WIND LOADING CALCULATIONS

SITE CONDITIONS

- Roof slope of 17.5°, likely even for load

Height of podium:
- $= 32.210 - 31.310$
- $= 900\text{mm}$

Height of hard rain:
- $= 31.910 - 31.310$
- $= 600\text{mm}$

Notes about Calculations:

- Reduced levels by Hundreds of the podium have been measured from scale architectural drawings of the roof.

- Calculations assume the hard rains disrupt wind flow enough to be considered a solid "top" and disrupt wind flow. In reality, they act more like the wind instead of stopping it.

- Height of each hard rains and the podium are taken with respect to reduced level in the center of the roof.

ROOF PLAN NOT TO SCALE
Determining Site Wind Load: $\text{V}_{\text{des},0}$

- Taking height of roof as 7.9m (Plan detailed plans)

\[
\text{V}_{\text{des},0} = \text{V}_{\text{R}} \times \text{M}_\text{d} \times \text{M}_\text{cat} \times \text{M}_\text{t} \times \text{M}_\text{t}
\]

\[
\text{V}_{\text{R}} = \text{Site wind speed for VR (M/S)}
\]

- $\text{M}_\text{d}$: directional multiplier
- $\text{M}_\text{cat}$: category multiplier
- $\text{M}_\text{t}$: topographical multiplier

- Christchurch is in wind region A7

- From Table 3.7, NT51170.0
  - the building has an impingement level of 3
  - it is expected that the design wind load of the building in its current state in 50 years

Taking these into account, the annual probability of exceedance ($V_R$)

- Thus using Table 3.1 for NT51170.2, $V_R = 46$ m/s

\[
\text{M}_\text{d} = \text{Cardinal direction of NE, Region A7, } M_d = 0.90 \quad \text{(Table 3.2) NT51170.2}
\]

\[
\text{M}_\text{cat} = \text{Category & building height of 8m, } M_{\text{cat}} = 0.83 \quad \text{(Table 4.1A NT51170.2)}
\]

\[
\text{M}_\text{t} = \text{Assume little shielding (conservative) } M_t = 1.0
\]

\[
\text{M}_t = \text{Use } M_t = 10 \text{ at terrain II that are not far above sea level, } M_t = 1.0
\]

- $V_{\text{des},0} = \text{V}_{\text{R}} \times \text{M}_\text{d} \times \text{M}_\text{cat} \times \text{M}_\text{t} \times \text{M}_\text{t}$

\[
\text{V}_{\text{des},0} = 46 \times 0.90 \times 0.83 \times 1.0 \times 1.0
\]

\[
\text{V}_{\text{des},0} = 34 \text{ m/s}
\]

$V_{\text{des},0} = 34$ m/s
DESIGN WIND FORCES PRESENT ON ROOF SURFACE

- For these calculations, it is assumed that the parapet end railing is sufficient to block the wind load on the roof, as well as provide sufficient clearance to prevent the likely drop of small debris (e.g., leaves) on the roofing.

- Therefore, these calculations are to determine the uplift force on the roof.

Design wind pressure:

\[ p = (0.8 \text{ sec}) [V_{10}] C_f (c_{dyn}) \]

For wind buildings, \( C_f = C_{p,e} K_a K_e K_r K_p \)

- \( C_{p,e} \): roof height, \( z = 7.9 \text{ m} \) (h)
- \( z \text{ above top eave, } z = 3.12 \text{ m} \) (d)
- with \( h/d = 0.554 \), Table 5.7(A), NBS 1170-2

\[ C_{p,e} = (0.9) \cdot 0.4 \]

\( C_f \) use larger value.

- \( K_a \): area reduction factor - approx. area of roof ≈ 14 × 11.5

\[ K_a = 0.80 \quad (\text{tab} \# 5.4 \text{ NBS 1170-2}) \]

- \( K_e \): wind action factor \( K_e = 1.0 \) (close 5.4.2 NBS 1170-2)

- \( K_r \): not assessing cladding \( K_r = 1.0 \)

- \( K_p \): reduction due to parapet height - using stainless steel handrail

\[ h_p = 0.6 \text{ m}, \quad h = 7.9 \text{ m} \quad \Rightarrow \quad \frac{h_p}{h} = 0.076 \]

Interpolated from Table 5.1, NBS 1170-2

\[ K_r = 0.96 \]
Project: GREENWOOD PROJECT
Subject: WIND LOADING CALCULATIONS
By: N. Clarke L. Colinas
Date: __________________ Page 4 of 4

\[ C_{ty} = \frac{C_{px} K_a K_c K_i K_p}{0.90 \times 0.8 \times 1.0 \times 0.96 \times 1.0} \]
\[ = -0.691 \]

\[ p = 0.50 \times \left[ \frac{\text{Volts, o}^2}{C_{ty}} \right] C_{ty} \]
\[ = 0.5 \times 12 \times 34.4^2 \times 0.691 \]
\[ = -4.1 \text{ Pa} \]

Uplift force \( \text{not} \) at ULS and \text{not safe for} \text{design.}

As \( \text{Uplift} \text{ Pa}, \text{uplift} \text{ is less than the weight of the} \text{load} \text{.}

\( \text{Uplift} \text{ will not occur and the} \text{site is safe for the load.} \)
LOAD COMBINATIONS CALCULATIONS

- Combinations:
  - 1.35G + 0.9G/\text{Wu}
  - 1.2G, S\text{w}/\text{UQ}
  - 1.2G, \text{Wu} + \text{UQ}

- Assumptions:
  - Earthquake loads excluded in seismic analysis
  - Assumed that slim governed by gravity
  - Live load, \( q = 0.25 \text{ kN/m} \)
  - Maintenance load, \( (A)^{(S/25)} \text{ kN/m} \)

  - Non-structural topping has been estimated as \( 0.5 \text{ m} \) from structural plans
  - Concrete density is \( 2400 \text{ kg/m}^3 \)

Dead Load, \( G \):

\[
G = \text{structural gravity of topping} + \text{non-structural topping}
\]

\[
= 2.1 \text{ kN/m} + \left( 0.25 \times 2400 \text{ kN/m} \right) / 1000
\]

\[
= 2.1 \text{ kN/m} + 0.6 \text{ kN/m}
\]

\[
G = 2.7 \text{ kN/m}
\]

Live Load, \( q \):

\[
q = 0.25 \text{ kN/m} \quad \text{from AS/25} \text{ 11.70.1 section} \]

Wind Load, \( W \):

- Uplift case (considered only), \( W_w = -0.49 \text{ kPa} \)

Snow Load, \( S \):

- Snow load is given only

\[
S = S_0 \cdot C_{e,\text{w}} \cdot C_{e,\text{snow}}
\]

where:
- \( S_0 \) = characteristic snow load (kPa)
- \( C_{e,\text{w}} \) = expansion coefficient
Characteristics: Snow load;

\[ S_g = K_p 1.2 \left( \frac{P_{10}}{1000} + 0.3 \right) \text{ (N\,m\,s^2)} \]

Christchurch - sub-alpine area, NA region.

- \( K_p \) - probability factor - design for a return period of 1000 years
  - \( K_p = 1.65 \)

- \( h_s \) - taking elevation of site as 22m above sea level (from internet)

\[ S_g = 1.65 \times 1.2 \left( \frac{3 \times 22}{1000} + 0.3 \right) \]

\[ S_g = 0.13 \text{kPa} \]

Clause 4.2.1, NZ 11703 - if \( S_g < 0.18 \text{kPa} \) for sub-alpine area, we \( S = 0.18 \text{kPa} \)

\[ S = 0.18 \text{kPa} \]

Calculating load combinations:

- \( 1.2C_g = 1.35 \times 3.3 = 4.5 \text{kPa} \)
- \( 1.2C_g, 1.5Q = 1.2 \times 3.3 + 1.5 \times 0.15 = 4.3 \text{kPa} \)
- \( 1.2C_g, W_N, Q_f = 1.2 \times 3.3 + 0.4 = 3.47 \text{kPa} \) (Factored load 0.6)

- \( 0.9C_g, W_N = 0.9 \times 3.3 - 0.4 = 7.4 \text{kPa} \)

- \( 1.2C_g, S = 1.35 \times 3.3 + 0.4 + 0 = 4.36 \text{kPa} \)

Load case: 1.35C governs.
RIG DESIGN CALCULATIONS

PROPOSED LAYOUT

SIDE ELEVATION

PLYWOOD BASE

WOOD-FILLED RUBBER (EPDM) ROOF COVER

ROOF MEMBRANE

SECTION A-A

DIMENSIONS:

1.7m

2.1m

3.7m

Timber

UPPERFRAME (MORE TIMBER)

UNIVERSITY OF CANTERBURY

CIVIL ENGINEERING

Department

By: N. CLAYDON L. EDWARDS

Date: ___________ Page 1 of ______
Project: Rig Design Calculations
Subject: Green Roof Project
By: N. Clarke, L. Edwards
Date: _______________ Page 2 of ____________

PLAN (Without Trays)

13 PLASTIC TRAYS

WOOD LIPS, LATERAL RESISTIVE FOR TRAYS

Calculation will be made for the following components:

1/ Crushing load for plywood protective board (timber under frame)

2/ Plywood base calculations

3/ Connection calculations


1. PLYWOOD PROTECTION LAYER AND TIMBER UNDERFRAME CHECKS

- Plywood protective layer:
  - Will use F8 grade ply which is rated as having
    a characteristic shear of $W = 70 \text{ kPa}$ in $N/m^2$.
    
    
- Maximum load of $21 \text{ kPa} < 70 \text{ kPa}$, so
  plywood will not take load.

- Timber underframe:
  - Members in the underframe are restrained laterally
    along the compression edge.
  - In the plane of compression, members are very short
    and square. It has been assumed that this will not
    occur and it is applied load.
  - Bearing is also not a factor as the entire compression
    edge of the member is engaged by the load.
  - For MSAS' factor, it is rated as $f_0 = 8.4 \text{ kPa}$
    for compression perpendicular to the grain
    
    $f_0 = 21 \text{ kPa} < 8.4 \text{ kPa}$ OK.

2. PLYWOOD BASE CALCULATIONS

- Checking the amount of underframe required:
  
  a) with no extra underlining:

Note that during calculations assume no double area
1) $3.6 \text{ m} \times 4 \text{ m}$

This is conservative as each
area actually only loads on areas
$1.8 \text{ m} \times 1.5 \text{ m}$ instead of
the $2 \text{ m} \times 2 \text{ m}$ intended.
- Loading on slab can be determined using the Hillerborg Strip Method.

- Strips in x-direction:
  - Max \( M_y \) when UDL across strip.
  - Calculated using:
    - \( M_{x,\text{max}} = \frac{W y^2}{2} \)

  - Example:
    - \( W = 2.1 \text{ kN/m} \)
    - \( y = 1.5 \text{ m} \)
      - \( M_{x,\text{max}} = 7.1 \times 1.5^2 \)
      - \( M_{x,\text{max}} = 3.4 \text{ kN/m} \) per width

- Strips in y-direction:
  - Max \( M_y \) when \( x = 1.8 \) (discontinuity line shows point UDL does not move past this point).
  - Calculated using:
    - \( M_{y,\text{max}} = 3.4 \text{ kN/m} \) per width
Capacity calculations

- will try grade C 0 plywood, dress grade F8 (t = 25 mm)
- As 3.5 kN/m² is a large moment required for plywood, will use 2.8 kN/m² for plywood
- will assess by both flexure and shear

Flexure: $M_r < 0.6 M_n$

Where $M_n = K_i K_y K_8 K_9 F_6 F_7$

$K_i$ - duration of load factor - 0.6 (prop)
$K_y$ - parallel support factor - 1.0
$K_8$ - Stability factor
$K_9$ - sample orientation factor - 0.95

$M_n = 25 M_p (\text{Table 11.7, T06})$
$F_6 = 3.14 \times 10^{-2} \text{ per 1200 mm} \text{ width (Table 12.3, T06)}$

$M_n = 0.6 \times 1.04 \times 10 \times 10 \times 0.95 \times 25 \times 3.14 \times 10^{-3}$

$M_n = 0.45 \text{ kN.m} < 3.4 \text{ kN.m}$

(undertaken beam is required to reduce the design moment, $M_r$)
 Reinforce along the ARMS of each bay.

6 equivalent min. slabs (connect "slabs" feature from west "bed" at max load of \( \frac{wL^2}{8} \) as opposed to \( \frac{wL^2}{6} \) at middle "slab")

Strips in \( y \):

Maximum tens at fixed ends:

Strips in \( x \):

\[
M_{\text{max}} = \frac{wL^2}{8} = \frac{2.1 \times 12^2}{8} = 0.378 \text{ kNm}
\]
As \( M_n = 0.45 \text{ KNm} \) (calculated earlier).

\[
\frac{0.45 \text{ KNm}}{5 \text{ m}} > M = 0.0378 \text{ KNm/m}
\]

Checking for shear:

\[
V_{\text{max}} = 5 \times \frac{1}{8} \quad \text{(at fixed end)}
\]

\[
V_{\text{max}} = \frac{5 \times 2.1 \times 1.2}{8} = 1.58 \text{ KN/m width}
\]

Rolling shear: \( V < DNv \) (eqn 6.4.1.2, NZS 3603)

\[
\frac{\frac{1}{2} Nv K_n K_{ff} b}{t} \quad \text{(1/2)}
\]

where \( K_n = 0.6 \) - duration of load factor

\( K_{ff} = \text{multilateral factor} \rightarrow 10 \) - cosine

\( K_f = 0.95 \) (controlled partially)

\( b = 1 \text{ m} \) (width of panel x anchoring)

\( f = 11.8 \text{ KN/m}^2 \)

\( f_{vm} = 1.7 \text{ KN/m}^2 \)

\[
0.4 \times 0.6 \times 10 \times 1.0 \times 1.000 \times 14.8 \times 1.2 = 1.58 \text{ KN} < 10.83 \text{ KN/m width} \quad \text{ok}
\]

Checking bearing strength:

\[
N_n = 1 \text{ nd provided by concrete} = 4.3 \text{ KN}
\]
\[ N_b^2 \leq 2N_{sh} \]

where:

\[ N_{sh} = K_s K_{ty} K_{pp} A_p \]  

(Section 6.4.13 MS3603)

- \( K_s = 0.6 \)
- \( K_{ty} \) - bearing area factor \( \approx 1.0 \)
- \( K_{pp} = 1.10 \) assume m/c 15\%
- \( A_p = 0.6 \text{m}^2 \) (Table 6.1 MS3603)

- \( A_p \) in using an appropriately sized and properly treated timber substructure with a 50\% volume of the required cross section.

\[ A_p = \frac{1200 \times 50}{7250 \times 50} = \frac{62500}{362500} = 0.17 \text{m}^2 \]

\[ \phi N_{sh} = 0.9 \times 1.10 \times 10 \times 10 \times 0.6 \times 152.5 \times 10^3 \]

\[ \phi N_{sh} = 1757 \text{KN} \]

\[ \phi N_{sh} > 4.3 \text{KN} \]
**Connection Calculations - for Lateral Loading**

To connect the timber trim to the plywood backing, four gauge 8 (4.19 mm) screws per piece are used. Each piece has a span of 600 mm.

![Diagram of connection](Attachment)

**Earthquake Loading**

To greatly simplify calculations, it has been assumed that:

- The building is not connected rigidly to the foundation, and it is soft.
- The response is elastic and the system can be modeled as a SDOF system.
- The period of the building is very small (≈ 0.5 s).

From NZS 1170.5, for buildings, $V_{neq} = C_d T_{neq} W_T$ where:

- $C_d$ = horizontal drift coefficient
- $T_{neq}$ = natural period of structure
- $W_T$ = total weight of structure

As $C_d(0.5) < 1$, this equation is valid. The lateral drift of a structure is transferred to a lateral load in the event of an earthquake.

For a very rough estimate of an ES (elastic base), 50% of the total system weight is assumed to be lateral load.

This has been chosen in the following three ways:

1. The horizontal drift given for a 50% transfer, in Christchurch 12 $C_d = 0.52$ for short period structures. However, it's very small scale in comparison to a building's period.
2. Likely to be rejected by Title 6.
Total System Weight = Load from + weight from brackets

= (0.1 x 117 x 186) x 6 + 2 KN (concrete ultimate)

= 7.6 KN

and estimate, 50% transferred to lateral loading;

Vertical = 7.6 KN

As this load is resisted by a minimum of 6 timber bars in each direction;

S^x = 2.3 KN

Capacity Provided

S > \phi_D n (will carry the entire span at the beam end direction)

where \phi_D = 0.7 x 0.8 K

= 0.7 x 4 x 125 x 0.8 x 10

= 2.75 KN

\phi_D n = 2.75 KN

\phi = 0.7 for hanger

K = 0.6 for green cond. tbl 4.8

K_1 = 0.5 (hanger)

crossed

as \phi_D n > S^x to OK.

The screws also need to be positioned in accordance with minimum edge distances as specified in figure 4.7 of NAS 3603 for touch-y parallel to the gusset.
<table>
<thead>
<tr>
<th>From Inner Edge (mm)</th>
<th>Between Screws (mm)</th>
<th>To Outer Edge (mm)</th>
<th>To End (mm)</th>
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<tbody>
<tr>
<td>14</td>
<td>18</td>
<td>7</td>
<td>18</td>
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