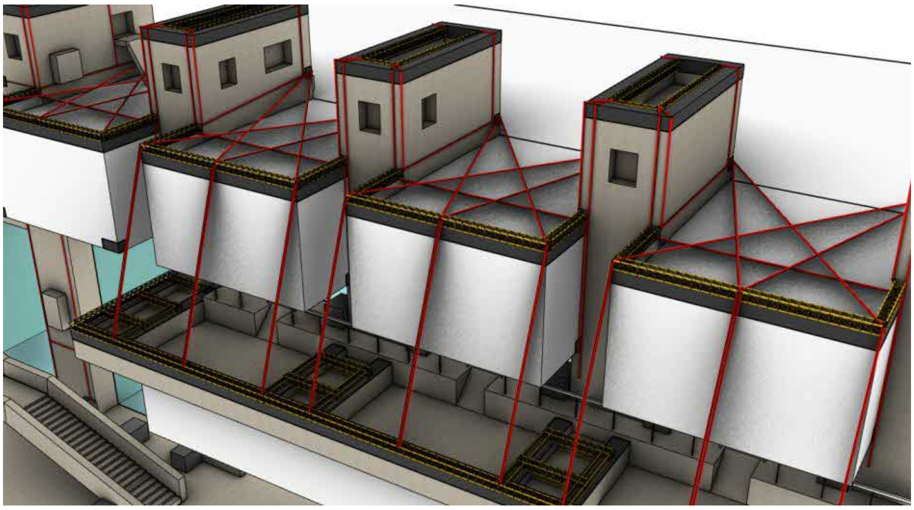


Figure 8: Close-up showing the horizontal cross-bracing at roof level

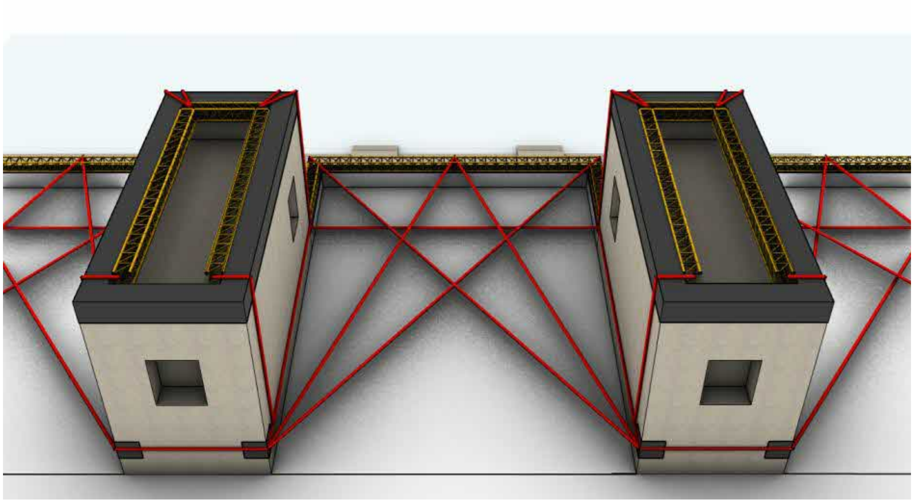


Figure 9: Close-up showing hor. wires wrapping up the towers

In addition to the above, to ensure a suitable structural behaviour in normal service situation, the steel strands will be pre-tensioned to provide sufficient stiffness to the system hence limiting displacements below the tolerances limits.

Figure 10: Front view of the completed artwork

Regarding the structure located on the terrace level, some further adjustments have been made to the structure and fabric to account for public access and mean of escape.

Emergency escapes will be kept clear of obstructions. Kentledges have been relocated.

The bottom of the fabric will be set to ensure a minimum 3.0m clear height with the external staircase.

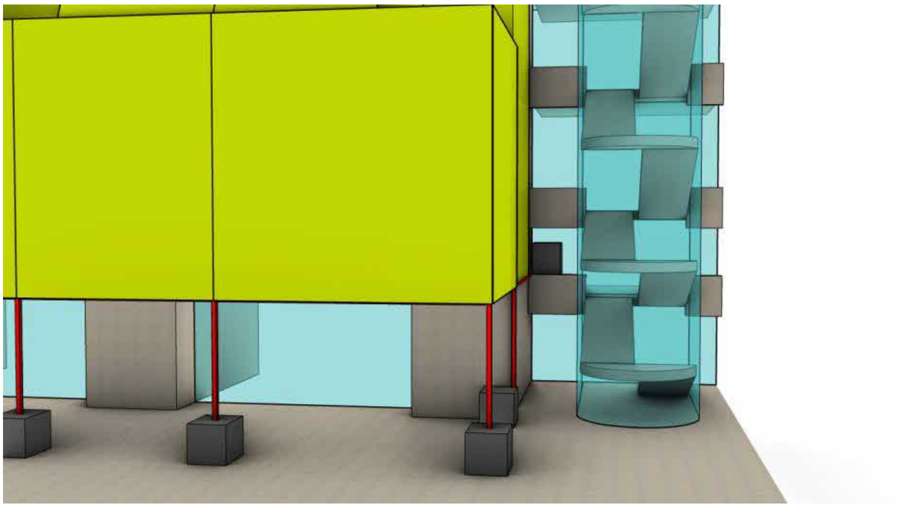


Figure 11: Ground floor East emergency exit. Kentledges moved left and to 1st floor balcony to prevent obstruction

Figure 12: Ground floor West emergency exit. Kentledge to the moved away for the façade to free emergency access route.

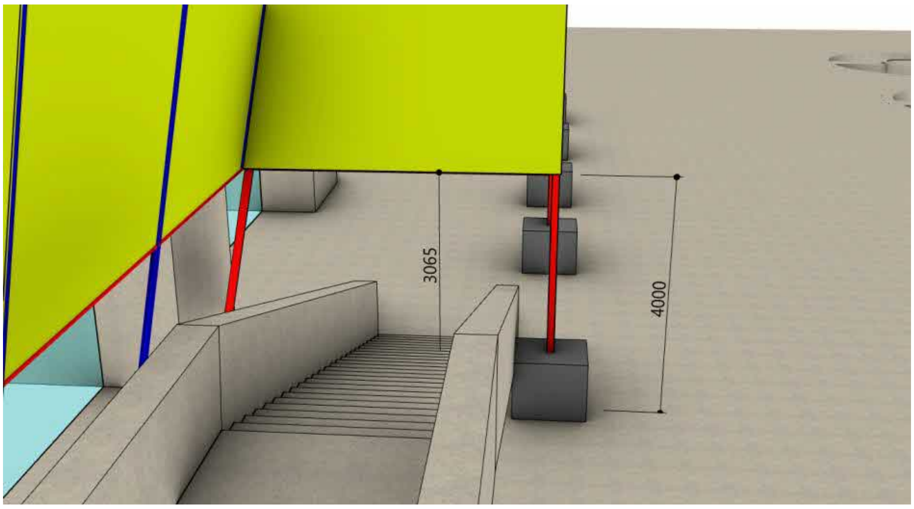


Figure 13: Minimum clear height over the external staircase

Our proposed construction sequence is as below. More details are available in our Construction Methodology Proposal in Appendix

1. Application of surface and edge protection

2. Installation of the kentledges

3. Installation of the strands from roof to terrace (top to bottom). Staged, iterative pre-tensioning from top to bottom to ensure correct positioning.

4. Installation of the supporting frame including SHS and CHS steel sections.

5. Installation of the fabric from top to bottom, starting with the towers, then the roof to balcony section and finishing with the balcony to terrace sections.

4 Management Plan

4.1 High Wind Management

A management plan for dealing with extreme wind conditions will be put into place. This will be monitored on a weekly basis with weather conditions as stated by the Met Office . It aims to identify operating limits and defines the actions to be taken at certain weather conditions. Safety during high winds needs to be considered, not just during the event, but also during the erection and dismantling phases.

Both fabric and supporting structure (steel frame, 4-chords trusses, strands) are designed to withstand a

Our proposal assumes that a regular review of wind speed forecast. Site management should monitor wind speed both predicted (through weather forecasts) and actual (via site measurement), if necessary. Our proposed Wind Management Plan is as below:

Action level	Gust Speed [m/s]	Gust Speed [mph]	Action
Green	< 11	< 25	Review of wind speed forecast weekly
Amber	> 11	> 25	Site management on alert Continuous on-site wind speed monitoring Contractor on stand-by, ready to intervene
Red 1)	> 19	> 43	All non-fixed panels to be retracted. If not done before red level is reached, Lakeside terrace to be temporary closed to public until all panels are retracted.
Black	> 25	> 60	Lakeside terrace closed to public

1): or the highest speed the Specialist Contractor can safely take down the structure.

Figure 14: Retractable (orange) vs fixed (green) fabric panels

5 Structural Calculations

5.1 Stage 1

The fixed panels are positioned flush against the existing façade, resulting in most of the wind loading being directly transmitted to the existing structure through direct contact.

The fabric panels are assumed to be hung from the top of the tower, their self-weight being integrally transferred to the tower directly.

Therefore, the strands' primary function is to hold the fabric in place. To achieve that a pre-tensioned force of 10.0 kN is assumed for all strands.

Design verified by inspection with Ø12mm galvanised steel strand (g_z = 1770 N Q) b

Minimum Breaking Load = 128 kN

5.2 Stage 2

5.2.1 Front Raking Strand

Strand design based on most exposed panel, i.e. front retractable panel 2B.

Loading summary

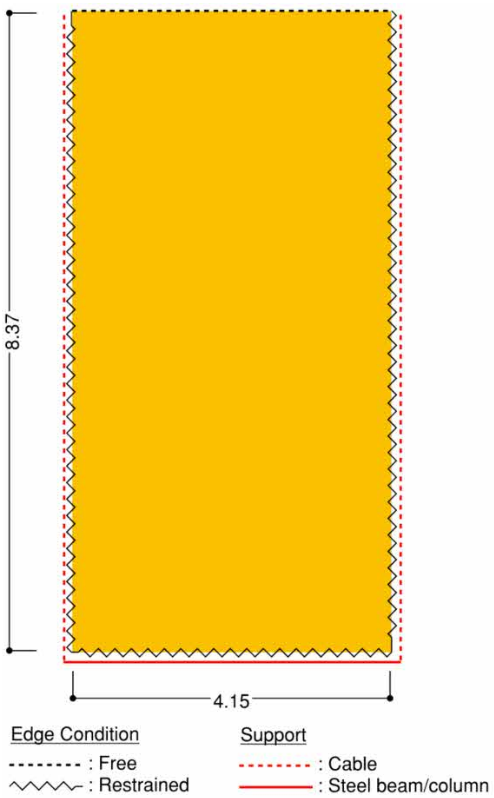
Pre-tension in strand to ensure initial straightness: $G_0 = 10.0\ l\ \ O$
Dead load (fabric + mesh): $_k = 0.11 * \frac{4.15}{2} = 0.23\ l\ \ \emptyset n$
Wind load (25 m/s): $r_k = 0.703 * \frac{4.15}{2} = 1.46\ l\ \ \emptyset n$

$$N\ \ b\,uxf\ \ o\ t\ \ i\,i\,dp\,b\ \ o\!:\ \boldsymbol{O}_E\ f_e = \mathbf{6\ \ 87\ l\ \ O}$$

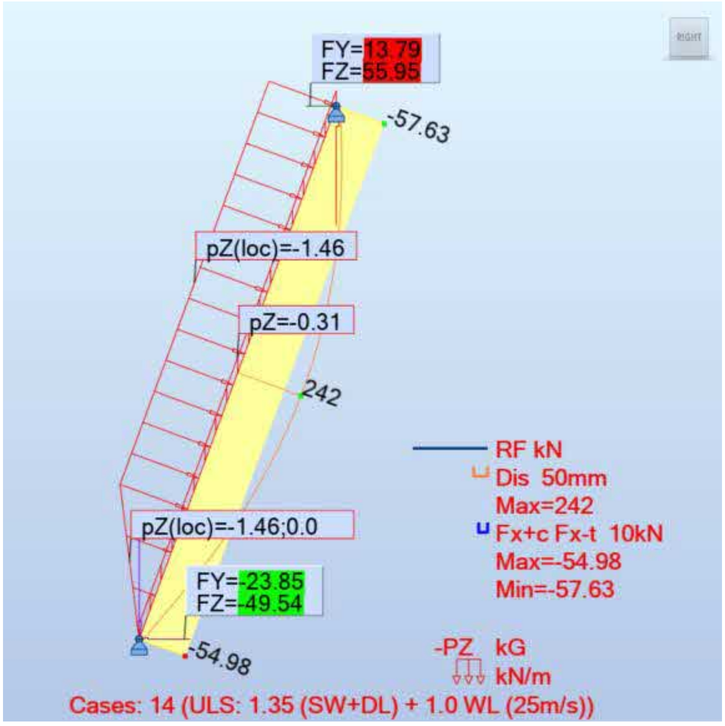
Design verified with Ø12mm galvanised steel strand (g_z = 1770 N Q) b

Minimum Breaking Load = 128 kN

Static System



FEA results



5.2.2 Roof horizontal cross-bracing

Horizontal resultant at the top of front racking strand $G_z = 13.8\ l\ \ O$

Assumed pre-tension load $G_0 = 10.0\ l\ \ O$

Strand horizontal angle $\alpha = 54°$

Central pair of strands assumed to transfer their horizontal load to the corner via the edge 4-chords trusses. Therefore, the resultant in horizontal strand at the roof corner is $O_{F\ d} = \frac{2*G_y}{\sin(\alpha)} + 10 = \mathbf{5\ \ 52\ l\ \ O}$

Design verified with Ø8mm galvanised steel strand ($g_z = 1770\ N\ \ Q)\ b$

Minimum Breaking Load = 59 kN

Horizontal resultant load to each tower $G_H = O_{F\ d} * \sin(\alpha) * 2 = \mathbf{8\ \ 25\ l\ \ O}$

Verified by inspection

5.3 Stage 3

5.3.1 Balcony Front

5.3.1.1 Steel Post

Vertical steel posts (SHS) @5.30m c/c either side of retractable fabric panels. Bottom of panel restrained by tubular section connected to the post. Most loaded post at the interface between panel 3D and 3E

Loading summary

Dead load (fabric + mesh): the panels are hung to the balcony. None transfers to the bottom pipe or posts.

ULS Wind load (25 m/s): $r_k = r_{p\ ULS} * \frac{6.85+5.30}{2} = 0.703 * 6.08 = 4.27\ l\ \emptyset n$

$R_k = r_{p\ ULS} * (\frac{6.85^2}{8} + \frac{5.30^2}{8}) = 0.703 * 9.38 = 6.59\ l\ O$

SLS Wind load (4.2 m/s): $r_k = r_{p\ SLS} * \frac{6.85+5.30}{2} = 0.02 * 6.08 = 0.121\ l\ \emptyset n$

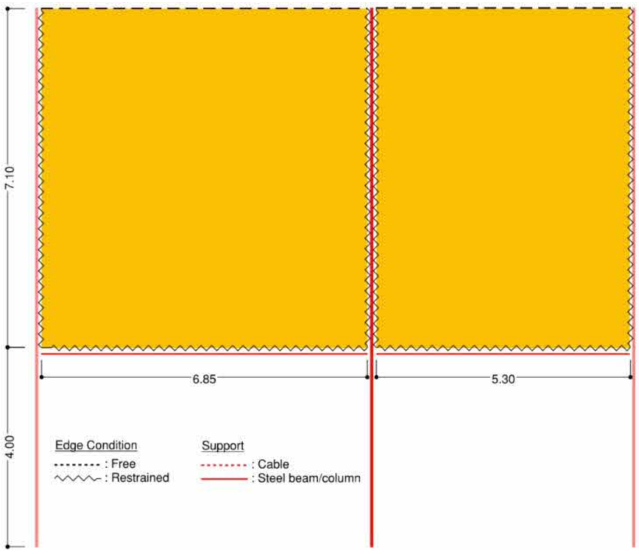
$R_k = r_{p\ SLS} * (\frac{6.85^2}{8} + \frac{5.30^2}{8}) = 0.02 * 9.38 = 0.188\ l\ O$

N bcxf o e ii qop :tN_{Ee} = 5 .04 l O n

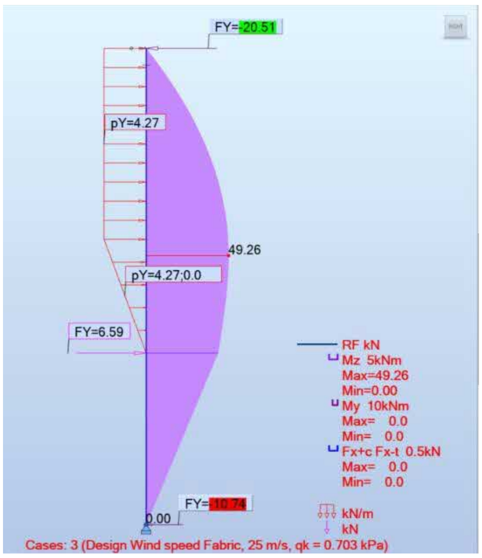
F x q f ds bu cfp eg efi t q m b idl of fns f dio pb efi :uwi$_{f\ p}$ ≈ 2 1 n n

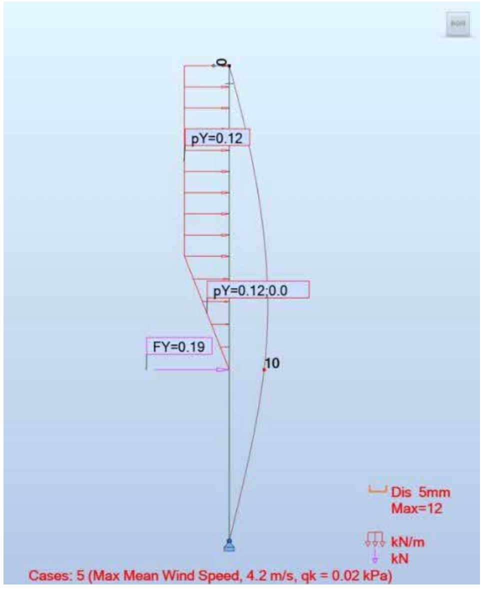
Design verified with 120x120x8.SHS (S355), $N_{S\ e}$ = 6 29 l O n

Static System



FEA results





5.3.1.2 Bottom tubular section

Loading summary

Dead load (fabric + mesh)- only in retracted position: $_{k} = 0.11 * 7.10 = 0.78\ l\ \emptyset n$

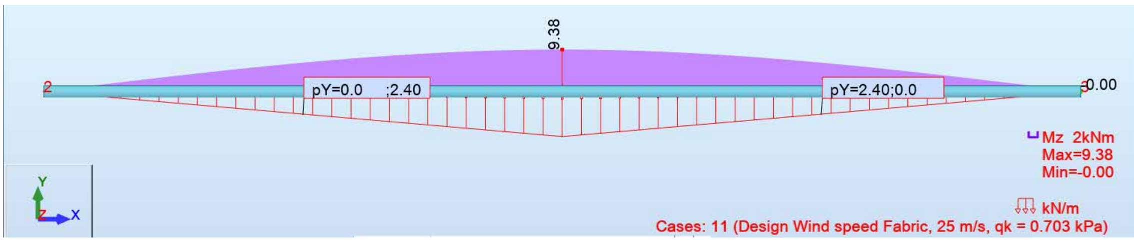
ULS Wind load (25 m/s): $r_{k,m\ b} = r_{p\ U\ L} * \frac{6.85}{2} = 0.703 * 3.43 = 2.40\ l\ \emptyset n$

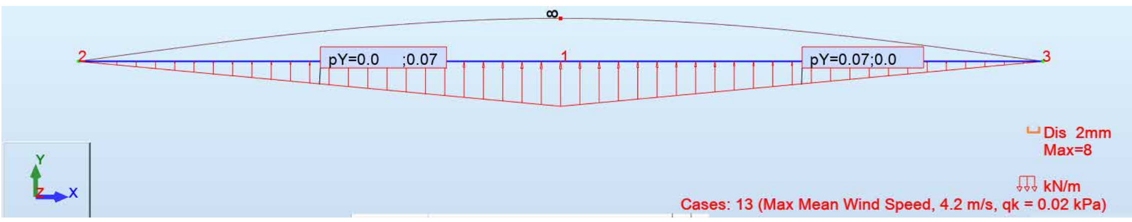
SLS Wind load (4.2 m/s): $r_{k,m\ b} = r_{p\ S\ L} * \frac{6.85}{2} = 0.02 * 3.43 = 0.07\ l\ \emptyset n$

$N\ b c x f\ o e\ i i q o p :t N_{E\ e} = 0.4\ 9\ O\ n$

$F\ x\ q\ f\ d\ b\ f p\ e g f i\ t\ q\ m f\ b o i d u f f\ s\ v d i p\ b\ e f i\ :u w_{i\ f\ S} \approx 9\ n t\ n$

Design verified with 76.1x6.CHS (S355), $N_{S\ e} = 2\ .16\ l\ O\ n$





5.3.2 Strand on westernmost tower

Strand only on the western corner of the westernmost tower. SHS section at the other side up to the 3rd floor balcony.

Loading summary

Pre-tension in strand to ensure initial straightness: $G_0 = 10.0\, l \quad O$

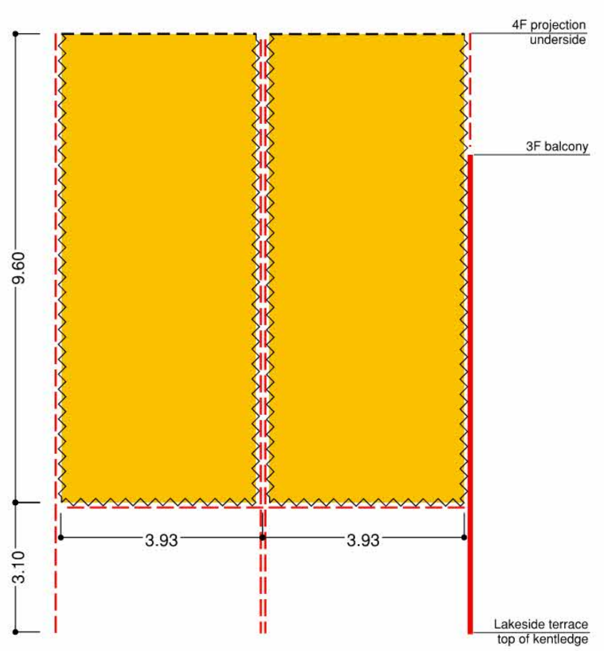
Dead load (fabric + mesh): $_k = 0.11 * \frac{3.93}{2} = 0.22\, l \quad Øn$

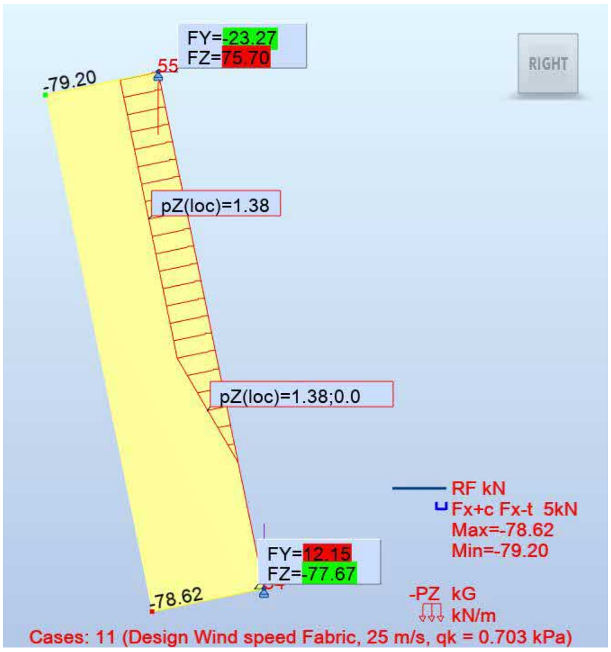
Wind load (25 m/s): $r_k = r_p \; ⁄U \; L * S \frac{3.93}{2} = 1.38\, l \quad Øn$

$$N \; b\,uxf \; o\,t \; i\,i\,d\,p\,b \; c: \; \mathbf{O}_E\, f_e = \mathbf{8} \; .\mathbf{03}\, \boldsymbol{l} \quad \boldsymbol{O}$$

Design verified with Ø12mm galvanised steel strand (g_z = 1770 $N \quad Q$) b

Minimum Breaking Load = 128 kN





5.3.3 Kentledge

5.3.3.1 Ground Floor Typical

Kentledge will serve as a counterweight to maintain the artwork in place, relying on friction between the kentledge and the terrace finish to transfer lateral resultant loads. To enhance friction, rubber pads will be added, and no fixings will be utilized.

The wind calculations above include a force coefficient accounting for dynamic amplification (gust), assuming a short-term (3s) impulse on a limited area, as per EC-1-3 [3]. Notably, this coefficient is independent of the tributary area and is considered reasonable and conservative for designing the strands and steel frame directly supporting the fabric.

However, the same assumption for the design of the kentledge on the terrace level is deemed overly conservative for the following reasons:

- The amplification factor is set for permanent structure with design life of typically 50 years.
- The catchment area of each steel post being much larger than 10.0 m^2 (37.6 m^2), making it unlikely that dynamic pressure will uniformly apply across the entire area.
- A sizeable portion of the dynamic pressure will dissipate through the stretch of the fabric and its connection to the structural frame.

In light of these considerations, it is recommended to base the design of the kentledge on static wind pressure only, omitting the amplification factor.

Therefore, the verification of the kentledge is as follow.

The proposed lateral force to be considered is $G_{F\ d} = \frac{G\ Y}{1.80} = \frac{10.7}{1.80} = \mathbf{6.0}\ 6\ O$

Friction coefficient - Wet concrete to rubber

Static $\boldsymbol{\mu_t} = \mathbf{1.8}\ 1(0.55 – 0.85)$

Kinetic $\mu_k = 0.60\ (0.45 – 0.75)$

Weight of the kentledge, incl. partial safety γ_g $H_k = \frac{G_{E\ d}}{\mu_s \gamma_f} = \frac{595\ k}{0.7*0.9} = 945\ l \equiv \mathbf{2.1}\ u\ f\ o\ u\ m@f6.4\ g/d$

5.3.3.2 Westernmost tower

Unlike typical cases, the design of the kentledge is influenced by the strand's inclination, which is dependent on the wind direction.

Positive pressure (critical case)

The pre-tension in the strand inherently prevents any displacements of the kentledge toward the façade, creating a meta-stable system. To enhance the secure positioning of the kentledge, we recommend the addition of soft rubber packing against the façade to counteract potential uplift. For the central kentledge, we propose a ground beam connection to the others, serving both as anchorage and redundancy in case of strand tension, thus extending the self-weight of the kentledge.

Negative pressure (suction)

Displacements of the kentledge away from the façade will naturally release tension in the strand, unloading the kentledge. This underscores the need to firmly secure the top end in position, ensuring the structural system remains functional even if the strand is slightly unloaded.

Appendix A - Proposed Construction Methodology

Appendix B – Fabric Model

Maxime Chollet
Buro Happold Limited
17 Newman Street
London
W1T 1PD
UK

T: +44 (0)207 927 9700
F: +44 (0)870 787 4145
Email: Click here to enter text.