Aerodynamic Wind load values on Medium-rise Storey Building in Maiduguri

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Abstract

Simple static behaviour of wind loading, which is universally applied to design of typical low to medium-rise structures, can be satisfactorily inappropriately conservative for design of high-rise buildings. This conservativeness could be due to several reasons, and one such factor could be the blind use of analysis and design software’s. Most cases the user have little access to the main program frame either for auditing or otherwise. It is highly probable that the resulting outcomes from such interfaces might lead to erroneous results and possible under or over estimations. Additionally, the simplified treatment for deriving lateral loads has issues in addressing challenges such as the effects of resonance, acceleration, damping, structural stiffness, interference from other structures, e.t.c.. These are all important factors in wind design considerations for high-rise buildings. Hence, this paper provides an assessment for high-rise building wind design using both software’s and other computation method in exploring gains or otherwise over the simplified approaches of British wind code. The resulting outputs from the use of ETABS and the simplified approach show insignificant variation between them because the p-value from the statistical analysis is less than 0.05.

Keywords: wind load, Multi-Storey Building, Maiduguri

1 INTRODUCTION

Proper anticipation and correct assessment of wind effects is an important aspect of successful design of multi storey buildings in areas of the world not affected by earth tremor. The effects are complicated and depend on various Physio-mechanical and environmental conditions. The wind flow around buildings creates loading and associated response on both structural and cladding elements. Wind effects are naturally of dynamic nature since its gust or pulsation is time dependent, whereas the structural loading and response can be considered as either static, dynamic or aero-elastic depending on the response characteristics of the structure (Snebjörnsson, 2002). The wind loading on structures involves in certain cases, considerable complexities that must be taken into account in order to achieve safe and serviceable design. The development of modern materials and construction techniques has resulted in emergence of a new generation of structures that are often to a degree unknown in the past, remarkably flexible, low in damping and light in weight. Such structures generally exhibit an increased susceptibility to the action of wind. Accordingly, it is the task of the engineer or designer to ensure that the performance of the structures subjected to the action of the wind will be adequately addressed during their anticipated life span in the context of structural safety, stability and serviceability.

The slenderness ratio of buildings gives an indication of the wind flow around buildings and the respective response characteristics. Building slenderness ratios (H/B) whose ratios of height H to minimum width B are used to represent low, medium and high-rise buildings; Tall building if H/B > 0.5 else otherwise (Cook, 1990). Other definitions consider the building flexibility to dynamic response. For example, when buildings or other structures have a height exceeding five times the least horizontal dimension (h/d > 5) or when there is reason to believe that the natural frequency is less than 1 Hz (natural period greater than 1 sec) (Taranath, 2010).

Therefore, objectives of this research work is attempted to provide an outline of the simple but appropriate method of wind assessment and design presented in the EC1 (2005) applicable to framed buildings less than 200 m tall. This does not however include very slender or unusual structures such as telecommunication masts or similar structures where more detailed calculations may be required to justify their use. The approach discussed is therefore, expected to provide a practical guide to students, young and inexperienced engineers and researchers of how to successfully use the principles and recommendations of the European Wind Code to assess the influence of lateral wind loadings from stand point of both stiffness and stability, strength and serviceability requirements that are structurally safe and satisfactory but also practically economical on buildings subjected to varying topographical and ambient environmental conditions.

2 REVIEW OF WIND AT THE SURFACE BOUNDARY LAYER AND ITS GRADIENT SPEED

According to Onundi (2010), wind is air in motion. The term is usually applied to the movement of air through the atmosphere, resulting from differences in air pressure, which are in turn due to differential atmospheric heating over various parts of the earth. Wind in a vertical motion or nearly vertical direction is called a current. The movement of air near the surface of the earth is three dimensional, with horizontal much greater than the vertical motion and the horizontal motion near the ground surface, are of great importance to building engineering and other structures. Wind speed near the ground varies with terrain roughness. The friction force from terrain
roughness and the concentration or blockage effects from topography influence the atmospheric boundary layer from the ground to the gradient height. Terrain roughness causes a gradual decrease in wind speed toward the ground. In urban areas, this zone of turbulence extends to a height of approximately 366 m above the ground and is called the surface boundary layer. Above this layer, the horizontal airflow is no longer affected by the ground effect. The wind speed at this height is called the gradient windspeed and it is precisely in this boundary layer where most human activity is conducted. Therefore, how wind effects are felt in this zone is of great concern (Taranath, 2005 and Onundi, 2010).

Wind is a phenomenon of great complexity because of the many flow situations arising from the interaction of wind with structures. A significance of turbulence is that dynamic loading on a structure depends on the size of eddies generated. The gustiness of strong winds in the lower levels of the atmosphere largely arises from shear drag with features (i.e. hills, grasses, trees, building etc.). The average wind speed over a period of time in the order of 10 minutes or more tends to increase with height, while the gustiness tends to decrease with height. Structural gustiness decrease with height but vibration increases; therefore, gustiness and vibration are inversely proportional with respect to height (Mendis, et al; 2007). It is because of these varying phenomena that Onundi et al., 2012 concluded that the results of the study of a 72 m building with H/B= 5.538 was static from the base to a height of 0.535 H but beyond that was dynamic.

2.1 Effects of terrain roughness categories and topography on wind speed profile

When the wind speed near the ground varies with terrain roughness, i.e. buildings, trees, etc., and topography. The friction force from terrain roughness and the concentration or obstruction effects from topography have influence the atmospheric boundary layer from the ground to the gradient height. Terrain roughness causes a gradual decrease in wind speed toward the ground. Where the wind speed profile changes with terrain roughness category; the boundary layer depth increases with fetch length, which means that the wind speed profile extends to a higher elevation downstream. In addition, the boundary layer tends to develop faster when the terrain is rougher (AIJ 2004). According to Plate et al., (2002), as reported by Onundi (2010) the structure of the urban atmospheric boundary layer is understood as a multi-layer air flow. For neutral stratification conditions, the open country surrounding the city yields a uniformly adjusted constant stress boundary layer (an equilibrium boundary layer), in which, the wind velocity profile can be expressed, for modest to high (4 m/s and up) wind speed conditions, by a power law equation 2.1 (Plate and Kiefer, 2001):

\[ \frac{u(z)}{u_{ref}} = \left( \frac{z}{d} \right)^{\alpha} \]  ... 2.1

\( u(z) \) is the wind speed at height \( z \), \( u_{ref} \) is the wind speed at a reference height of 10 m where anemometers are placed to collect wind data at airports and \( \alpha \) being the exponent of the power law, which reflects the roughness conditions of the surface upstream of the city. It has been realized from many observation data that the power law exponent becomes greater as the terrain becomes rougher. This exponent usually is of the order of 0.15 to 0.3 (Onundi, 2010) and in the city, the wind profile has to adjust to its roughness. However, it is rare for the terrain roughness to be uniform over a long fetch; therefore, roughness conditions usually vary as a function of many topographical, environmental and surface developmental variations. When the terrain roughness changes suddenly, a new boundary layer develops according to the new terrain roughness which gradually propagates with height, such that wind speeds above this new boundary layer remain unchanged after the roughness change. Thus, the wind speed profile corresponding to the new roughness condition cannot be applied to the high elevation. Between internal and outer boundary layer, a transition region is formed, but if the fetch of constant city roughness extends far enough, - for a 100 m thick boundary layer approximately 1000 m - the outer layer and the internal layer merge into the new, locally adjusted equilibrium boundary layer corresponding to the aerodynamic properties of the city (Plate and Kiefer, 2001).

2.2 Application Range of EC1

Characteristics of EN 1991-1-4 (2005) enables the assessment of wind actions for the structural design of buildings and civil engineering structures up to a height of 200 m. The wind actions are given for the whole or parts of the structure, e.g. components, cladding units and their fixings. The application range of the European wind load standard is much larger than compared to some older national standards. Particularly the specification of wind loads for high-rise buildings and for structures which are susceptible to wind induced vibrations is described in detail.

Table 1.0: Application range of EC1

<table>
<thead>
<tr>
<th>Structure</th>
<th>Limitation of EN 1991-1-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings Height:</td>
<td>max. 200 m</td>
</tr>
<tr>
<td>Viaducts Span:</td>
<td>max. 200 m</td>
</tr>
<tr>
<td>Suspension Bridge/Stay Cable Bridge</td>
<td>Particular Investigations</td>
</tr>
<tr>
<td>Pedestrian Bridge Span:</td>
<td>max. 30 m</td>
</tr>
</tbody>
</table>


The last classification of wind actions is done according to their nature and/or structural response. This classification depends on the response of the structure due to wind actions. For the wind actions, both the quasi-static, dynamic and aero elastic responses were covered (EN 1991, 2005). Due to these different types of wind loading the European wind load standard is subdivided into two parts. The main part gives information and load assumptions for common structures that are not susceptible to wind induced vibrations. Concerning the dynamic response due to turbulence EN 1991-1-4 (2005) covers only the along wind vibration response of a fundamental mode shape with constant sign. For the avoidance of doubts, EN 1991-1-4 (2005) recommended that
the quasi-static response needs to be calculated for all structures, while for most buildings it is not needed to take account of the dynamic and aero elastic response. If particular considerations are necessary for dynamic and aero elastic response wind tunnel tests should be performed.

3 METHODOLOGY

3.1 Wind Assessments and Loadings of the Building

One of the main parameters in the determination of wind actions on structures is the characteristic peak velocity pressure $q_p$. This parameter is in fact the characteristic pressure due to the wind velocity of the undisturbed wind field. The peak wind velocity accounts for the mean wind velocity and a turbulence component. The characteristic peak velocity pressure $q_p$ is influenced by the local wind gust, local factors (e.g. terrain roughness and orography/terrain topography) and the height above terrain.

The wind speed for different regions in Nigeria was achieved by the subdivision of the country into five main categories with a 100 year mean recurrence intervals such that 35 to 42 m/s (Category I), 42 to 45.8 m/s (Category II), 45.8 to 50 m/s (Category III), 50 to 55 m/s (Category IV) and 55 to 60 m/s (Category V) respectively (Onundi et al., 2009 and Onundi 2010).

Hence, the parameters $v_{bk}$ and $v_r$ represent the fundamental and basic wind velocities, and the $v_r$ value in any region within Nigeria can be computed using the expression in equation (0.1)

$$v_r = c_{dr} * c_{mo} * v_{bk}$$  \hspace{1cm} (0.1)

The coefficients $c_{dr}$ and $c_{mo}$ are the directional and seasonal factors. These factors takes into account the changes associated with both wind directions and seasonal changes, and both have effects on the wind velocity ($c_{mo} = c_{dr} = 1$).

However, in cases where the return period for the design defers from $T = 50$ years, a probability factor $c_{prob}$ (see equation (0.2) ) is also taken in to consideration (Bouassida, et al., 2010).

$$c_{prob} = \frac{1 - k \ast \ln(-\ln(1 - p)))^{n}}{1 - k \ast \ln(-\ln(0.98))}$$  \hspace{1cm} (0.2)

Where $k$ and $n$ are the shape parameter depending on the coefficient of variation of the extreme-value distribution and the exponent, respectively. The recommended values for $k$ and $n$ are 0.2 and 0.5 (EC 1, 2005). The basic value of the velocity pressure has to be transformed into a value at the referenced structure height, and this depends on the terrain roughness $c_r(z)$ and topography $c_t(z)$ factors. Hence, the mean wind velocity $v_m(z)$ was determined according to the expression given in equation (0.3).

$$v_m(z) = c_r(z) * c_t(z) * v_r$$  \hspace{1cm} (0.3)

$$c_r(z) = k \ln \left( \frac{z}{z_o} \right) \text{ for } z_{min} \leq z \leq z_{max}$$  \hspace{1cm} (0.4)

For cases where $z < z_{min}$, $c_r(z) = c_r(z_{min})$, and the values are given in Table 1. The terrain factor $k$, depends on the roughness length $z_o$ and is calculated from

$$k = 0.19 \left( \frac{z_o}{z_{0,11}} \right)^{0.07}$$  \hspace{1cm} (0.5)

The $z_{0,11}$ value is equivalent to 0.05 m roughness length and $z_{min}$ length adopted in this study was 200 m. Figure 1 shows the variation of the mean velocity with height of building.

![Figure 1. Variation of wind velocity depending on height z](source: Aachen, 2005 and EC 1, 2005)

<table>
<thead>
<tr>
<th>Terrain Category</th>
<th>Terrain Characteristics</th>
<th>$z_o(m)$</th>
<th>$z_{min}(m)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Sea or coastal area</td>
<td>0.003</td>
<td>1.0</td>
</tr>
<tr>
<td>I</td>
<td>Lakes; no obstacles</td>
<td>0.010</td>
<td>1.0</td>
</tr>
<tr>
<td>II</td>
<td>Low vegetation; isolated obstacles with distances of at least 20 times of obstacle heights</td>
<td>0.050</td>
<td>2.0</td>
</tr>
<tr>
<td>III</td>
<td>Regular vegetation; forests; suburbs; villages</td>
<td>0.030</td>
<td>5.0</td>
</tr>
<tr>
<td>IV</td>
<td>At least 15% of the surface covered with buildings (Av. Building height is 15 m)</td>
<td>1.0</td>
<td>10.0</td>
</tr>
</tbody>
</table>

3.2 Determination of Internal and External Wind Pressures

In EN 1991-1-4 (2005) regulations are given not only for the determination of the external wind pressure $w_e$ on the structure’s cladding but also for the application of the internal wind pressure $w_i$ in case of openings (exposure factor $c_r(z)$, ...
external coefficient $c_{pe}(z)$ and internal coefficient $c_{pi}(z)$.

Hence, equation (0.6) shows the appropriate function.

$$w_{e} = q_{pe} * c_{e}(z) * c_{pe}(z)$$

$$w_{i} = q_{pi} * c_{i}(z) * c_{pi}(z)$$

(0.6)

### 3.3 Determination of the Wind Induced Force

The resulting wind force can be determined by integration of the wind pressure over the whole surface or by applying appropriate force coefficients that are given in EN 1991-1-4 for different kinds of structures. It is noted here, that for many structures force coefficients result into more accurate results than integration of pressure coefficients. The wind force $F_{w}$ is determined using equation (0.7)

$$F_{w} = c_{a} * c_{d} * c_{f} * q_{p}(z) * A_{ref}$$

(0.7)

Where $c_{a}c_{d}$ is the structural factor which is $\geq 1.0$ depending on the $v_{m}(z)$ value, but taken as 1.0 for building height $< 15$ m. similarly, $c_{f}$ and $A_{ref}$ represents the force coefficient and the reference area for the structure, respectively. In EN 1991-1-4, for rectangular and/or polygonal shapes, the $c_{f}$ value is by

$$c_{f} = c_{fo} * \Psi_{r} * \Psi_{i}$$

(0.8)

The coefficients $c_{fo}$ and $\Psi_{r}$ are the force coefficient for shapes with sharp corners and reduction factor for rounded corners in a rectangular structures. Similarly, $\Psi_{i}$ represents the end-effect factor. For all elements without free-end flow the recommended $c_{fo}$ value is 2.0, and it’s assumed to be safest (EC 1, 2005).

### 3.4 An Analytical Example

The proposed building was assumed to be situated on a relatively flat terrain in an open area in Maiduguri, Borno state in Nigeria where it is exposed to winds blowing from all directions. The local prevailing wind speed of 47m/s for Maiduguri with 100-year mean recurrence intervals (Omundi 2010). The model was evaluated for flexural rigidity of shear walls and network of frames Figure 2 and 3.0, (12.20 m x 55.20 m x and 60 m, 20 storey building) for aerodynamic resistance of medium rise multi-storey building subjected to wind loadings. The procedures for the estimation of the characteristic wind load on the building were carried out in accordance with the EC1 (2005; Part 1-4) and ETABS 2015 software packages used for the analysis.

### 3.5 Three Dimensional Modelling of the 60 m (20 Storey-height)

The modelling of the 60 m, 20 Storey Building as 3D-Space Frame structure was with ETABS 2015 (version-15.22) Software (Figure 2). The model was created in such a way that the different structural components represent as accurately as possible the characteristics such as mass, strength, stiffness and deformability of the structure. However, other none-structural components were not modelled due to some limitations, but the loading effects were incorporated in to the design. The networks of framed members (beams and columns) were modelled through the assignment of properties such as the type of material used, cross sectional area, and reinforcement details. The Slab was considered as shell element, and constraints in the form of rigidity of the diaphragm for each floor have been used for the analysis for providing stiffness in all directions and transfer mass of slab to columns and beams. All shear walls in the building were modelled as pier elements and are considered as slender with wall height-to-length ratio well above 3 and therefore aerodynamic response of the shear walls is expected to be dominated by flexure.

Figure 2. Typical Plan and section of the Building Model with Shear walls arrangement (ETABS)
3.6 Application of loading on the Building Model

The loadings are applied on the building model in accordance with the provision of EN 1990, EN 1991-1-1 and EN 1991-1-4, the two basic loadings are permanent and variable actions. The permanent actions comprise the self-weight of the structural members as dead load. The ETABS automatically generate the self-weight of the permanent action on the structure. The variable actions consist of imposed load and wind loadings. The wind loads represent the critical loading condition to which the building is subjected, considering the peculiar nature of wind characteristics of the Maiduguri in Nigeria. With regards to wind action, EN 1991-1-4 explicitly provide guidance on how to estimate wind forces on structures. Table 2 contains the detailed procedure for the determination of the wind forces on the building.

Table 2. Design wind speed with corresponding wind pressure for various Zones (Onundi et al., 2009)

<table>
<thead>
<tr>
<th>Zones</th>
<th>Zone 1</th>
<th>Zone 2</th>
<th>Zone 3</th>
<th>Zone 4</th>
<th>Zone 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind speed, (m/s)</td>
<td>35 - 42</td>
<td>42 - 45.8</td>
<td>45.8 - 50</td>
<td>50 - 55</td>
<td>55 - 56</td>
</tr>
<tr>
<td>Wind Pressure, q (kN/m²)</td>
<td>0.941-1.088</td>
<td>1.088-1.283</td>
<td>1.283-1.535</td>
<td>1.535-1.855</td>
<td>1.855-1.925</td>
</tr>
</tbody>
</table>

Figure 3. The building 3D-model of the 60 m length, 20-storey height

Figure 4. Classification of Nigeria into Wind Speeds Isopleths Zones (Onundi et al., 2009)
Table 3. Procedure for assessment of the aerodynamic wind load on the 60 m, 20 storey building

<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>CALCULATIONS</th>
<th>OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isopleths Map Table 3.0, Figure 3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Summary Building Data</strong></td>
<td>Location of building: Maiduguri on a relatively flat terrain in an open area.</td>
<td></td>
</tr>
<tr>
<td>Height of Building: 60 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length: 55.2 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width: 12.2 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storey Height: 3 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>This is the procedure of determination of loads on the high rise multi-storey building for the fundamental value of the basic wind velocity $v_{h,o} = 47 m/s$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$v_{h,o} = 47 m/s$ [see isopleths map of Nigeria]</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Basic wind velocity, $V_b$</strong></td>
<td>$v_b = c_{dir} \cdot c_{sea} \cdot v_{h,o}$ having both $c_{dir} = c_{sea} = 1.0$</td>
<td></td>
</tr>
<tr>
<td>For simplification, the directional factor $c_{dir}$ and the seasonal factor $c_{sea}$ are in general equal to 1.0.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$v_b = 1<em>1</em>47 = 47 m/s$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>But for this study where the return period is 100 year mean recurrence intervals $T = 100$ is considered as the duration of the design life, which should lead to $c_{prob} &gt; 1.0$. The probability factor $c_{prob}$ is given as:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c_{prob} = \left( \frac{\ln(\ln(1-p))}{\ln(\ln(0.98))} \right)^n$</td>
<td>$c_{prob}=1.04$</td>
<td></td>
</tr>
<tr>
<td>$p = 1/100 = 0.01$ (100 year mean recurrence intervals)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c_{prob} = \left( \frac{1-0.2 \cdot \ln(-\ln(1-0.01))}{1-0.2 \cdot \ln(-\ln(0.98))} \right)^{0.5} = 1.04$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The recommended values for K and n are 0.2 and 0.5, respectively.</td>
<td></td>
<td></td>
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<tr>
<td>Therefore,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$v_b = c_{prob} \cdot c_{dir} \cdot c_{sea} \cdot v_{h,o} = 47 m/s$</td>
<td>$v_b = 48.81 m/s$</td>
<td></td>
</tr>
<tr>
<td>$v_b = 1.04 \cdot 1 \cdot 1 \cdot 47 = 48.81 m/s$</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mean wind velocity, $v_m (z)$</strong></td>
<td>$v_m (z) = c_r (z) \cdot c_s (z) \cdot v_b$</td>
<td></td>
</tr>
<tr>
<td>Terrain orography:</td>
<td>$c_r (z) = 1$</td>
<td></td>
</tr>
<tr>
<td>$c_s (z) = 1$ [for flat terrain $c_s (z) = 1.0$]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[for other types of terrain see section 4.3.3 &amp; Annex A.3]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terrain category</td>
<td>Category: II</td>
<td></td>
</tr>
<tr>
<td>Category: II</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$z_0 = 0.05$ m; roughness lengths</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$z_{min} = 2$ m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$z_{max} = 200$ m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roughness Factor, $&lt;c_r(z)$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Table 4.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 4.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eq. 4.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EC 1, (2005)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sect. 4.3.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c_r (z) = k_r \ln \left( \frac{z}{z_0} \right)$ for $z_{min} \leq z \leq z_{max}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$k_r = 0.19$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Reference Calculations

<table>
<thead>
<tr>
<th>Sect. 4.3.2</th>
<th>Equ. 4.4</th>
<th>EC1, (2005),</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_r(z) = c_r(z_{\text{min}})$ for $z &lt; z_{\text{min}}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terrain Factor, $k_r$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$k_r = 0.19 \times \left( \frac{z}{z_0} \right)^{0.07} = 0.19$, because $z_{\text{ref}} = 0.05$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The terrain category of the Building location falls in category (II) For terrain, roughness length $z_0 = 0.05$ and $z_{\text{ref}} = 0.05$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With this value of $k_r$, the roughness factor $C_r(z)$ is then calculated for varying heights $Z$ from 3 m to 60 m at an intervals of 3 m.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thus, the mean wind velocities at these heights are also obtained for the various height levels.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Wind Turbulence Intensity at a Height $z$,

$$k_i = 1$$ turbulence factor recommended value $1.0$

$$I_v(z) = \frac{k_i}{c_a(z) \ln \left( \frac{z}{z_0} \right)} = \frac{\sigma_v}{V_m(z)} \text{ for } z_{\text{min}} \leq z \leq z_{\text{max}}$$

$I_v(z) = I_v(z_{\text{min}})$ for $z < z_{\text{min}}$

- standard deviation of the turbulence, $\sigma_v = k_c \times v_\text{h} \times k_i$

- $\rho = 1.25 \text{ kg/m}^3$

- $q_p = 1488.9 \text{ N/m}^2$

#### Peak Velocity Pressure at Height $z$, $q_p$

Basic velocity pressure

$$q_p(z) = c_r(z) \times q_b$$

$$c_r(z) = [1 + 7 \times I_v(z)] \times c_r(z) \times c_r(z) = q_p(z) / q_b$$ and

$$q_b = 0.5 \times \rho \times v_h^2 = 1488.91 \text{ N/m}^2$$

- $\rho$ is the air density, and has a value of 1.25 kg/m$^3$.

#### Wind Pressure on the Building, $w_e$

The wind pressure acting on the external surfaces, $w_e$,

$$w_e = q_p(z) \times c_{pe}$$

$$w = q_p(z) \times c_a \left( c_{p,\text{wind}} + c_{p,\text{lee}} \right)$$

**Vertical Wall**

$h = 60 \text{ m, Height of the building}$

$b = 55.2 \text{ m, cross wind dimension}$

For $h/b = 60/55.2 = 1.0869$

- D side: $c_{pe} = +0.8$

- E side: $c_{pe} = -0.5$

(coefficients for the windward and leeward)

#### The Force Coefficient $c_f$ Value Determination

The force coefficient $c_f$ for the rectangular section with the wind blowing normally to a face is

$$c_f = c_{pe} \times \Psi_r \times \Psi_f$$

From Figure 7.36 — Indicative values of the end-effect factor $\lambda$ as a function of solidity ratio $\phi$ versus $\psi$ slenderness $\lambda$ (EC 1, 2005), the solidity ratio $\phi$ is given by
\[ \phi = \frac{A}{A_c} \]

where

A is the sum of the projected areas of the members

\[ A_c = L \times b = \Sigma A = A_{nc} \]

therefore, Using Figure 7.36 and Table 7.16 the value of \( \psi \), is determine for the calculated solidity ratio. Considering Structural elements with sharp edged section

The solidity ratio \( \phi = \frac{A_{nc}}{A_c} = 1 \), and this gives the end-factor value \( \psi = 0.68 \)

Then, the force coefficient of the structural elements with sharp edged section is now

\[ c_f = c_{f0} \times \psi \]

\( c_{f0} \) is the force coefficient of rectangular section with sharp corners and without free-end flow, and this is determined by the building dimension behaviour as follows

\[ \frac{d}{b} = 55.2/12.2 = 4.52 \]

\( c_{f0} = 1.104 \)

\[ c_f = c_{f0} \times \psi = 1.1 \times 0.68 = 0.748 \]

The Building corresponds to the recommended shape of Figure 6.1(a) of clause 6.3.1, thus this expression was used:

\[ C_a C_d = \frac{1 + (2 \times k_p \times I_x(Z_s) \times \sqrt{B^2 + R^2})}{1 + (7 \times I_c(Z_s))} \]

where

\( z_r \) is the reference height for determining the structural factor, and is equal to

\[ 0.6h = 0.6 \times 60 = 36m \]

The corresponding turbulence intensity at this height is:

\[ l_t(36m) = 0.152 \]

\[ B^2 = \frac{1}{1 + 0.9 \times (b + h)/L(Z_s)^{0.63}} \]

\( b = 12.2 \) m, the width of the structure, \( h = 36 \) m, the turbulent length scale at height \( z_r \)

\[ L(x) = L_s (z_r/z_s)^{\alpha} \]

for \( z > z_{min} \)

Given \( z_s = 200m, L_s = 300m, \alpha = 0.67 + 0.05 \ln(z_s) = 0.52 \)

\[ L(36) = 300(36/300)^{0.52} = 99.56 \]

\[ B^2 = \frac{1}{1 + 0.9 \times (5.02 + 60)/(99.5632)^{0.63}} = 0.218 \]

The resonance response factor, \( R^2 = \frac{\pi^2 \times S(t) \times R_b(\eta_b) \times R_b(\eta_b)}{20} \)

The non-dimensional power spectral density function,

\[ S_L(z,n) = \frac{n \times S_r(z,n)}{\sigma^2} = \frac{6.8 \times f_L(Z_s n_{1x})}{\left(1 + 10.2 \times f_L(Z_s n_{1x})\right)^{2/3}} \]

non-dimensional frequency,

\[ l_t(36m) = 0.152 \]
The fundamental flexural frequency \( n_1 \) of multi-storey buildings for a height \( > 50 \) m can be estimated using:

\[
n_1 = \frac{46}{h} = \frac{46}{60} = 0.767 \text{ Hz} < 1
\]

The aerodynamic admittance functions were obtained using:

\[
\eta_h = 4.6 \times h \times f_L(Z, n_1, x) = 4.6 \times 60 \times 1.299 = 3.602
\]

\[
\eta_b = 4.6 \times b \times f_L(Z, n_1, x) = 4.6 \times 12.2 \times 1.299 = 0.732
\]

Logarithmic decrement of damping, \( \delta \) was estimated by:

\[
\delta = \delta_s + \delta_a + \delta_d
\]

where

\[
\delta_s = \text{the logarithmic decrement of structural damping} = 0.10
\]

\[
\delta_a = \text{the logarithmic decrement of damping due to special devices} = 0
\]

\[
\delta_d = \text{the logarithmic decrement of aerodynamic damping for along wind vibrations, this may be estimated by:}
\]

\[
\delta_d = \frac{c_f \times \rho \times b \times v_m(z)}{2 \times n_1 \times m_e}
\]

Where, \( m_e \) represents the mass per unit length over the upper third of the structure; i.e. \( H / 3 = 60 / 3 = 20m \) from the building top. Hence, from the Building information, average mass \( h_f = 4863.86kg \).

The equivalent mass per unit length, \( m_e \) value is

\[
m_e = 4863.86 / 3 = 231.612 \text{ kg/m}
\]

, with \( c_f = 0.748 \) then

\[
\delta_a = \frac{1.36 \times 1.25 \times 12.2 \times 58.75}{2 \times 0.767 \times 231.61} = 1.887
\]

Therefore,

\[
\delta = \delta_s + \delta_a + \delta_d = 0.03 + 3.43111 = 1.987
\]

Now, the resonance response factor, \( R \)

\[
R^2 = \frac{\pi^2 \times 0.106 \times 0.258 \times 1.007}{2 \times 3.461} = 0.072
\]

The peak factor, \( K_p \)

\[
K_p = \left( \sqrt{2} \ln v \times T \right) + \frac{0.6}{\left( \sqrt{2} \ln v \times T \right)} \text{ or } K_b = 3 \text{ whichever is larger}
\]

Where

\[
f_L(z, n) = \frac{n \times L(z)}{v_m(z)}
\]

\[
S_L(z, n) = \frac{1}{1 + 10.2 \times 1.299}^{5/3}
\]

\[
\eta_b = \frac{4.6 \times 12 \times 1.299}{99.56} = 0.732
\]

Anx. B.3 (2)
EC1,(2005).

Anx. B.1 (1)
EC1,(2005).

Equ.F.2 Anx F
EC 1, 2005.
The up-crossing frequency, \( v \)

\[
v = \frac{n_b \sqrt{R^2}}{\sqrt{B^2 + R^2}} = \frac{0.767 \sqrt{(0.395)}}{\sqrt{(0.218^2 + 0.072^2)}} = 0.381, \quad v \geq 0.08 \text{ Hz}
\]

The limit of \( v \geq 0.08 \) Hz corresponds to a peak factor of 3.0.

Hence,

\[
K_p = \left( \sqrt{2 \ln(0.381 \times 600)} \right) + \frac{0.6}{\sqrt{2 \ln(0.381 \times 600)}} = 3.323
\]

Thus, the structural factor, \( c_a c_d \) for the fundamental wind velocity of 47 m/s is:

\[
c_a c_d = \frac{1 + (2 \times 3.323 \times 0.152 \times 0.218^2 + 0.072^2)}{1 + (7 \times 0.152)} = 1.1
\]

Hence, for framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth, the \( c_a c_d \) value may be taken as 1.0 as stipulated by (clause 6.2 (c), EC 1, 2005). For buildings with \( h/d > 5 \), the total wind loading may be based on the provisions given in 7.6 to 7.8 and 7.9.2. (Note 2 clause 7.2.2 EC 1, 2005). But for this study the building ratio \( h/d = 4.92 < 5 \) and \( c_a c_d = 1.0 \)

Therefore,

Wind load, \( w \)

\[
w = q_p(z)c_a c_d (c_{p,\text{wind}} + c_{p,\text{lee}}) \times A
\]

Accordingly, applied wind loads for the various height level of the building are obtained. This is also consistent with the automatically generated lateral wind loads for load pattern Wind according to EC1, (2005), as calculated by ETABS package. These results are presented under the results and discussion section of this paper.

### 4 RESULTS AND DISCUSSION

Table 4 presents the results of the lateral wind loads generated in accordance with the dictates of the EN 1991-1-4 (2005) code of practice. The clear pictorial representation is represented using Figure 5 for the applied wind forces at different storey levels under both the software and the corresponding manual computations for the critical Y-direction.

Table 4. Lateral Wind Forces of the Building Model

<table>
<thead>
<tr>
<th>Story</th>
<th>Elevation (m)</th>
<th>ETABS</th>
<th>Manual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X (kN)</td>
<td>Y (kN)</td>
<td>Y-Direction</td>
</tr>
<tr>
<td>Story20</td>
<td>60</td>
<td>74.945</td>
<td>332.7557</td>
</tr>
<tr>
<td>Story19</td>
<td>57</td>
<td>148.7897</td>
<td>660.6264</td>
</tr>
<tr>
<td>Story18</td>
<td>54</td>
<td>147.0774</td>
<td>653.0237</td>
</tr>
<tr>
<td>Story17</td>
<td>51</td>
<td>145.2767</td>
<td>645.0285</td>
</tr>
<tr>
<td>Story16</td>
<td>48</td>
<td>143.3774</td>
<td>636.5956</td>
</tr>
<tr>
<td>Story15</td>
<td>45</td>
<td>141.3675</td>
<td>627.6718</td>
</tr>
<tr>
<td>Story14</td>
<td>42</td>
<td>139.2326</td>
<td>618.1928</td>
</tr>
<tr>
<td>Story13</td>
<td>39</td>
<td>136.9552</td>
<td>608.081</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Story</th>
<th>Elevation (m)</th>
<th>X (kN)</th>
<th>Y (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story12</td>
<td>36</td>
<td>134.5137</td>
<td>597.2406</td>
</tr>
<tr>
<td>Story11</td>
<td>33</td>
<td>131.881</td>
<td>585.5517</td>
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<tr>
<td>Story10</td>
<td>30</td>
<td>129.0229</td>
<td>572.8616</td>
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<tr>
<td>Story9</td>
<td>27</td>
<td>125.8943</td>
<td>558.9709</td>
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<tr>
<td>Story8</td>
<td>24</td>
<td>122.4352</td>
<td>543.6124</td>
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<td>Story7</td>
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<td>118.562</td>
<td>526.4154</td>
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<td>Story6</td>
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<tr>
<td>Story5</td>
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<td>109.0261</td>
<td>484.076</td>
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<tr>
<td>Story4</td>
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<td>102.8723</td>
<td>456.753</td>
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<td>Story3</td>
<td>9</td>
<td>95.1238</td>
<td>422.3498</td>
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<tr>
<td>Story2</td>
<td>6</td>
<td>84.4957</td>
<td>375.1608</td>
</tr>
<tr>
<td>Story1</td>
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<td>68.4917</td>
<td>304.1032</td>
</tr>
<tr>
<td>Base</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

As observed from Figure 5, the variation of the lateral wind load applied to the model building generally increases logarithmically with height from the bottom to the top of the building.
modelled building. This behaviour corroborates what was stipulated in EC 1, 2005, and other literature results similarly show same (Mendis et al.; 2007, Taranath, 2005 and Smith & Coull, 1991).

Figure 5. Wind Forces result Profile at different storey-heights (a) Manual (b) ETABS

Table 5. t-test statistical analysis

<table>
<thead>
<tr>
<th></th>
<th>Manual</th>
<th>ETABS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>510.2816</td>
<td>505.3669</td>
</tr>
<tr>
<td>Variance</td>
<td>24846.61</td>
<td>23734.16</td>
</tr>
<tr>
<td>Observations</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>Pearson Correlation</td>
<td>0.999532</td>
<td></td>
</tr>
<tr>
<td>Hypothesized Mean Difference</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>df</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>t Stat</td>
<td>3.781704</td>
<td></td>
</tr>
<tr>
<td>P(T&lt;=t) one-tail</td>
<td>0.000586</td>
<td></td>
</tr>
<tr>
<td>t Critical one-tail</td>
<td>1.724718</td>
<td></td>
</tr>
<tr>
<td>P(T&lt;=t) two-tail</td>
<td>0.001171</td>
<td></td>
</tr>
<tr>
<td>t Critical two-tail</td>
<td>2.085963</td>
<td></td>
</tr>
</tbody>
</table>

Turbulence Intensity, \( I_v \) was derived from equation \( I_v = \frac{k_l}{c_l(z)\ln(Z)} \) for \( z_{min} \leq z \leq z_{max} \) significantly influences the global profiles of wind loadings along the model height. Apart from this, the values generated from both the manually computed and the software converged satisfactorily since a negligible percentage difference of less than 2% was observed. The statistical analysis (Table 5) supports the argument, where the \( P-value \) is less than 0.05 and this indicates insignificant difference between the two variables.

5 CONCLUSION

High-rise buildings are specifically characterised by the wind load acting on it, and there computations could be conservative or otherwise. This conservativeness could be due to several reasons, and one such factor might be associated to approximation in either the use of design software or other forms of non-software-based computations. Certainly, because of the complication rigors associated in the wind-load calculation acting on high-rise building, the use of computing and analysis software becomes a necessity but it requires utmost care and skills to handle it. However, sometimes the
manual computation is also very necessary if not for anything
to confirm the resulting output from the use of software that
we do not have control over it. Most cases the software users
have little or no access to the main program frame; it is highly
probable that the resulting outcomes from such interfaces
might lead to erroneous results and possible under or over
estimations. To address this challenge, this paper provides an
assessment for high-rise building wind design using both the
software and other computation method. Interestingly, the
resulting outputs from the use of ETABS and the simplified
approach show insignificant variation between them because
the p-value from the statistical analysis is less than 0.05.

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