BMT Fluid Mechanics

# 130 William Street
# New York, NY, USA

**Wind Tunnel Testing**

**Overall Wind Loading Study**

**December 21st 2016**

**For**
**The Lightstone Group**

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| **Report Title** | **130 William Street<br>Overall Wind Loading Study** | |
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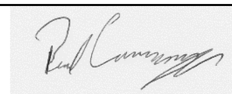
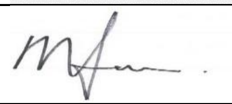
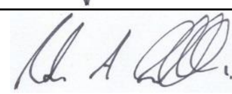
| **Author** | **Dr. R. Cummings** |  | **12/21/2016** |
| **Approver** | **Mr. M. Lee** |  | **12/21/2016** |
| **Authoriser** | **Dr. R. Stanfield** |  | **12/21/2016** |

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# 130 William Street
# New York, NY, USA
# Overall Wind Loading Study

## Contents

# EXECUTIVE SUMMARY

## Background

This report summarizes the results of a program of boundary layer wind tunnel studies conducted by BMT Fluid Mechanics Limited to assess the wind effects relevant to the structural and serviceability design of the proposed at 130 William Street located in New York, NY, USA.

A **1:400** scale model of the proposed development was constructed for the purpose of conducting wind tunnel studies to determine the overall wind loads acting on the structure.

The study defines design wind loads for the proposed development based on design wind speeds derived from the 50-year return period wind speed of **98mph** (referenced to 3-second gust averaging period at 33ft height over Exposure C terrain) as stipulated in the New York City Building Code. Furthermore, the design wind speeds have been used in conjunction with the directionality information derived for the extreme wind events within the region.

Fluctuating base wind loads were measured on the proposed development for a full range of wind directions. Dynamic analysis, using structural and dynamic properties provided by *McNamara Salvia* on *December 9th and 16th, 2016* was conducted.

A set of peak dynamic floor-by-floor loads was estimated and the percentage of load components are calculated in order to account for the correlation between of peak base wind loads.

For serviceability design, namely wind induced building accelerations, the building responses have been assessed based on the 1-year and 10-year return period design wind speeds in conjunction with the use of the directionality information derived for the extreme wind events in the region.

# Principal Results and Conclusions

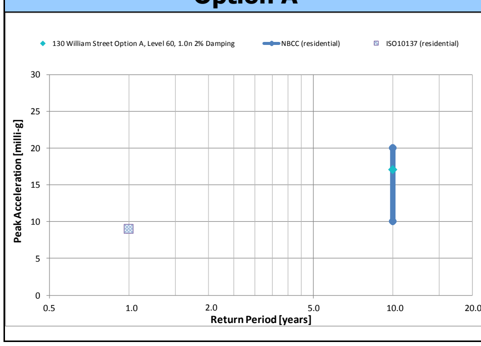
The wind loads given in this report are design loads as per the requirements of the New York Building Code. These ultimate wind loads should be used in conjunction with the appropriate load factors given in the New York Building Code[1].

Peak *magnitude* instantaneous dynamic base loads for the proposed development has been estimated in conjunction with a structural damping level of 2% of critical:

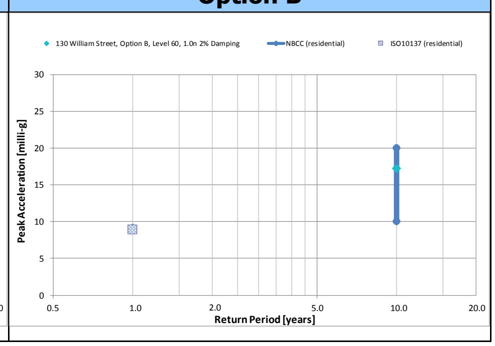
| Base Loads | Option A | Option B |
|---|---|---|
| **Fx [$10^3$ kip]** | 1.6 | 1.6 |
| **Fy [$10^3$ kip]** | 2.5 | 2.5 |
| **Mx [$10^3$ kip-ft]** | 1301.1 | 1297.4 |
| **My [$10^3$ kip-ft]** | 832.7 | 829.9 |
| **Mz [$10^3$ kip-ft]** | 29.3 | 29.0 |

The peak-combined wind-induced accelerations at the "Level 60" of both Option A and Option B of the proposed development, for the 1- and 10-year return periods, structural damping of 2% of critical are shown.

**Option A**




**Option B**




The wind-induced accelerations on Level 60 for structural damping of 2.0% of critical for both building design options satisfy the 10-year return period criteria stipulated by the NBCC for residential buildings. Furthermore, the wind-induced peak accelerations marginally satisfy the frequency dependent occupant comfort criteria (such as ISO 10137-2007) for the 1-year return period for structural damping of 2.0% of critical.

# 130 William Street
# New York, NY, USA
# Overall Wind Loading Study

## 1. Introduction

### 1.1. Background

This report summarizes the results of a program of boundary layer wind tunnel studies conducted by BMT Fluid Mechanics Limited (BMT) and commissioned by Renaissance Construction and Development to assess the wind effects relevant to the structural and serviceability design of the proposed at 130 William Street located in New York, NY, USA.

### 1.2. Site / Building Details

#### 1.2.1. Location / Surrounding Area

The proposed development site is located in New York, USA. The site is bounded by William Street to the northwest, Fulton Street to the northeast, and John Street to the southwest.

At present the area immediately surrounding the proposed development principally comprises condense high-rise buildings. Further afield to the East and South-East is the East River.

An aerial view of the proposed site is shown in Figure 1.1.

#### 1.2.2. Proposed Development

The total height of the proposed development is approximately 770ft measured from the local ground level comprising of a 690ft tower of generally rectangular plan atop an 80ft high podium.

A plan view of the proposed development is shown in Figure 1.2.

# 2. Methodology

## 2.1. Technical Standards

The technical standards pertaining to the execution of the relevant boundary layer wind tunnel tests are fully compliant with the guidelines of the American Society of Civil Engineers (ASCE) Manual of Practice No.67 for Wind Tunnel Studies[1].

## 2.2. Details of Study

### 2.2.1. Design Wind Speeds

Details of the wind analysis conducted for the site to determine wind properties, including design wind speeds, are presented in Appendix A.

### 2.2.2. Wind Tunnel Model

Details of the model scale and construction, along with the wind tunnel set-up are included in Appendix B.

### 2.2.3. Measurement and Analysis

The technical details relating to the fluctuating wind pressure measurements, on which this analysis is based, are summarized in Appendix C. This gives details of the instrumentation and scaling parameters that were used in the wind tunnel tests and in the analysis of overall wind loads. Details on the analysis techniques themselves (being a modal analysis technique) can be provided upon request.

### 2.2.4. Building Properties

The calculations were based on structural/dynamic properties provided by McNamara Salvia, received on December 9$^{th}$ and 16$^{th}$, 2016 for the proposed development. A summary of these properties relevant to the wind load analysis are summarized in Appendix D.

### 2.2.5. Force/Moment Axis System

Details of the axis systems used for the presentation of wind loading results for the proposed development are given in Figures 2.1a and 2.1b.

### 2.2.6. Definition of Wind Direction

The 0° wind direction has been chosen to coincide with the geographical north (90° east, 180° south, 270° west). It should be noted that the wind angle denotes the wind direction that the wind is blowing *from* (see Figure 1.2).

# 3. Results

## 3.1. Structural Design

The study defines design wind loads for the proposed development based on design wind speeds derived from the 50-year return period wind speed of **98mph** (referenced to 3-second gust averaging period at 33ft height over Exposure C terrain) as stipulated in the New York City Building Code[2]. Furthermore, the design wind speeds have been used in conjunction with the directionality information derived for the extreme wind events within the region.

The structural damping level used in the analysis is 2.0% of critical and the post-processing of the measured wind tunnel data has been focused on all three modes of vibration of the structure, as supplied by McNamara Salvia. The base loads reported in the present study are specified at "Cellar".

It should be noted that all loads stated are **working** design loads and need to be applied in design in conjunction with the appropriate code-compliant load combination factors for wind as given in Chapter 2 of the ASCE7-05 code[3].

### 3.1.1. Base Loads

Table 3.1 provides the highest peak magnitudes of the dynamic base loads ($F_x$, $F_y$, $M_x$, $M_y$ and $M_z$) for the proposed development with a structural damping level of 2.0% of critical.

### 3.1.2. Floor-by-Floor Loads

The highest peak floor-by-floor load distributions for $F_x$, $F_y$ and $M_z$ for the 50-year return period wind speeds is presented in Tables 3.2a and 3.2b, and Figures 3.1a and 3.1b, with a structural damping level of 2.0% of critical for both Option A and B.

### 3.1.3. Combination of Loads

For the structural design of the proposed towers it is recommended that the load and the associated percentage of load components calculated in order to account for the correlation (or lack thereof) between peak wind base loads described in Tables 3.3a and 3.3b (with a structural damping level of 2.0% of critical), are considered.

Details of the methodology used to derive the load cases and load envelopes are presented in Appendix E.

## 3.2. Serviceability Design

For serviceability design, namely wind induced building accelerations, the building responses have been assessed based on the 1-year and 10-year return period design wind speeds in conjunction with the use of the directionality information derived for the extreme wind events in the region.

The level of structural damping in the analysis are 2.0% of critical (as specified by the design team) and the post-processing of the measured wind tunnel data has been focused on all three modes of vibration of the structure.

The wind-induced peak accelerations have been assessed at several locations on "Level 60" for the proposed development as illustrated in Figure 3.2.

Full details of the methodology used for deriving the buildings accelerations can be provided upon request.

### 3.2.1. Building Accelerations

The peak-combined wind-induced accelerations on "Level 60" for the proposed development for the 1- and 10-year return periods, structural damping level of 2.0% of critical, are shown in Tables 3.4 for Option A and Option B.

# 4. Discussion

## 4.1. Serviceability Design

A comparison of the peak accelerations listed in Table 3.4 can be made with Figure 4.1 illustrating several commonly used occupant comfort criterion for the assessment of wind-induced building motion.

The wind-induced accelerations on Level 60 for structural damping level of 2.0% of critical for both building design options satisfy the 10-year return period criteria stipulated by the NBCC[4] for residential buildings. Furthermore, the wind-induced peak accelerations marginally satisfy the frequency dependent occupant comfort criteria (such as ISO 10137-2007) for the 1-year return period for structural damping level of 2.0% of critical.

# 5. References

[1] American Society of Civil Engineers (ASCE) Manual of Practice No.67 for Wind Tunnel Studies

[2] New York City Building Code, 2014 NYCBC

[3] The American Society of Civil Engineers "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-05)

[4] National Building Code of Canada (NBCC). National Research Council of Canada (NRCC), Ottawa, Canada

[5] ESDU (Engineering Science Data Unit) Item 01008. Computer Program for Wind Speeds and Turbulence Properties: Flat or Hilly Sites in Terrain with Roughness. 2001

# Tables

**Table 3.1: Peak magnitude dynamic base loads, 50-year return period wind speeds, structural damping of 2% of critical**

| Base Loads | Option A | Option B |
|---|---|---|
| Fx [MN] | 1.6 | 1.6 |
| Fy [MN] | 2.5 | 2.5 |
| Mx [MNm] | 1301.1 | 1297.4 |
| My [MNm] | 832.7 | 829.9 |
| Mz [MNm] | 29.3 | 29.0 |

**Table 3.2a: Highest peak floor-by-floor loads (Fx, Fy and Mz) against height above "Cellar", 50-year return period wind speeds, structural damping of 2.0% of critical, Option A**

| Level<br>- | Height<br>[ft] | Fx<br>[Kip] | Fy<br>[Kip] | Mz<br>[Kip·ft] |
|---|---|---|---|---|
| 63TBH | 755.5 | 25.4 | -42.1 | -183.2 |
| 62EMR | 736.1 | 45.2 | -115.4 | -643.6 |
| 61RF | 706.8 | 70.2 | -121.7 | -698.2 |
| 60 | 691.3 | 70.2 | -101.9 | -1124.8 |
| 59 | 678.8 | 62.4 | -90.6 | -972.7 |
| 58 | 666.3 | 60.6 | -88.2 | -959.2 |
| 57 | 653.8 | 58.8 | -85.8 | -942.5 |
| 56 | 641.3 | 57.0 | -83.5 | -925.2 |
| 55 | 628.8 | 55.2 | -81.1 | -907.9 |
| 54 | 616.3 | 53.4 | -78.7 | -889.8 |
| 53 | 603.8 | 51.6 | -76.4 | -871.5 |
| 52 | 591.3 | 50.1 | -74.4 | -855.1 |
| 51 | 578.8 | 48.3 | -72.1 | -834.2 |
| 50 | 566.3 | 37.4 | -55.9 | -645.2 |
| 49 | 555.6 | 33.7 | -50.9 | -638.2 |
| 48 | 544.9 | 33.4 | -50.3 | -580.0 |
| 47 | 534.3 | 31.8 | -48.4 | -609.8 |
| 46 | 523.6 | 31.4 | -47.7 | -556.9 |
| 45 | 512.9 | 29.9 | -45.9 | -583.1 |
| 44 | 502.3 | 29.5 | -45.2 | -532.3 |
| 43 | 491.6 | 28.3 | -43.7 | -557.2 |
| 42 | 480.9 | 28.0 | -43.2 | -512.6 |
| 41 | 470.3 | 26.8 | -41.7 | -534.0 |
| 40 | 459.6 | 26.8 | -41.8 | -505.1 |
| 39 | 448.9 | 26.2 | -40.9 | -497.6 |
| 38 | 438.3 | 25.4 | -39.9 | -491.0 |
| 37 | 427.6 | 24.8 | -38.9 | -485.7 |
| 36 | 416.9 | 24.1 | -37.9 | -478.3 |
| 35 | 406.3 | 23.4 | -36.8 | -468.5 |
| 34 | 395.6 | 22.7 | -35.7 | -456.2 |
| 33 | 384.9 | 22.0 | -34.5 | -445.7 |
| 32 | 374.3 | 21.3 | -33.5 | -434.4 |
| 31 | 363.6 | 20.6 | -32.4 | -423.2 |
| 30 | 352.9 | 20.0 | -31.4 | -412.5 |

**Table 3.2a: Highest peak floor-by-floor loads (Fx, Fy and Mz) against height above "Cellar", 50-year return period wind speeds, structural damping of 2.0% of critical, Option A (*con't*)**

| Level<br>- | Height<br>[ft] | Fx<br>[Kip] | Fy<br>[Kip] | Mz<br>[Kip·ft] |
|---|---|---|---|---|
| 29 | 342.3 | 19.4 | -30.3 | -403.6 |
| 28 | 331.6 | 22.2 | -36.2 | -470.5 |
| 27MEZZ | 320.9 | 20.7 | -33.0 | -353.0 |
| 27MEP | 310.3 | 21.4 | -34.6 | -475.7 |
| 26 | 299.6 | 17.8 | -27.6 | -387.1 |
| 25 | 288.9 | 17.1 | -26.5 | -374.0 |
| 24 | 278.3 | 16.6 | -25.5 | -364.8 |
| 23 | 267.6 | 16.1 | -24.6 | -356.0 |
| 22 | 256.9 | 15.5 | -23.6 | -344.4 |
| 21 | 246.3 | 15.0 | -22.7 | -334.2 |
| 20 | 235.6 | 14.4 | -21.7 | -320.2 |
| 19 | 224.9 | 13.8 | -20.8 | -307.0 |
| 18 | 214.3 | 13.2 | -19.6 | -294.5 |
| 17 | 203.6 | 12.7 | -19.0 | -280.4 |
| 16 | 192.9 | 12.2 | -18.0 | -267.2 |
| 15 | 182.3 | 11.6 | -17.2 | -254.4 |
| 14 | 171.6 | 11.0 | -16.1 | -242.3 |
| 13 | 160.9 | 10.5 | -15.4 | -229.1 |
| 12 | 150.3 | 9.9 | -14.4 | -215.2 |
| 11 | 139.6 | 9.3 | -13.6 | -204.5 |
| 10 | 128.9 | 8.8 | -12.8 | -193.3 |
| 9 | 118.3 | 8.2 | -11.9 | -180.8 |
| 8 | 107.6 | 7.7 | -11.0 | -168.6 |
| 7 | 96.9 | 4.2 | -4.8 | -95.6 |
| 6 | 86.3 | 3.9 | -4.4 | -87.3 |
| 5 | 75.6 | 3.9 | -4.4 | -86.6 |
| 4 | 63.1 | 4.4 | -5.5 | -94.8 |
| 3 | 46.7 | 5.6 | -7.3 | -129.0 |
| 2 | 23.0 | 3.6 | -5.2 | -107.1 |
| 1 | 0.0 | 1.4 | -2.4 | -44.1 |

**Table 3.2b: Highest peak floor-by-floor loads (Fx, Fy and Mz) against height above "Cellar", 50-year return period wind speeds, structural damping of 2.0% of critical, Option B**

| Level<br>- | Height<br>[ft] | Fx<br>[Kip] | Fy<br>[Kip] | Mz<br>[Kip·ft] |
|---|---|---|---|---|
| 63TBH | 755.5 | 25.5 | -42.2 | -184.5 |
| 62EMR | 736.1 | 45.3 | -115.5 | -644.1 |
| 61RF | 706.8 | 70.2 | -121.8 | -692.1 |
| 60 | 691.3 | 69.7 | -101.2 | -1105.7 |
| 59 | 678.8 | 62.1 | -90.4 | -963.7 |
| 58 | 666.3 | 60.3 | -88.0 | -947.7 |
| 57 | 653.8 | 58.6 | -85.6 | -933.6 |
| 56 | 641.3 | 56.8 | -83.2 | -916.7 |
| 55 | 628.8 | 55.0 | -80.9 | -899.3 |
| 54 | 616.3 | 53.2 | -78.5 | -881.7 |
| 53 | 603.8 | 51.4 | -76.2 | -863.4 |
| 52 | 591.3 | 49.9 | -74.2 | -847.0 |
| 51 | 578.8 | 48.1 | -71.9 | -824.5 |
| 50 | 566.3 | 37.1 | -55.5 | -635.9 |
| 49 | 555.6 | 33.6 | -50.7 | -627.8 |
| 48 | 544.9 | 33.2 | -50.0 | -572.5 |
| 47 | 534.3 | 31.6 | -48.1 | -601.3 |
| 46 | 523.6 | 31.2 | -47.5 | -548.2 |
| 45 | 512.9 | 29.8 | -45.7 | -575.0 |
| 44 | 502.3 | 29.4 | -45.0 | -525.8 |
| 43 | 491.6 | 28.2 | -43.5 | -549.8 |
| 42 | 480.9 | 27.8 | -43.0 | -504.7 |
| 41 | 470.3 | 26.7 | -41.5 | -526.9 |
| 40 | 459.6 | 26.7 | -41.6 | -499.5 |
| 39 | 448.9 | 26.1 | -40.8 | -492.9 |
| 38 | 438.3 | 25.3 | -39.7 | -486.0 |
| 37 | 427.6 | 24.7 | -38.7 | -480.9 |
| 36 | 416.9 | 24.0 | -37.8 | -473.7 |
| 35 | 406.3 | 23.3 | -36.6 | -463.7 |
| 34 | 395.6 | 22.6 | -35.5 | -451.9 |
| 33 | 384.9 | 21.9 | -34.4 | -439.9 |
| 32 | 374.3 | 21.3 | -33.3 | -430.1 |
| 31 | 363.6 | 20.6 | -32.3 | -417.6 |
| 30 | 352.9 | 20.0 | -31.3 | -408.8 |

**Table 3.2b: Highest peak floor-by-floor loads (Fx, Fy and Mz) against height above "Cellar", 50-year return period wind speeds, structural damping of 2.0% of critical, Option B (*con't*)**

| Level<br>- | Height<br>[ft] | Fx<br>[Kip] | Fy<br>[Kip] | Mz<br>[Kip·ft] |
|---|---|---|---|---|
| 29 | 342.3 | 19.4 | -30.2 | -398.6 |
| 28 | 331.6 | 22.2 | -36.1 | -467.0 |
| 27MEZZ | 320.9 | 20.7 | -32.9 | -348.6 |
| 27MEP | 310.3 | 21.3 | -34.6 | -471.8 |
| 26 | 299.6 | 17.8 | -27.5 | -383.9 |
| 25 | 288.9 | 17.1 | -26.4 | -370.9 |
| 24 | 278.3 | 16.6 | -25.5 | -362.0 |
| 23 | 267.6 | 16.1 | -24.5 | -353.4 |
| 22 | 256.9 | 15.5 | -23.6 | -341.6 |
| 21 | 246.3 | 15.0 | -22.6 | -330.0 |
| 20 | 235.6 | 14.4 | -21.6 | -317.9 |
| 19 | 224.9 | 13.8 | -20.7 | -304.9 |
| 18 | 214.3 | 13.1 | -19.6 | -292.4 |
| 17 | 203.6 | 12.7 | -18.9 | -278.4 |
| 16 | 192.9 | 12.2 | -18.0 | -265.6 |
| 15 | 182.3 | 11.6 | -17.1 | -252.9 |
| 14 | 171.6 | 11.0 | -16.0 | -241.0 |
| 13 | 160.9 | 10.5 | -15.4 | -227.9 |
| 12 | 150.3 | 9.9 | -14.4 | -214.2 |
| 11 | 139.6 | 9.3 | -13.5 | -203.4 |
| 10 | 128.9 | 8.8 | -12.8 | -192.6 |
| 9 | 118.3 | 8.2 | -11.9 | -180.3 |
| 8 | 107.6 | 7.7 | -11.0 | -168.2 |
| 7 | 96.9 | 4.2 | -4.8 | -94.9 |
| 6 | 86.3 | 3.9 | -4.4 | -87.0 |
| 5 | 75.6 | 3.9 | -4.4 | -86.1 |
| 4 | 63.1 | 4.4 | -5.5 | -94.4 |
| 3 | 46.7 | 5.6 | -7.3 | -128.2 |
| 2 | 23.0 | 3.6 | -5.2 | -107.1 |
| 1 | 0.0 | 1.4 | -2.4 | -43.0 |

**Table 3.3a: Load case wind directions, 50-year return period wind speeds, structural damping of 2.0% of critical, Option A**

| Load Cases | Description | %Fx | %Fy | %Mz |
|---|---|---|---|---|
| 1 | Peak positive Mx 1 | 25% | **100%** | 20% |
| 2 | Peak positive Mx 2 | -45% | **100%** | 20% |
| 3 | Peak positive Mx 3 | 25% | **100%** | 75% |
| 4 | Peak positive Mx 4 | -45% | **100%** | 75% |
| 5 | Peak negative Mx 1 | 35% | -80% | -35% |
| 6 | Peak negative Mx 2 | 80% | -80% | -35% |
| 7 | Peak negative Mx 3 | 35% | -80% | -75% |
| 8 | Peak negative Mx 4 | 80% | -80% | -75% |
| 9 | Peak positive My 1 | **100%** | -15% | -75% |
| 10 | Peak positive My 2 | **100%** | -50% | -75% |
| 11 | Peak positive My 3 | **100%** | -15% | -40% |
| 12 | Peak positive My 4 | **100%** | -50% | -40% |
| 13 | Peak negative My 1 | -95% | 10% | 75% |
| 14 | Peak negative My 2 | -95% | 60% | 75% |
| 15 | Peak negative My 3 | -95% | 10% | 25% |
| 16 | Peak negative My 4 | -95% | 60% | 25% |
| 17 | Peak positive Mz 1 | 70% | -40% | -95% |
| 18 | Peak positive Mz 2 | 20% | -40% | -95% |
| 19 | Peak positive Mz 3 | 70% | -65% | -95% |
| 20 | Peak positive Mz 4 | 20% | -65% | -95% |
| 21 | Peak negative Mz 1 | -85% | 40% | **100%** |
| 22 | Peak negative Mz 2 | -20% | 40% | **100%** |
| 23 | Peak negative Mz 3 | -85% | 90% | **100%** |
| 24 | Peak negative Mz 4 | -20% | 90% | **100%** |

**Table 3.3b: Load case wind directions, 50-year return period wind speeds, structural damping of 2.0% of critical, Option B**

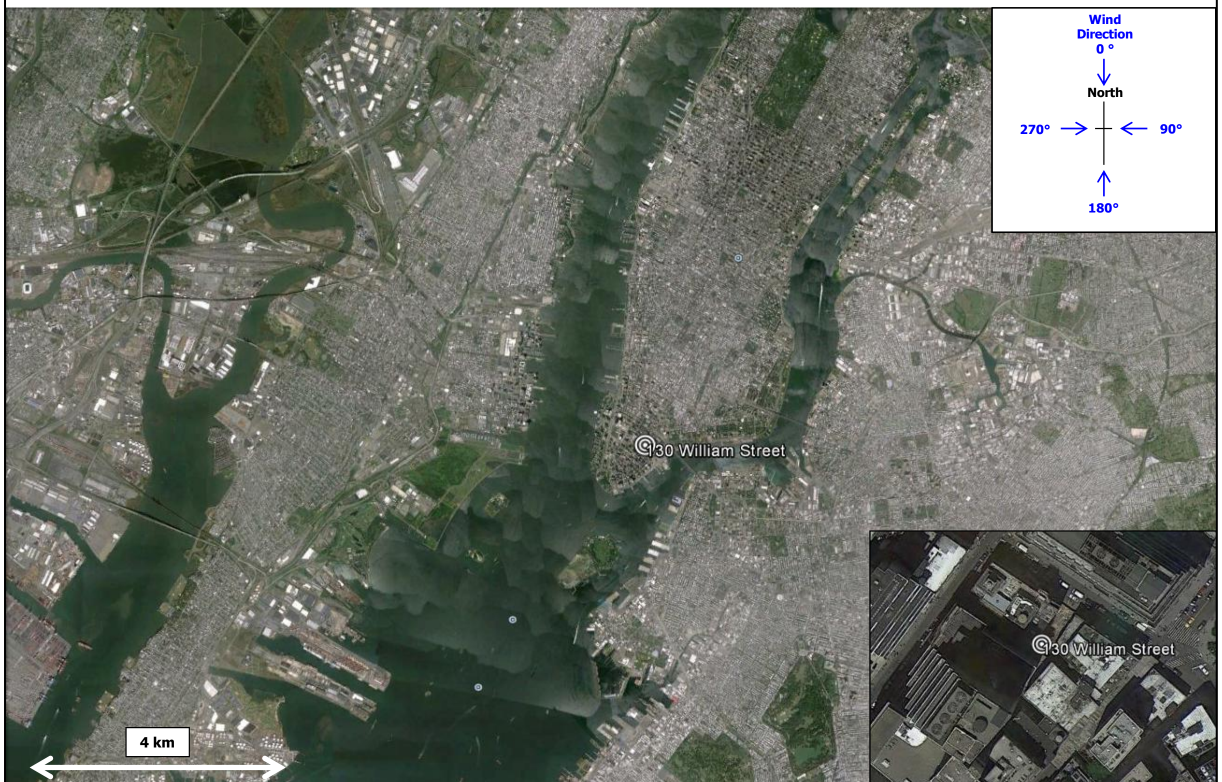
| Load Cases | Description | %Fx | %Fy | %Mz |
|---|---|---|---|---|
| 1 | Peak positive Mx 1 | 25% | **100%** | 20% |
| 2 | Peak positive Mx 2 | -45% | **100%** | 20% |
| 3 | Peak positive Mx 3 | 25% | **100%** | 75% |
| 4 | Peak positive Mx 4 | -45% | **100%** | 75% |
| 5 | Peak negative Mx 1 | 35% | -80% | -35% |
| 6 | Peak negative Mx 2 | 80% | -80% | -35% |
| 7 | Peak negative Mx 3 | 35% | -80% | -75% |
| 8 | Peak negative Mx 4 | 80% | -80% | -75% |
| 9 | Peak positive My 1 | **100%** | -15% | -80% |
| 10 | Peak positive My 2 | **100%** | -50% | -80% |
| 11 | Peak positive My 3 | **100%** | -15% | -40% |
| 12 | Peak positive My 4 | **100%** | -50% | -40% |
| 13 | Peak negative My 1 | -95% | 10% | 75% |
| 14 | Peak negative My 2 | -95% | 60% | 75% |
| 15 | Peak negative My 3 | -95% | 10% | 25% |
| 16 | Peak negative My 4 | -95% | 60% | 25% |
| 17 | Peak positive Mz 1 | 70% | -40% | -95% |
| 18 | Peak positive Mz 2 | 20% | -40% | -95% |
| 19 | Peak positive Mz 3 | 70% | -65% | -95% |
| 20 | Peak positive Mz 4 | 20% | -65% | -95% |
| 21 | Peak negative Mz 1 | -85% | 40% | **100%** |
| 22 | Peak negative Mz 2 | -20% | 40% | **100%** |
| 23 | Peak negative Mz 3 | -85% | 90% | **100%** |
| 24 | Peak negative Mz 4 | -20% | 90% | **100%** |

**Table 3.4: Peak-combined accelerations [milli-g] on "Level 60" for Options A and B on "Level 60" for 1-year and 10-year return period wind speed**

| Accelerations on Level 60 | Option A | | | Option B | | |
|---|---|---|---|---|---|---|
| | 0.9n | 1.0n | 1.1n | 0.9n | 1.0n | 1.1n |
| 1-Year | 9.5 | 9.0 | 8.5 | 9.6 | 9.1 | 8.6 |
| 10-Year | 18.7 | 17.0 | 16.0 | 18.8 | 17.2 | 16.2 |

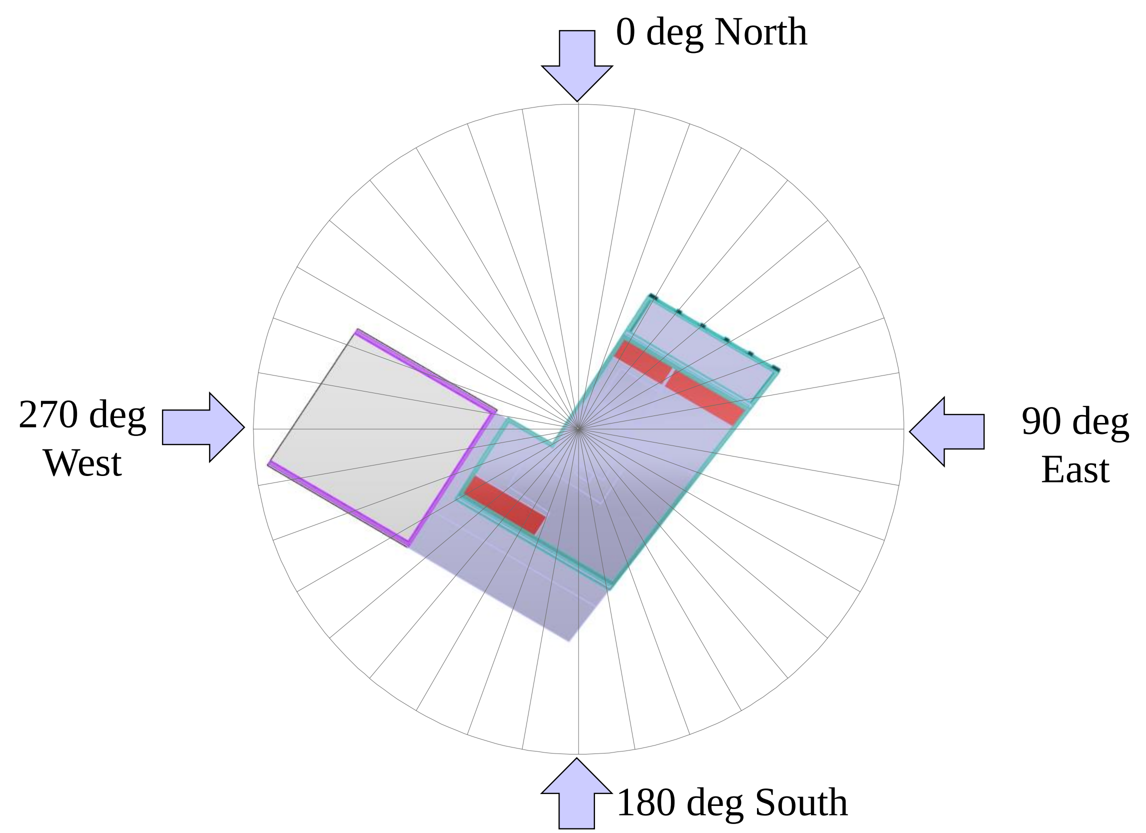
# Figures

## Figure 1.1: Aerial view of the proposed site




| | | | | |
|---|---|---|---|---|
| 431848 – 130 William Street<br>Aerial view of the proposed site | BMT Fluid Mechanics | Drawing no.:<br>431848-FIG- 1.1 | Status:<br>Preliminary | By: K Srisopa<br>Date: 29-Mar-16 |

**Figure 1.2: Plan view of the proposed development**




| | | | |
|---|---|---|---|
| 431848 – 130 William Street<br>Plan View and Wind Direction Definition | Drawing no.:<br>431848-FIG- 1.2 | Status:<br>Preliminary | By: K Srisopa<br>Date: 29-Mar-16 |

**Figure 2.1a: Axis system definition, plan view of the proposed development**

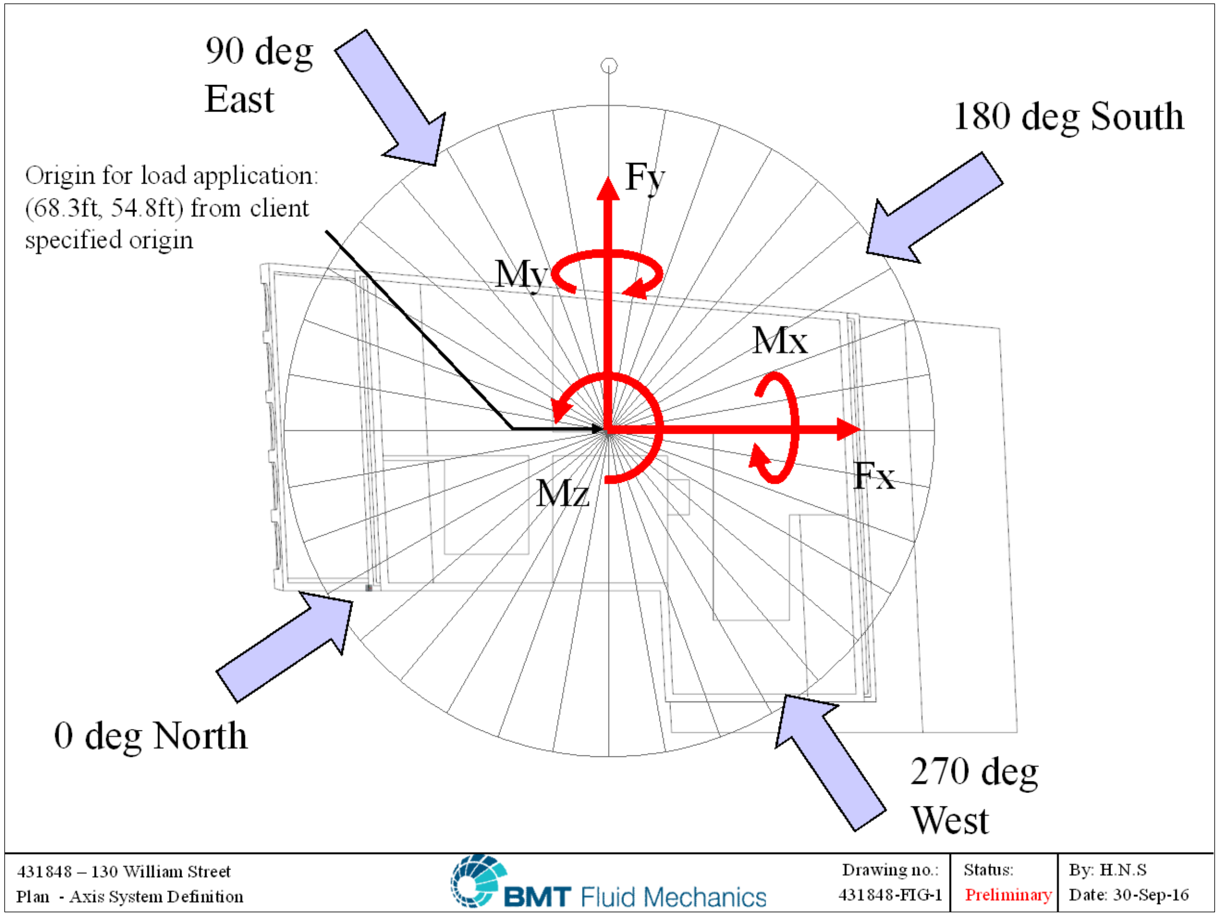


**Figure 2.1b: Axis system definition, southeast elevation of proposed development**

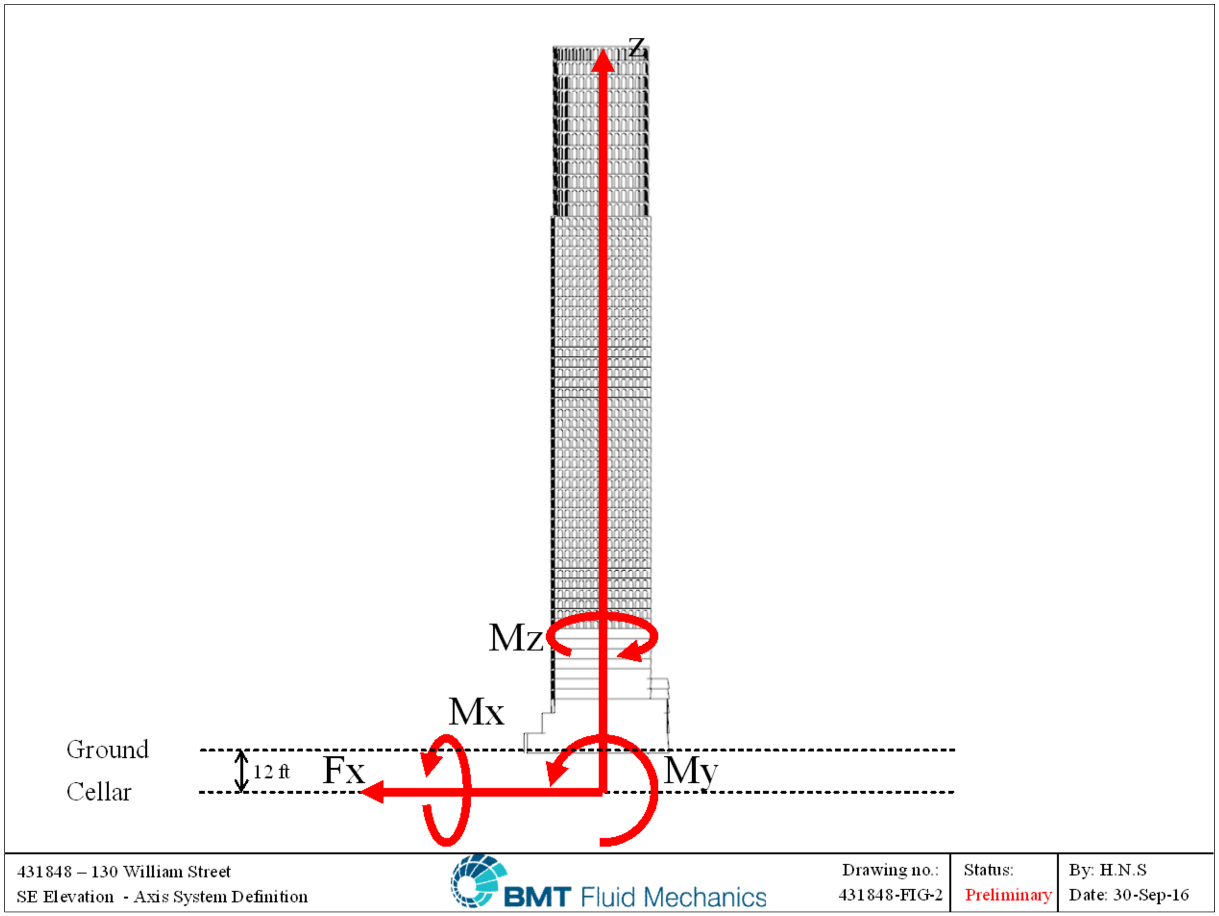


**Figure 3.1a: Highest peak floor-by-floor loads ($F_x$, $F_y$ and $M_z$), 50-year return period wind speeds, against height above the "Cellar", structural damping of 2.0% of critical, Option A**

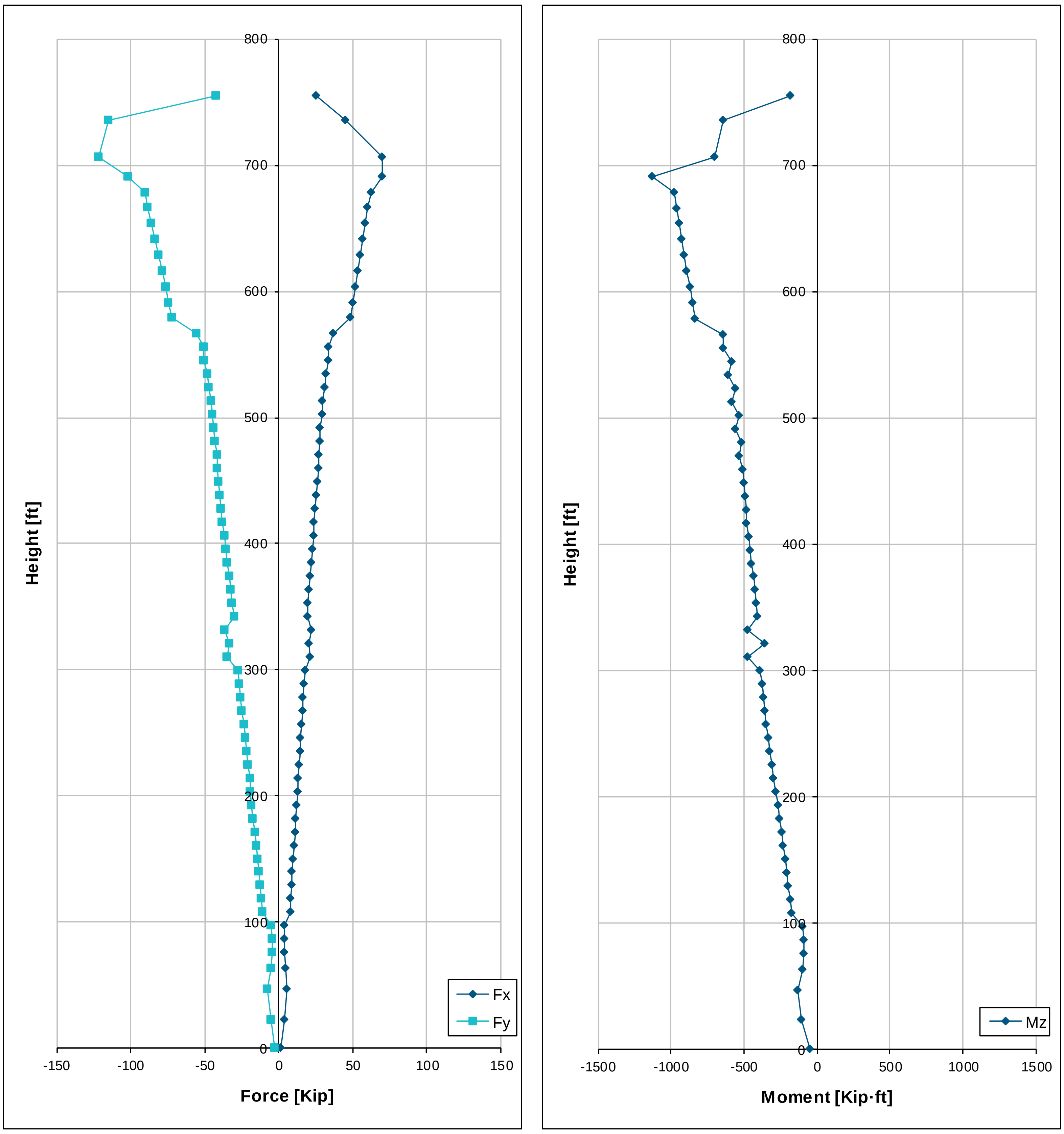
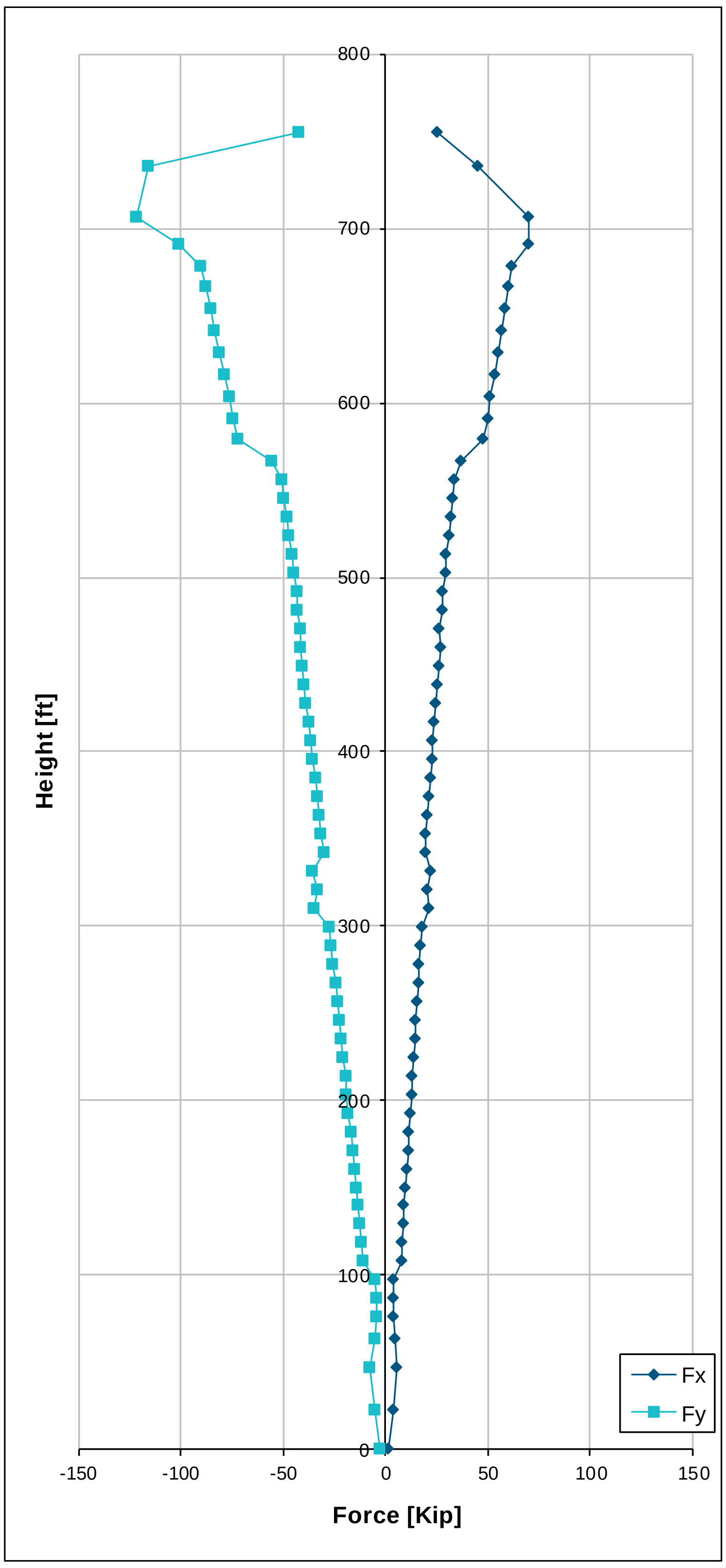


**Figure 3.1b: Highest peak floor-by-floor loads ($F_x$, $F_y$ and $M_z$), 50-year return period wind speeds, against height above the "Cellar", structural damping of 2.0% of critical, Option B**



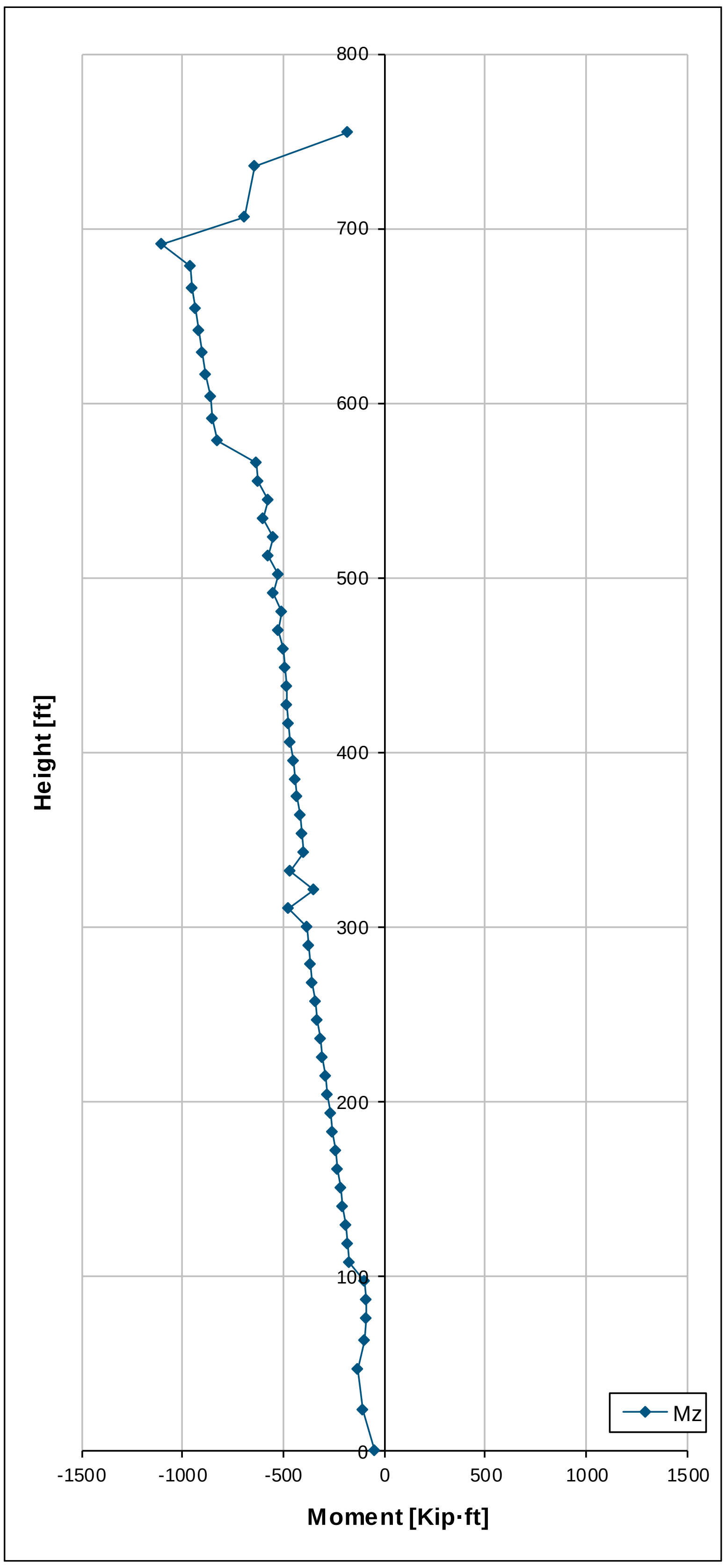
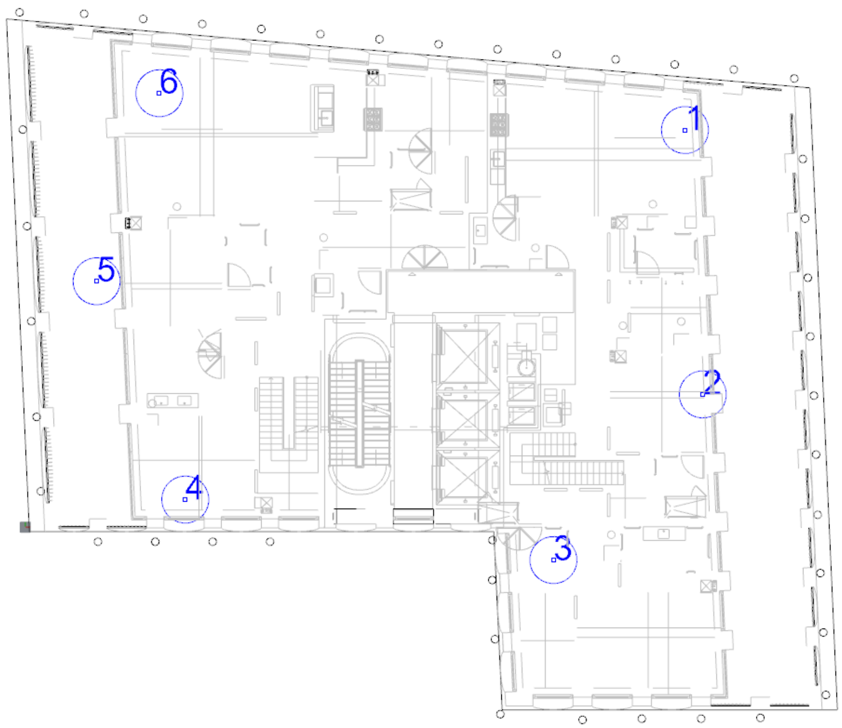
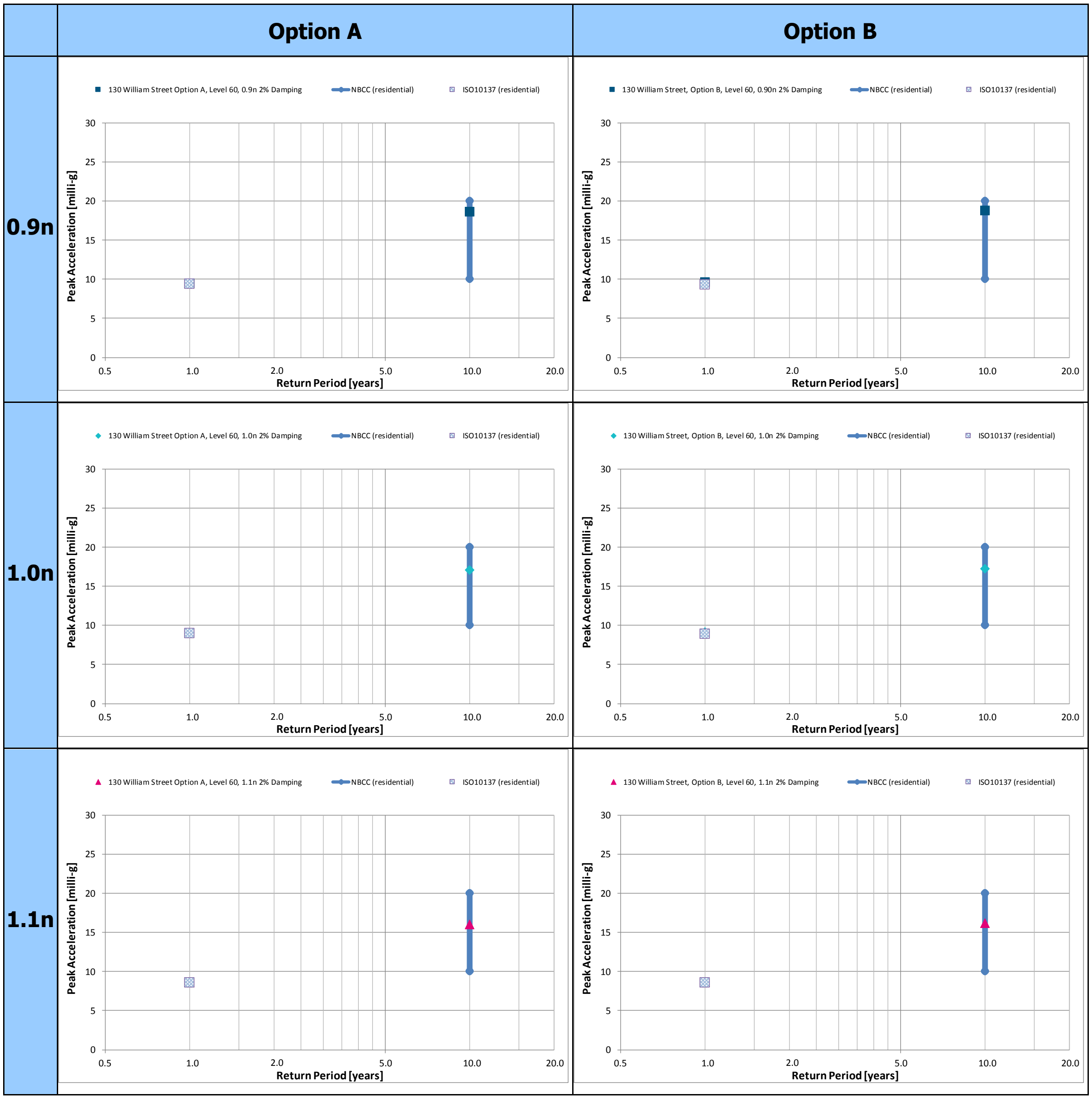



**Figure 3.2: Locations of interest for acceleration evaluation, "Level 60"**




| | | | | |
|---|---|---|---|---|
| 431848 – 130 William Street<br>Locations for Acceleration Evaluation, 60th Floor | BMT Fluid Mechanics | Drawing no.:<br>431848-FIG- 3.5 | Status:<br>Preliminary | By: K Srisopa<br>Date: 29-Mar-16 |

**Figure 4.1: Comparison of the peak combined accelerations on 1- and 10-year return period wind speeds, 0.9n, 1.0n and 1.1n of natural frequencies, with occupant comfort criteria on "Level 60" for Options A and B**



# APPENDIX A. WIND ANALYSIS

## A.1. ESDU Wind Analysis

A detailed analysis was carried out to determine the wind properties at the site. The wind analysis is based on the widely accepted Deaves and Harris model of the atmospheric boundary layer (ABL), as defined in ESDU Item 01008[5], and has provided wind profiles describing the variation of wind speed and turbulence intensity with height and wind direction. From this analysis representative profiles were defined as targets for the ABL simulation in the wind tunnel.

### A.1.1. Roughness Changes for ESDU Wind Analysis

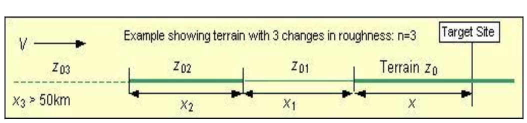
The wind analysis takes detailed account of the variation of the upwind terrain on each wind sector. The roughness changes used in the analysis for the current study are given in Table A.1 below.

**Table A.1: Terrain Roughness Changes from the Site**

| Wind Dir | $z_0$ [ft] | $x_0$ [ft] | $z_{01}$ [ft] | $x_{01}$ [ft] | $z_{02}$ [ft] | $x_{02}$ [ft] | $z_{03}$ [ft] | $x_{03}$ [ft] | $z_{04}$ [ft] | $x_{04}$ [ft] | $z_{05}$ [ft] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0° | 3.28 | 20,000 | 0.52 | 8,800 | 0.98 | | | | | | |
| 30° | 3.28 | 37,000 | 0.98 | | | | | | | | |
| 60° | 3.28 | 10,000 | 1.08 | 12,000 | 0.98 | | | | | | |
| 90° | 3.28 | 3,000 | 0.16 | 6,500 | 0.98 | | | | | | |
| 120° | 3.28 | 2,000 | Water | 2,200 | 1.64 | 6,000 | 0.98 | 38,500 | 0.16 | 47,300 | Water |
| 150° | 3.28 | 2,000 | Water | 3,400 | 1.64 | 3,400 | 0.98 | 57,400 | Water | | |
| 180° | 3.28 | 2,300 | Water | 3,400 | 0.16 | 3,400 | 0.98 | 44,600 | Water | 32,800 | 0.16 |
| 210° | 3.28 | 3,700 | Water | 37,000 | 0.43 | 30,400 | 0.03 | 44,000 | 0.98 | | |
| 240° | 3.28 | 3,300 | Water | 20,300 | 0.98 | | | | | | |
| 270° | 3.28 | 4,000 | Water | 6,700 | 0.98 | 20,300 | 0.33 | 25,700 | 0.98 | | |
| 300° | 3.28 | 4,000 | Water | 4,700 | 1.64 | 6,800 | 0.98 | | | | |
| 330° | 3.28 | 6,000 | Water | 6,800 | 1.64 | 6,800 | 0.98 | | | | |

Where **$x_0$**: upwind fetch
**$z_0$**: roughness groups length

N.B: **$z_0$** = water (sea, lakes, estuaries)
0.07 (open country)
0.33 (sparse suburban)
0.98 (suburban)
1.64 (urban)
2.30-3.28 (city center)



### A.1.2. Wind Profiles

Figure A.1 shows the variation of longitudinal turbulence intensity ($I_u$) with wind direction at a height of 770ft (considered the reference height of the proposed development). Due to the variation of wind properties with wind direction, three target profiles have been selected for the boundary layer simulation. The target profiles and range of wind angles for each wind tunnel profile are as follows:

| Profile | Wind Angle Range | Target Angle |
|---|---|---|
| Exp 1 | 190°-220° | 210° |
| Exp 2 | 100°-180°, 230°-280° | 150° |
| Exp 3 | 0°-80°, 290°-350° | 0° |

Figures A.2 and A.3 show the variation of mean wind-speed (normalized by the mean wind speed at the reference height of 770ft) and turbulence intensity with height for winds approaching the site from the four primary quarters.

Figures A.4a to A.4c present the profile of mean wind speed and longitudinal turbulence intensity used in the tests. The wind speed profiles are normalized by the mean wind speed at a height of 770ft. It can be seen that, over the range of heights of interest, the boundary layer simulation used in the tests was a good representation of that expected for the site at full scale which satisfies the experimental requirements of the American Society of Civil Engineering (ASCE Manuals and Reports on Engineering Practice No. 67[1]).

**Figure A.1: Variation of longitudinal turbulence intensity ($I_u$) with wind direction at 770ft height, including reference turbulence levels**

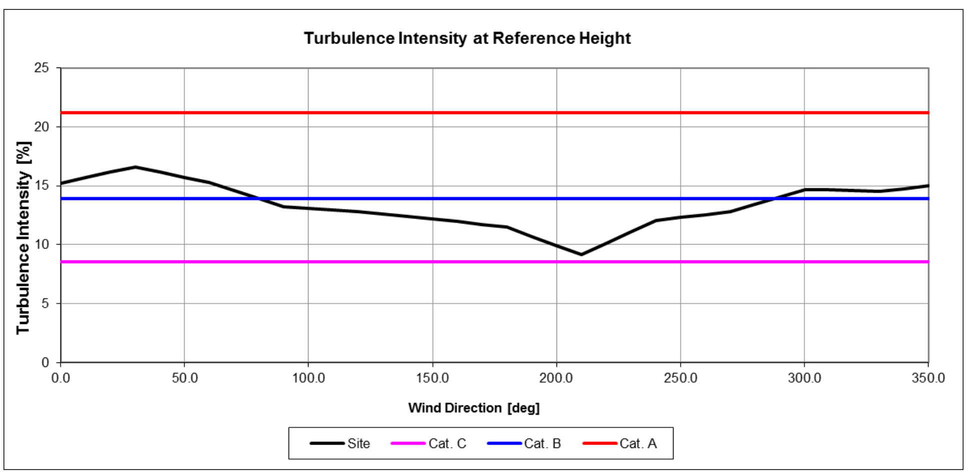


**Figure A.2: Variation of mean wind speed normalised by mean wind speed at a reference height of 770ft**

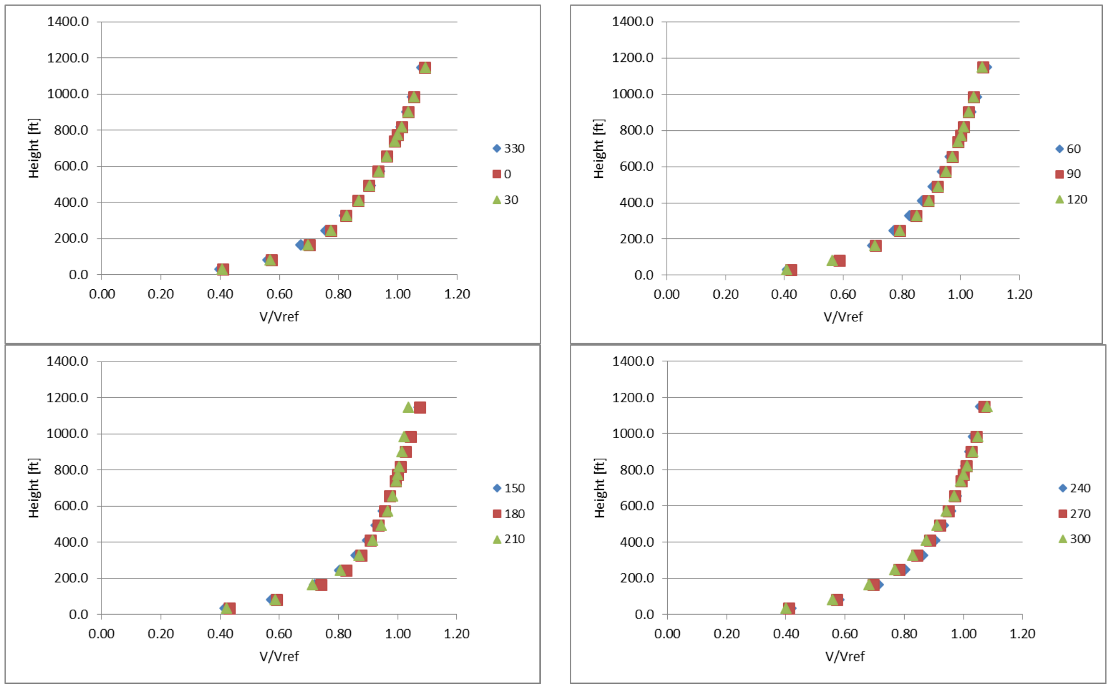


**Figure A.3: Variation of longitudinal turbulence intensity with wind direction a reference height of 770ft**

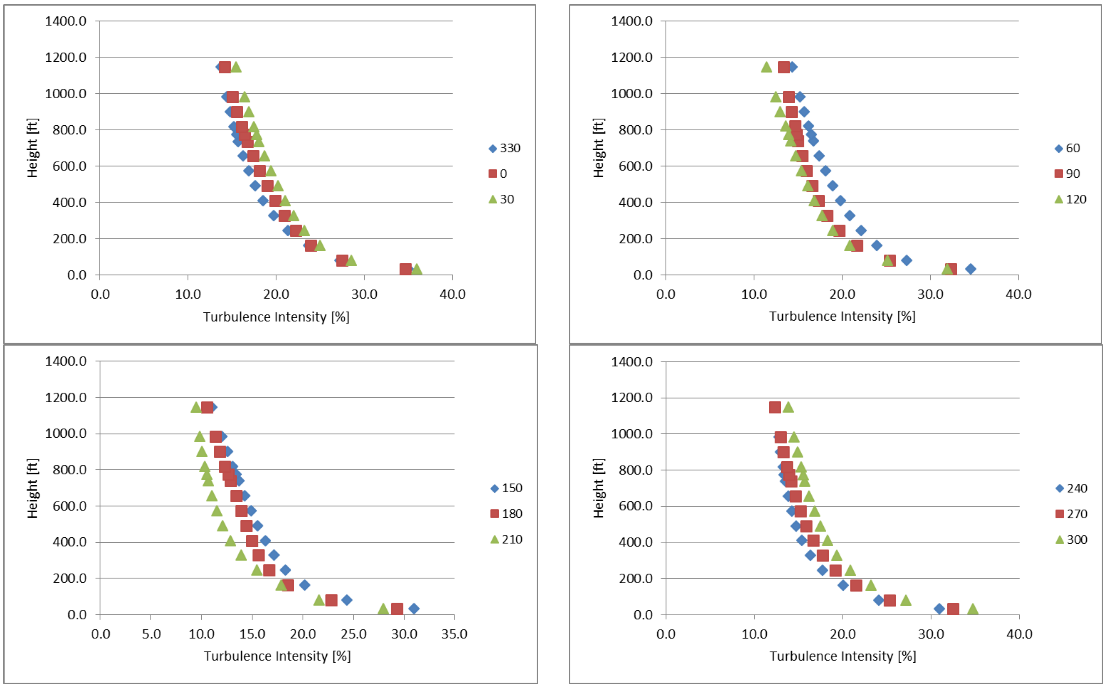



**Figure A.4a: Mean wind speed ($V_{mean}$/$Vmean_{(ref)}$), gust wind speed ($V_{gust}$/$V_{mean(ref)}$) and turbulence intensity profile (Iu) used in the study (Exp 1)**

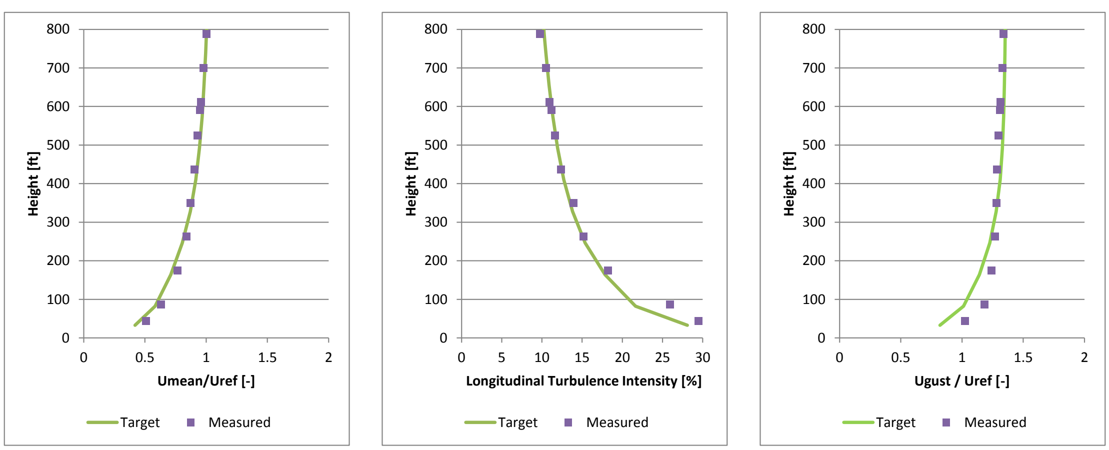


**Figure A.4b: Mean wind speed (Vmean/$V_{mean(ref)}$), gust wind speed ($V_{gust}$/$V_{mean(ref)}$) and turbulence intensity profile (Iu) used in the study (Exp 2)**

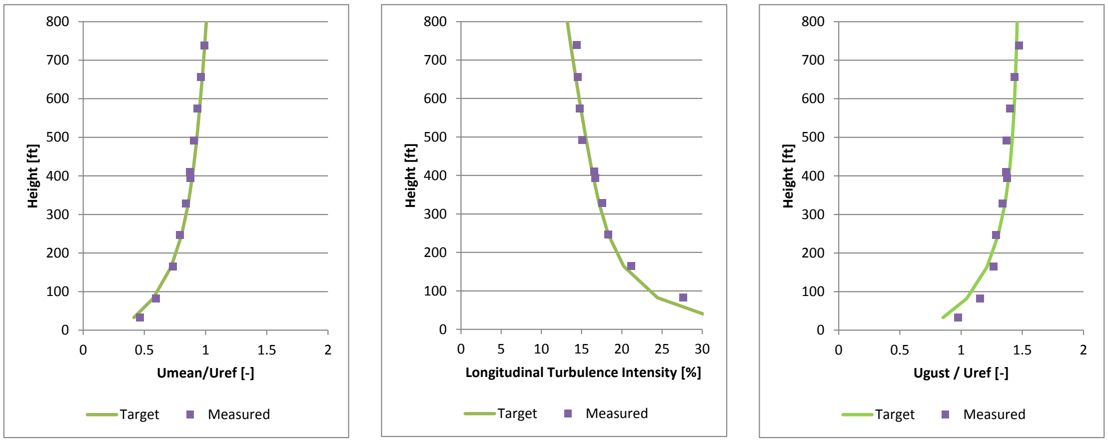
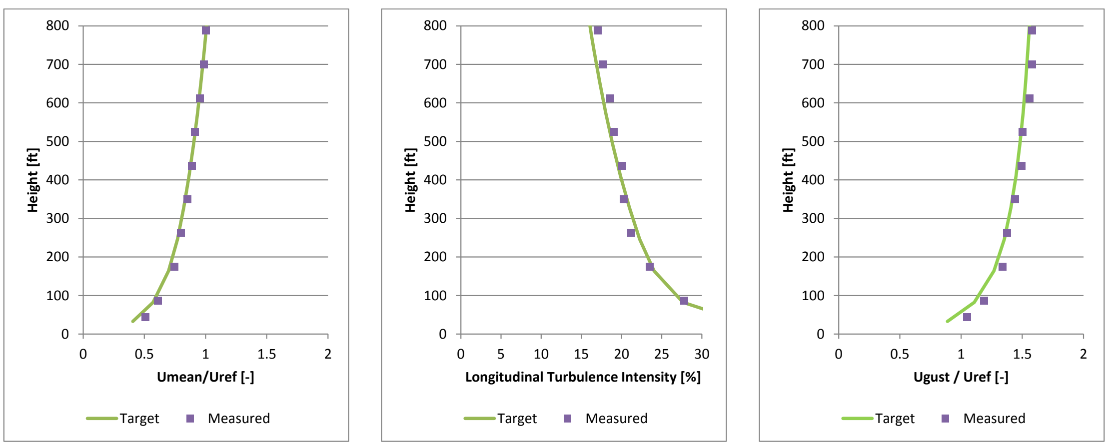



**Figure A.4c: Mean wind speed ($V_{mean}$/$V_{mean(ref)}$), gust wind speed ($V_{gust}$/$V_{mean(ref)}$) and turbulence intensity profile (Iu) used in the study (Exp 3)**



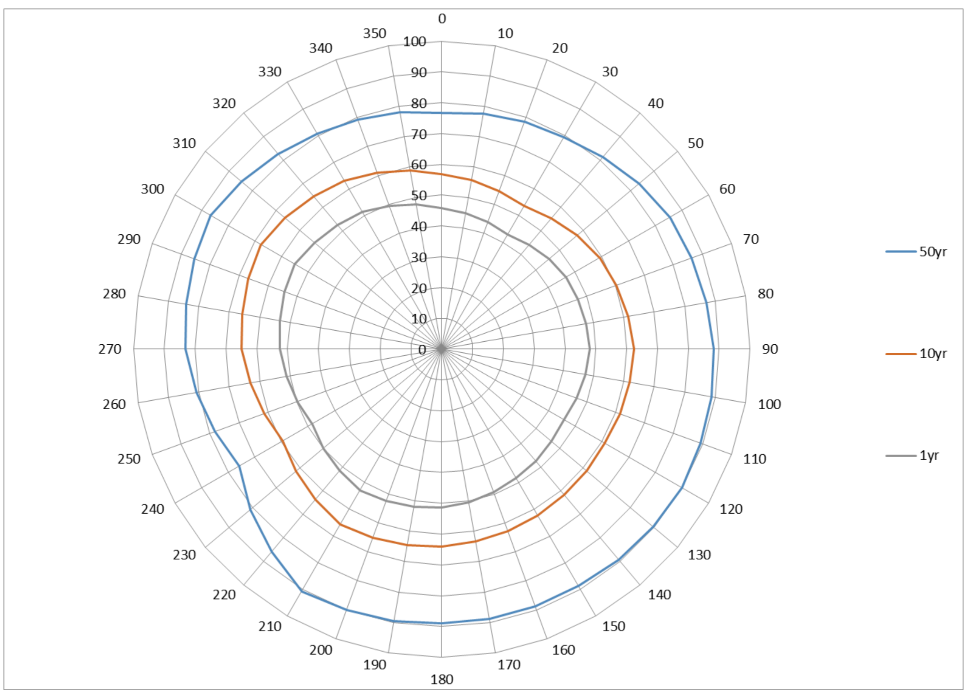
## A.2. Design Wind Speed

The wind analysis for the studies assumes a reference height of 770ft, the approximate height of the proposed development. The design wind speeds at the reference height are shown in Figure A.5 in terms of mean-hourly wind speeds predicted by the ESDU Item 01008[5] analysis using the terrain analysis for the site.

The study defines design wind loads for the proposed development based on design wind speeds derived from the 50-year return period wind speed of **98mph** (referenced to 3-second gust averaging period at 33ft height over Exposure C terrain) as stipulated in the New York City Building Code[2]. Furthermore, the design wind speeds have been used in conjunction with the directionality information derived for the extreme wind events within the region.

The reported serviceability studies are based on 1-year and 10-year return period design wind speed in conjunction with the use of the directionality information derived for the extreme wind events in the region.

**Figure A.5: Mean-hourly design wind speeds (mph) at reference height, with return periods of 1-, 10- and 50-year (all accounting for upstream terrain effect and wind directionality of extreme wind events)**



# APPENDIX B. WIND TUNNEL AND MODEL DETAILS

## B.1. Wind Tunnel Specifications

All the tests were conducted in BMT's Boundary Layer Wind Tunnel which has a test section 15.7ft wide, 7.9ft high and 49.2ft long with a 14.4ft diameter multiple plate turntable and a remotely controlled 3-dimensional traversing system. The operating wind speed range is 0.45 – 100.7mph.

The turbulent boundary layer is set up using an arrangement of roughness elements distributed over the floor of the wind tunnel and a two-dimensional barrier placed at the entrance to the test section.

## B.2. Model

### B.2.1. Information

The wind tunnel model of the proposed development was constructed based on the 3D models supplied by *Adjaye Associates*, the architects of the scheme, as follows:

| **Drawing** | **Date** |
|---|---|
| 20160303 WST FULL MODEL_WIND TUNNEL.3dm | March 7$^{th}$ 2016 |

The wind tunnel models representative of the surrounding building morphology was constructed based on public domain information.

All models were reviewed and approved by the design team, prior to testing.

### B.2.2. Scale

When choosing the scale for model scale wind tunnel testing of large scale building arrangements a compromise / balance needs to be struck between the need for:

- An accurate representation of geometric features that are deemed to affect the wind regime of the building(s) under consideration at full scale
- A comprehensive representation of all adjacent buildings and topographic features that are deemed to affect the wind regime of the building(s) under consideration at full scale
- A best match to the generic wind characteristics of the incident winds at full scale in terms of variation of wind speed and turbulence characteristics with height

- Well-conditioned experimental testing parameters that ensure that the measurable signals in the wind tunnel tests are large enough so that they can be recorded with sufficient accuracy by bespoke instrumentation installed on the wind tunnel models

In this regard a model scale of *1:400* has been adopted. At this scale the model is large enough to allow a good representation of the details that are likely to affect the local and overall wind flows at full scale. In addition, this scale enables a good simulation of the turbulence properties of the wind to be achieved.

### B.2.3. Model Photos

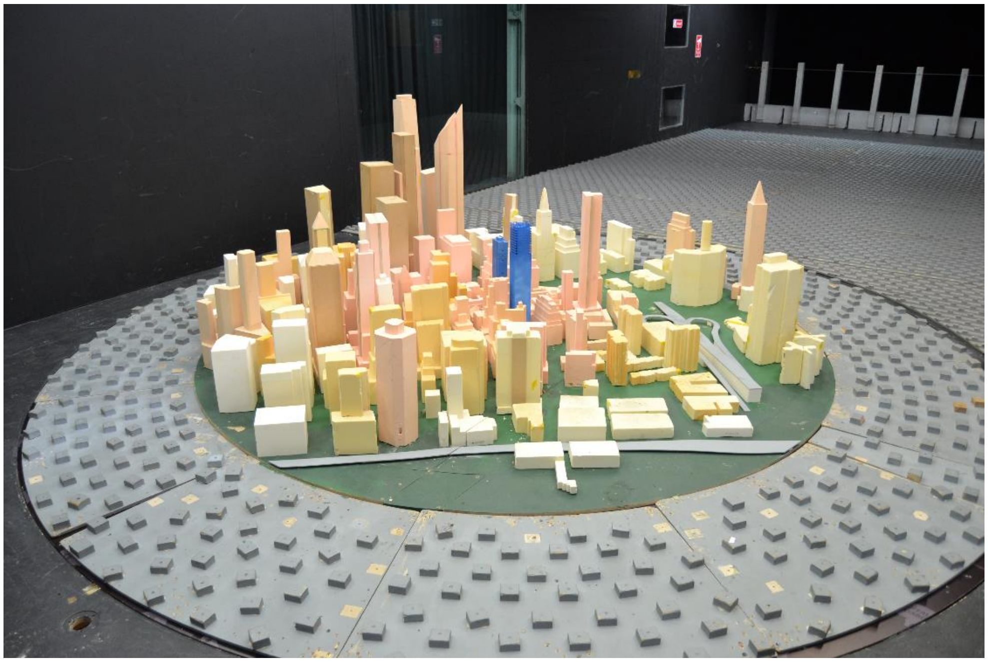
Images of the wind tunnel model are presented as follows:

- Figure B.1 – Proposed development, viewed from north
- Figure B.2 – Proposed development, close up view
- Figure B.3 – Wind tunnel setup, viewed from downstream

**Figure B.1: Proposed development, viewed from north**

**Figure B.2: Proposed development, close up view**

**Figure B.3: Wind tunnel set-up, viewed from downstream**



# APPENDIX C. INSTRUMENTATION AND EXPERIMENTAL TECHNIQUES

The wind tunnel tests were conducted in accordance with the requirements of the American Society of Civil Engineers (ASCE) Manual of Practice No.67 for Wind Tunnel Studies[1].

## C.1. Instrumentation

The fluctuating wind loads were measured using a six-component high frequency piezo electric force balance in conjunction with a single-conditioning unit.

The fluctuating wind loads were measured in terms of the shear forces, bending moments and torque (6 components: Fx, Fy, Fz, Mx, My & Mz) at ground level for 36 wind directions (10° increments). Data records were of sufficient length to enable the probability statistics and spectra to be computed in addition to the mean values.

## C.2. Experimental Conditioning

The base loads were acquired for a time corresponding to a fixed period at full scale. The time scaling is given by:

$$\frac{T_m}{T_f} = \frac{L_m}{L_f} . \frac{V_f}{V_m}$$

where T is time, L is length, V is wind speed and sub-scripts *m* and *f* refer to model and full-scale quantities respectively. With $L_m/L_f$ (the model scale) fixed at 1/400, $V_f/V_m$ was chosen to be 1/0.4 to give a value of $T_m/T_f$ of 1/160. The wind tunnel was therefore run to give a speed equivalent to 0.4 of the design wind speed.

# APPENDIX D. STRUCTURAL AND DYNAMIC PROPERTIES

The structural properties information is referred to the axis system presented in Figures 2.1a and 2.1b.

The analysis and calculations for the proposed development have been based on structural/dynamic properties provided by McNamara Salvia issued to BMT on December 9$^{th}$ and 16$^{th}$, 2016. The properties relevant to the wind load analysis are as follows:

| Data | File | Date |
|---|---|---|
| Structural Properties for Option A | 2016-12-16_130William-Wind Tunnel Prop_v29.1_rev1.xlsx | 12/16/2016 |
| Structural Properties for Option B | 2016-12-09_130William-Wind Tunnel Prop_v29.1_0.90Facade.xlsx | 12/09/2016 |

**Figure D.1a: Contribution to modal masses by different components for Option A**

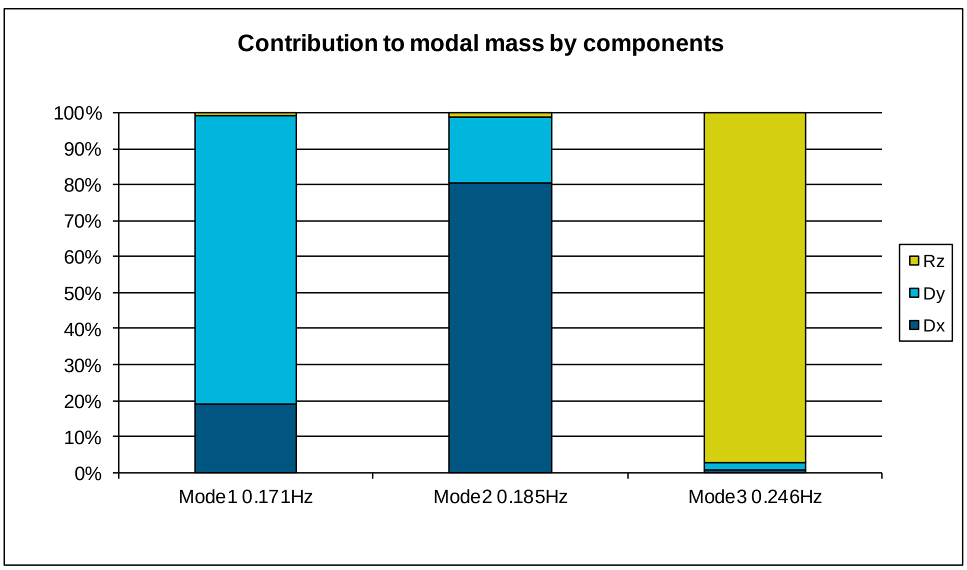
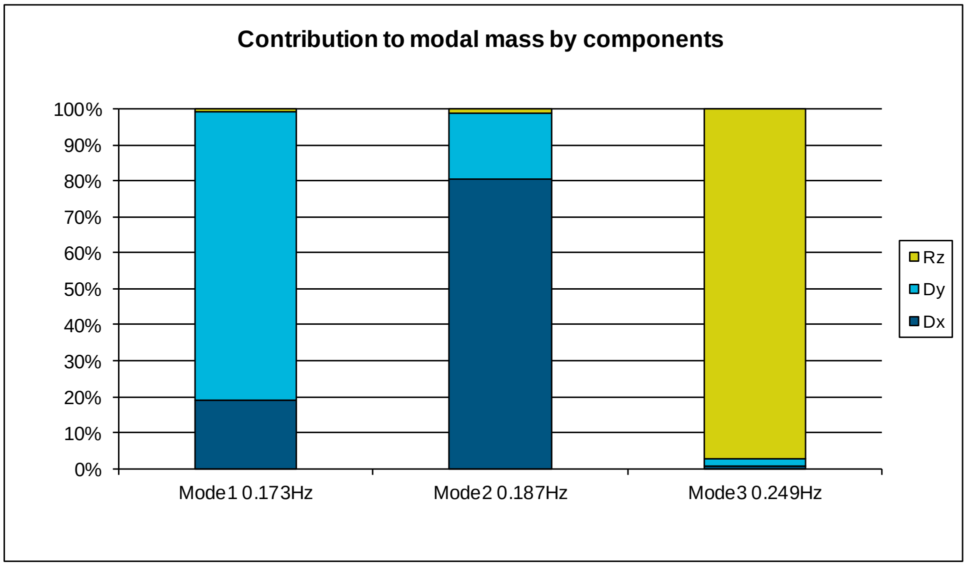


**Figure D.1b: Contribution to modal masses by different components for Option B**



# APPENDIX E. STRUCTURAL LOAD CASES

## E.1. Selection of Key Wind Directions

In order to provide an envelope of load cases for the structural design of the tower, the directional variation of base loads have been inspected to extract key wind directions relating to the specific peak load scenarios listed in the table below.

| Load Cases | Description |
|---|---|
| 1 | Peak positive Mx 1 |
| 2 | Peak positive Mx 2 |
| 3 | Peak positive Mx 3 |
| 4 | Peak positive Mx 4 |
| 5 | Peak negative Mx 1 |
| 6 | Peak negative Mx 2 |
| 7 | Peak negative Mx 3 |
| 8 | Peak negative Mx 4 |
| 9 | Peak positive My 1 |
| 10 | Peak positive My 2 |
| 11 | Peak positive My 3 |
| 12 | Peak positive My 4 |
| 13 | Peak negative My 1 |
| 14 | Peak negative My 2 |
| 15 | Peak negative My 3 |
| 16 | Peak negative My 4 |
| 17 | Peak positive Mz 1 |
| 18 | Peak positive Mz 2 |
| 19 | Peak positive Mz 3 |
| 20 | Peak positive Mz 4 |
| 21 | Peak negative Mz 1 |
| 22 | Peak negative Mz 2 |
| 23 | Peak negative Mz 3 |
| 24 | Peak negative Mz 4 |

## E.2. Joint Response of Load Components

The correlation coefficient enables a companion value of one load component to be determined that is expected to be present when a second component reaches its peak value. The companion load applied with peak load is defined as

$$\hat{M}_j = \overline{M}_j + C_{ij} m_j'$$

where

$\overline{M}_j$ is the mean load of component j

$C_{ij}$ is the correlation between dynamic components i and j

$m_j'$ is the fluctuating (background and resonant) load of component j

This approach enables the peak and companion floor-by-floor load distributions to be defined accounting for the non-simultaneous action of peak loading in time for the ten critical wind directions. These distributions are then presented as a proportion of their (all-directional) peak floor-by-floor distributions respectively.

# APPENDIX F. AERODYNAMIC LOAD COEFFICIENTS

Figure F.1: Mean load coefficients

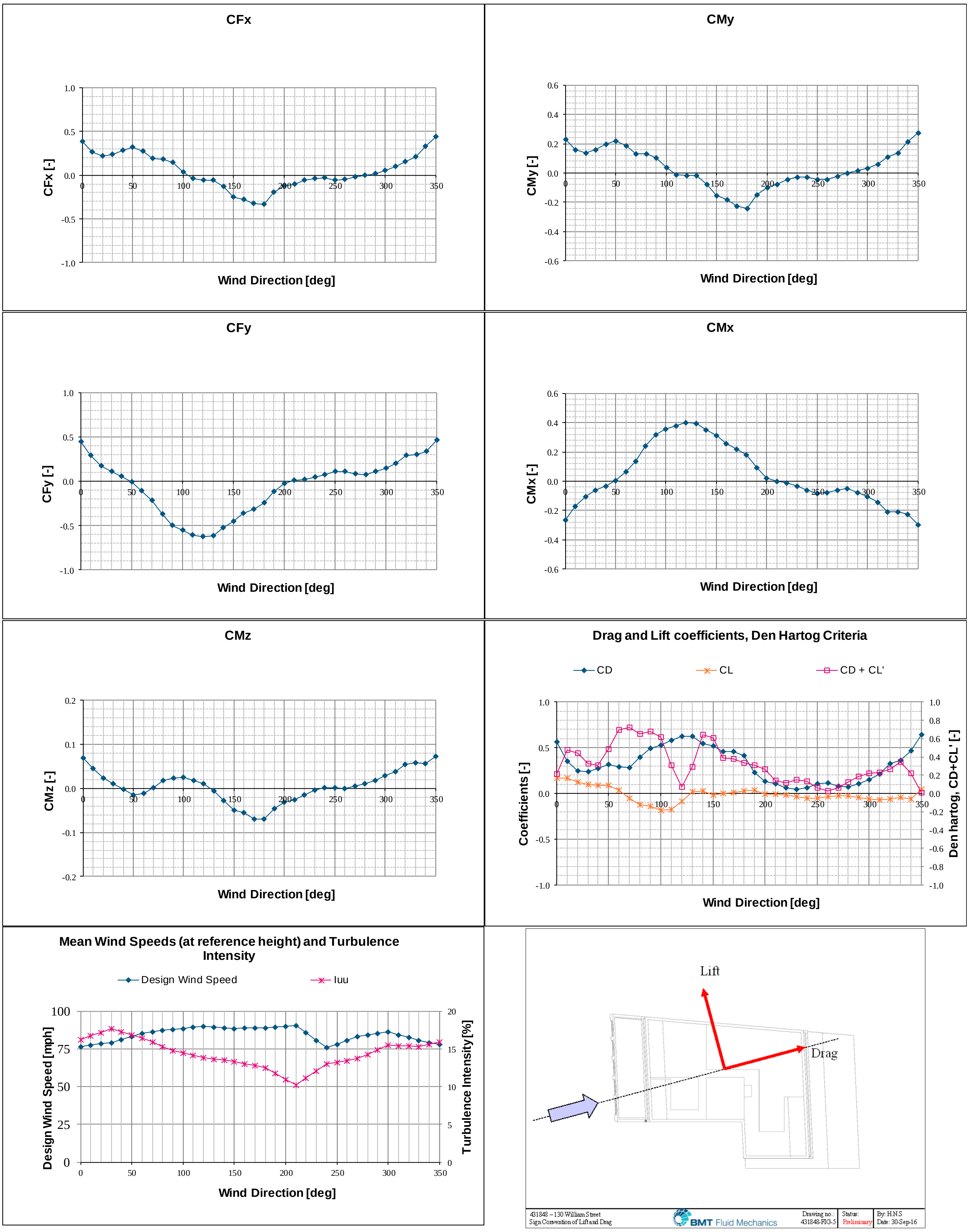


**Table F.1: Normalizing parameters for load coefficients**

| | |
|---|---|
| ***A*, Ref. Area [ft$^2$]** | 84,700 |
| ***W*, Reference Average Width for Mz [ft]** | 110 |
| ***H*, Reference Height [ft]** | 770 |
| ***U*, Reference Speed [mph]** | As shown in Appendix A.2 |

$$CFx, CFy = \frac{Fx, Fy}{\frac{1}{2}\rho U^2 A}$$

$$CMx, CMy = \frac{Mx, My}{\frac{1}{2}\rho U^2 AH}$$

$$CMz = \frac{Mz}{\frac{1}{2}\rho U^2 AW}$$

# APPENDIX G. OVERALL LOAD RESULTS

The main results of the overall wind loads study are provided in a series of Excel Spread Sheet tables and plots enclosed:

## G.1. Registry File

See File:

431848_Registry_v14.xlsx

## G.2. Floor by Floor Loads and Base Loads:

See File:

431848FBF001v8.xlsx

431848FBF001v9.xlsx

## G.3. Accelerations:

See File:

431848ACC001v5.xlsx

431848ACC001v6.xlsx