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 in to five main categories with a 100 year mean recurrence intervals such that 35 to 42 ms^{-1}(Category I), 42 to 45.8 ms^{-1} (Category II), 45.8 to 50 ms^{-1} (Category III), 50 to 55 ms^{-1}(Category IV) and 55 to 56 ms^{-1}(Category V) respectively(Onundi *et al.,* 2009and Onundi 2010).

Hence, the parameters $v_{h.0}$ and v_b represent the fundamental and basic wind velocities, and the v_b value in any region within Nigeria can be computed using the expression in equation (0.1)

$$v_b = c_{dir} * c_{sea} * v_{h.0} \tag{0.1}$$

The coefficients c_{dir} and c_{sea} 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 ($c_{sea} = c_{dir}$ =1). However, in cases where the return period for the design defers from T = 50 years, a probability factor c_{prob} c_{prob} (see equation (0.2)) is also taken in to consideration (Bouassida, *et al*., 2010).

$$c_{prob} = \left(\frac{1 - k * In(-In(1-p))}{1 - k * In(-In(0.98))} \right)^n \tag{0.2}$$

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 $c_o(z)$ factors. Hence, the mean wind velocity $v_m(z)$ was determined according to the expression given in equation (0.3).

$$v_m(z) = c_r(z) * c_o(z) * v_b \tag{0.3}$$

$$c_r(z) = k_r In\left(\frac{z}{z_o}\right) \text{ for } z_{min} \le z \le z_{max} \tag{0.4}$$

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_r depends on the roughness length z_o and is calculated from

$$k_r = 0.19 * \left(z_o / z_{o.ii} \right)^{0.07} \tag{0.5}$$

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

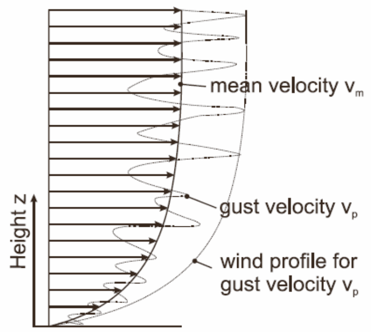


Figure 1. Variation of wind velocity depending on height z
Source: (Aachen, 2005 and EC 1, 2005)

Table 1. Terrain categories (EC 1, 2005)

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

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 $c_e(z)$, external coefficient $c_{pe}(z)$ and internal coefficient $c_{pi}(z)$). Hence, equation (0.6) shows the appropriate function.

$$\begin{aligned} w_e &= q_b * c_e(z) * c_{pe}(z) \\ w_i &= q_b * c_e(z_i) * c_{pi}(z) \end{aligned} \tag{0.6}$$

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)

$$F_w = c_a c_d * c_f * q_p(z_e) * A_{ref} \tag{0.7}$$

Where $c_s c_d$ is the structural factor which is ≥ 1.0 depending on the $v_m(z)$ value, but taken as 1.0 for building height $\prec$ 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

$$c_f = c_{fo} * \Psi_r * \Psi_i \tag{0.8}$$

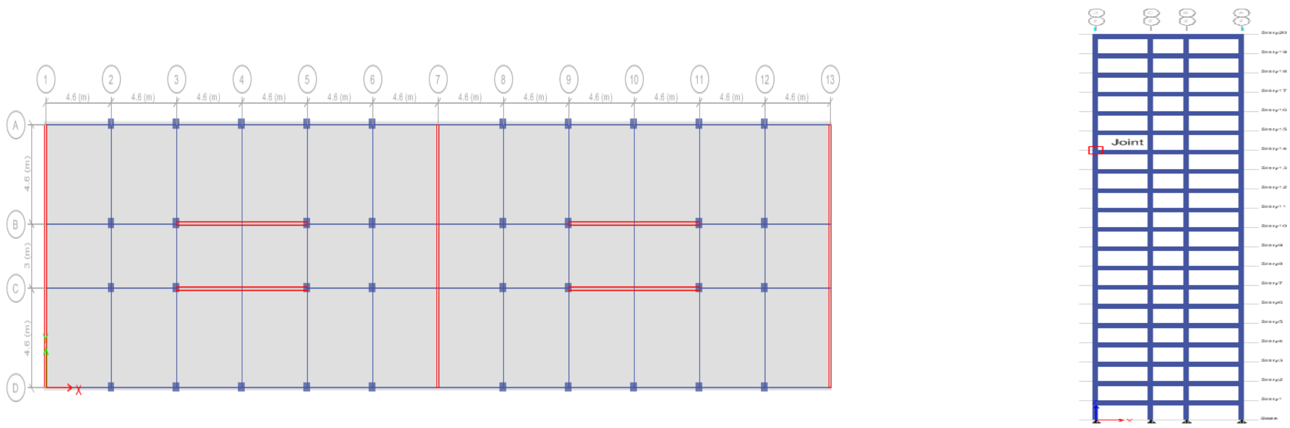
The coefficients c_{fo} and Ψ_r are the force coefficient for shapes with sharp corners and reduction factor for rounded corners in a rectangular structures. Similarly, Ψ_i represents the end-effect factor. For all elements without free-end flow the recommended c_{fo} value is 2.0, and it's assumed to 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 (Onundi 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 into 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.



Plan

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

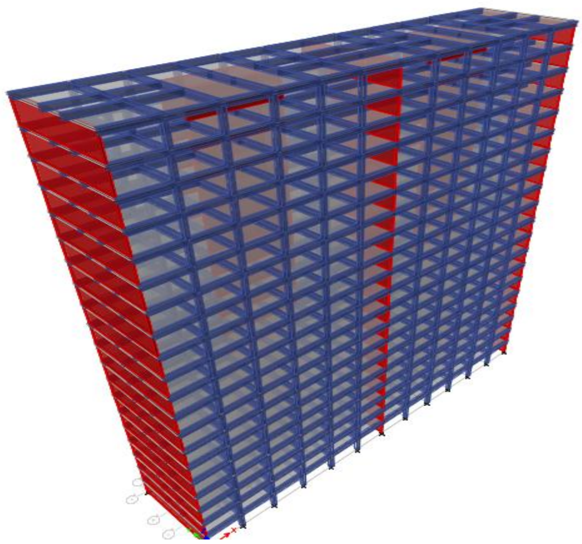


Figure 3. The building 3D-model of the 60 m length, 20-storey height

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** 3 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)

Zones	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Wind speed, (m/s)	35 - 42	42 - 45.8	45.8 - 50	50 - 55	55 - 56
Wind Pressure, q (kN/m^2)	0.941-1.088	1.088-1.283	1.283-1.535	1.535 - 1.855	1.855-1.925

or buildings, while EN 1991-1-4 also direct that provision of EN 1990 on loading should be considered. The specifications of the codes were followed to obtain the wind forces on the building model, ensuring that all specified parameters in the codes are calculated appropriately. The building model was loaded with the design wind speed of 47 m/s to represents the critical limits of the wind speed for Maiduguri shown in the **Figure 4** as obtained in the literature (Onundi*et al.*, 2009). The wind speed is applied orthogonally to the face of the building model, and **Table 2** shows the various zones designated design wind speed and the corresponding wind pressure. Consequently,

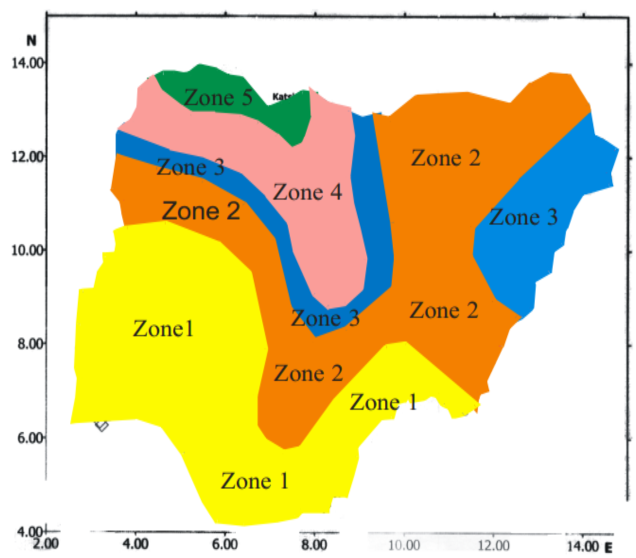
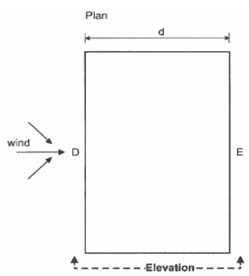


Figure 4. Classification of Nigeria into Wind Speeds Isopleths Zones (Onundi *et al.;*, 2009)

Table 3. Procedure for assessment of the aerodynamic wind load on the 60 m, 20 storey building

REFERENCE	CALCULATIONS	OUTPUT
	Summary Building Data Location of building: Maiduguri on a relatively flat terrain in an open area. Height of Building: 60 m Length: 55.2 m Width: 12.2 m Storey Height: 3 m This is the procedure of determination of loads on the high rise multi-storey building for the fundamental value of the basic wind velocity $v_{h.o} = 47m/s$	
Isopleths Map Table 3.0, Figure 3.0	$v_{h.o} = 47m/s$ [see isopleths map of Nigeria]	
	Basic wind velocity, V_b $v_b = c_{dir} * c_{sea} * v_{h.0}$ having both $c_{dir} = c_{sea} = 1.0_0$	
Sect.4.2 Note 2 & 3 EC1, 2005	For simplification, the directional factor c_{dir} and the seasonal factor c_{sea} are in general equal to 1.0. $v_b = 1*1*47 = 47m/s$ 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 $c_{prob}> 1.0$. The probability factor c_{prob} is given as: $$c_{prob} = \left(\frac{1 - k * In(-In(1-p))}{1 - k * In(-In(0.98))} \right)^n$$	
Equ. 4.2 EC 1,(2005)	p = 1/100 = 0.01 (100 year mean recurrence intervals) $$c_{prob} = \left(\frac{1 - 0.2 * In(-In(1-0.01))}{1 - 0.2 * In(-In(0.98))} \right)^{0.5} = 1.04$$ The recommended values for K and n are 0.2 and 0.5, respectively. Therefore, $v_b = c_{prob} * c_{dir} * c_{sea} * v_{h.o} = 47m/s$ $v_b = 1.04*1*1*47 = 48.81m/s$	c_{prob}=1.04 v_b= 48.81m/s
4.2(2) (5) EC1,(2005)	**Mean wind velocity, $v_m(z)$** $v_m(z) = c_r(z) * c_o(z) * v_b$ Terrain orography: c_o (z) = 1 [for flat terrain c_o (z) = 1.0] [for other types of terrain see section 4.3.3 & Annex A.3]	c_o (z) = 1
Sect. 4.3 Eq. 4.3 EC1, (2005) Sect. 4.3.3 A.3	Terrain category Category: II z_0 = 0.05 m; roughness lengths z_{min}= 2 m z_{max} = 200 m Roughness Factor, $<c_{r>(z)}$	Category: II z_0 = 0.05 m z_{min}= 2 m z_{max} = 200 m
Table 4.1 EC1, (2005)	$c_r(z) = k_r In\left(\frac{z}{z_o}\right)$ for $z_{min} \le z \le z_{max}$	k_r = 0.19

REFERENCE	CALCULATIONS	OUTPUT
Sect.4.3.2 Equ.4.4 EC1,(2005),	$c_r(z) = c_r(z_{\min})$ for $z \prec z_{\min}$ Terrain Factor, k_r $k_r = 0.19 * \left(\frac{z}{z_{o.ii}}\right)^{0.07} = 0.19$, because $z_{o.ii} = 0.05$ The terrain category of the Building location falls in category (II) For terrain, roughness length z_0=0.05 and $z_{0,ii}$ = 0.05 With this value of Kr, the roughness factor Cr (z) is then calculated for varying heights Z from 3 m to 60 m at an intervals of 3 m. Thus, the mean wind velocities at these heights are also obtained for the various height levels.	
	Wind turbulence intensity at a height z, $k_l = 1$ turbulence factor recommended value 1.0 $I_v(z) = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})} = \frac{\sigma_v}{V_m(z)}$ for $z_{\min} \le z \le z_{\max}$ $I_v(z) = I_v(z_{\min})$ for $z < z_{\min}$ standard deviation of the turbulence, $\sigma_v = k_r \text{ x } v_b \text{ x } k_l$ $\sigma_v = 0.19 \text{ x } 48.81 \text{ x } 1 = 9.27$ m/s The intensity is then calculated along the Building height for each storey level.	$k_l = 1$ $\sigma_v = 9.27$m/s
Sect.4.4 (1) EC 1,(2005)	**Peak velocity pressure at height z, q_p** Basic velocity pressure	$\rho = 1.25$ Kg/m^3 q_b = 1488.9N/m^2
Sect.4.4 (1) Equ. 4.7 EC1,(2005)	$q_p(z) = c_e(z) * q_b$ $c_e(z) = [1 + 7 * I_v(z)] * c_r(z) * c_o(z) \equiv q_p(z) / q_b$ and $q_b = 0.5 * \rho * v_b^{\,2} = 1488.91 N/m^2$	
Eq. 4.5 Eq. 4.8 EC1,(2005)	ρ is the air density, and has a value of 1.25 kg/m^3.	
4.5	**Wind pressure on the Building, w_e** The wind pressure acting on the external surfaces, w_e, $w_e = q_p(z) * c_{pe}$ $w = q_p(z) * c_a c_d (c_{p.wind} + c_{p.lee})$	
Eq. 4.9 EC1,(2005)	*Vertical Wall* h = 60 m, Height of the building b = 55.2 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) 	
Sect. 5.2 Equ.5.1 EC1,(2005)	**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 by $c_f = c_{fo} * \Psi_r * \Psi_i$ From Figure 7.36 — Indicative values of the end-effect factor λ as a function of solidity ratio ϕ versus ψ slenderness λ (EC 1, 2005), the solidity ratio ϕ is given by	

REFERENCE	CALCULATIONS	OUTPUT
Table 7.1 & figure 7.5 EC1, (2005) Sec.7.1.1(1) EC1, (2005) Equ.7.9 EC 1,2005 Eq. (7.28) EC 1, 2005 Figure 7.36 & Table.7.16	$\phi = A/A_c$ where A is the sum of the projected areas of the members A_c is the overall envelope area, $A_c = L*b = \sum A = A_{ref}$, therefore, Using Figure 7.36 and Table 7.16 the value of ψ_λ is determine for the calculated solidity ratio. Considering Structural elements with sharp edged section The solidity ratio ϕ $=A_{ref}/A_c = 1$, and this gives the end-factor value $\psi_\lambda = 0.68$ Then, the force coefficient of the structural elements with sharp edged section is now $c_f = c_{fo} * \Psi_\lambda$ c_{fo} 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 $d/b = 55.2/12.2 = 4.52$ $c_{fo} = 1.104$ $c_f = c_{fo} * \Psi_\lambda = 1.1 * 0.68 = 0.748$ *The factor* $c_a c_d$ The Building corresponds to the recommended shape of Figure 6.1(a) of clause 6.3.1, thus this expression was used: $C_a C_d = \frac{1 + (2 * k_p * I_v(Z_s) * \sqrt{B^2 + R^2}}{1 + (7 * I_v(Z_s)}$ where z_s is the reference height for determining the structural factor, and is equal to $0.6h = 0.6*60 = 36m$ The corresponding turbulence intensity at this height is: $l_v(36m) = 0.152$ $B^2 = \frac{1}{1 + 0.9 * (b+h)/L(Z_s)^{0.63}}$ the width of the structure, b = 12.2 m the turbulent length scale at height z_s $L(z_s) = L_t\left(z_s/z_t\right)^\alpha$ for $z \succ z_{min}$ Given $z_t = 200m,\ L_t = 300m,\ \alpha = 0.67 + 0.05In(z_o) = 0.52$ $L(36) = 300(36/300)^{0.52} = 99.56$ $B^2 = \frac{1}{1 + 0.9 * (5.02 + 60)/(99.5632)^{0.63}} = 0.218$ The resonance response factor , $R^2 = \frac{\pi^2 \text{ x } SL(Z_s n_{n1,x}) \text{x } R_h(\eta_h) \text{x } R_b(\eta_b)}{2\delta}$ The non-dimensional power spectral density function, $S_L(z,n) = \frac{n * S_s(z,n)}{\sigma_v{}^2} = \frac{6.8 * f_L(Z_s n_{1,} x)}{\left(1 + 10.2 * f_L(Z_s n_{n1,x})\right)^{5/3}}$ non-dimensional frequency,	$\phi = 1$ $c_{f,0} = 1.104$ $l_v(36\ m) = 0.152$

REFERENCE	CALCULATIONS	OUTPUT
Figure.7.23EC 1,2005.	$f_L(z,n) = \frac{n * L(z)}{\mathrm{v_m(z)}}$ The fundamental flexural frequency n_1 of multi-storey buildings for a height $\succ$ 50 m can be estimated using: $n_1 = 46/h = 46/60 = 0.767\ Hz \prec 1$	
Sect. 6.3.1 EC 1,2005	$f_L(z,n) = \frac{0.767 * 99.56}{58.75} = 1.299$	$n_{1,} = 0.767$Hz
Fig 6.1 (a) EC1,(2005).	$S_L(z,n) = \frac{6.8 * 1.299}{(1 + 10.2 * 1.299)^{5/3}} = 0.105$ The aerodynamic admittance functions were obtained using: $R_h = \frac{1}{\eta_h} - \frac{1(1-e^{-2\eta_h})}{2*\eta_h^2} = \frac{1}{3.603} - \frac{1(1-e^{-2*3.602})}{2*3.602^2} = 0.258$ $R_b = \frac{1}{0.7373} - .\frac{(1-e^{-2*0.7373})}{2*0.7373^2} = 1.007$	
Anx. B.3 (2)	where $\eta_h = \frac{4.6 * h * f_L(\mathrm{Z_s n_{1,} x})}{L(z)} = \frac{4.6 * 60 * 1.299}{99.56} = 3.602$	
Anx. B.1 (1)	$\eta_b = \frac{4.6 * b * f_L(\mathrm{Z_s n_{1,} x})}{L(z)} = \frac{4.6 * 12.2 * 1.299}{99.56} = 0.732$ Logarithmic decrement of damping, δ was estimated by: $\delta = \delta_s + \delta_a + \delta_d$ where δ_s= is the logarithmic decrement of structural damping = 0.10 δ_d= is the logarithmic decrement of damping due to special devices = 0 δ_a = is the logarithmic decrement of aerodynamic damping for along wind vibrations, this may be estimated by:	
Anx. B.6 (5)	$\delta_a = \frac{c_f * \rho * b * v_m(z_s)}{2 * n_1 * m_e}$ Where, m_e represents the mass per unit length over the upper third of the structure; i.e.	
Anx. B.2 (2) EC 1, 2005	$H/3 = 60/3 = 20m$ 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 $m_e = 4863.86/3 = 231.612$ kg/m ,with $c_f = 0.748$ then $\delta_a = \frac{1.36 * 1.25 * 12.2 * 58.75}{2 * 0.767 * 231.61} = 1.887$ Therefore, $\delta = \delta_s + \delta_a + \delta_d = 0.03 + 3.4311 = 1.987$	
Equ.F.2 Anx F EC 1, 2005	Now, the resonance response factor, R $R^2 = \frac{\pi^2 * 0.106 * 0.258 * 1.007}{2 * 3.461} = 0.072$ The peak factor, K_p $K_b = (\sqrt{2\ \ln v * T}) + \frac{0.6}{(\sqrt{2\ \ln v * T})} or K_b = 3\ whicheverislarger$ Where	

REFERENCE	CALCULATIONS	OUTPUT
	The up-crossing frequency, v $v = \frac{n_{1,} * \sqrt{R^2}}{\sqrt{B^2+R^2}} = \frac{0.767 * \sqrt{(0.395)}}{\sqrt{(0.218^2+0.072^2)}} = 0.381\,, \quad v \geq 0.08\,Hz$ The limit of $v \geq$ 0,08 Hz corresponds to a peak factor of 3.0. the averaging time for the mean wind velocity, T= 600 seconds Hence, $K_b = \left(\sqrt{(2*\ln(0.381*600))}\right) + \frac{0.6}{\left(\sqrt{(2*\ln(0.381*600))}\right)} = 3.323$ Thus, the structural factor, $c_a c_d$ for the fundamental wind velocity of 47 m/s is : $c_a c_d = \frac{1 + (2*3.323*0.152*\sqrt{0.218^2+0.072^2}}{1+(7*0.152)} = 1.1$ 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	δ_s= 0.10 δ_d= 0 No damping
Anx. B.2 (6)	Therefore, Wind load, w $\mathrm{w} = \mathrm{q_p(z)} c_a \mathrm{c_d}(\mathrm{c_{p,wind}} + \mathrm{c_{p,lee}}) * \mathrm{A}$ 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.	δ_a =1.887 R^2 =0.072

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

Story	Elevation (m)	ETABS X (kN)	ETABS Y (kN)	Manual Y-Direction
Story20	60	74.945	332.7557	329.8128
Story19	57	148.7897	660.6264	652.4697
Story18	54	147.0774	653.0237	644.9643
Story17	51	145.2767	645.0285	637.0718
Story16	48	143.3774	636.5956	628.7478
Story15	45	141.3675	627.6718	619.9398
Story14	42	139.2326	618.1928	610.5847
Story13	39	136.9552	608.081	600.6060
Story12	36	134.5137	597.2406	589.9097
Story11	33	131.881	585.5517	578.3781
Story10	30	129.0229	572.8616	565.8614
Story9	27	125.8943	558.9709	552.1645
Story8	24	122.4352	543.6124	537.0262
Story7	21	118.562	526.4154	520.0853
Story6	18	114.154	506.8437	500.8213
Story5	15	109.0261	484.076	478.4420
Story4	12	102.8723	456.753	451.6492
Story3	9	95.1238	422.3498	418.0777
Story2	6	84.4957	375.1608	372.6170
Story1	3	68.4917	304.1032	323.4754
Base	0	0	0	0

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 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).

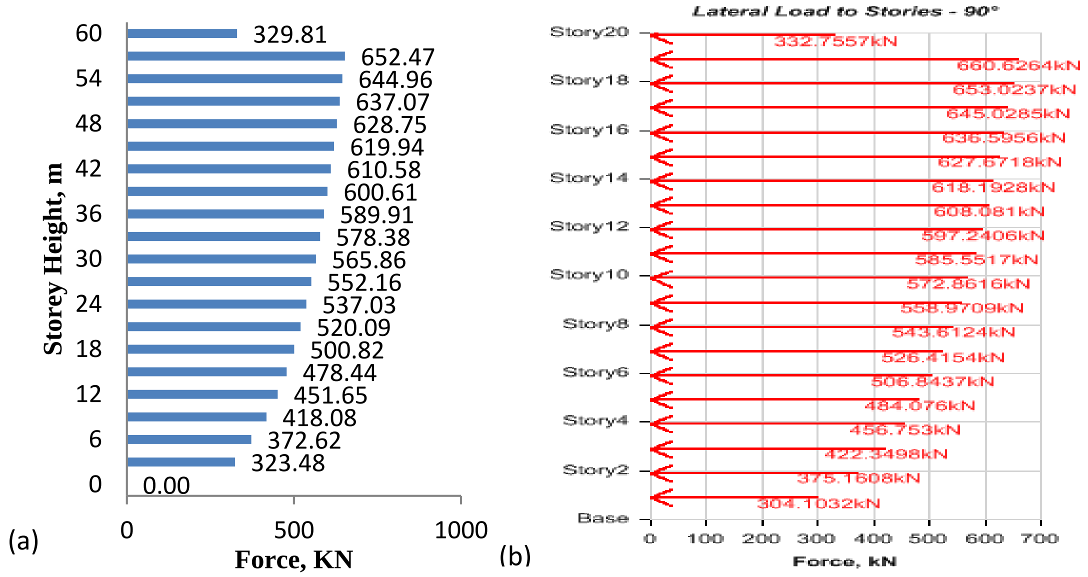


Figure 5. Wind Forces result Profile at different storey-heights (a) Manual (b) ETABS

Table 5. t-test statistical analysis

	Manual	*ETABS*
Mean	510.2816	505.3669
Variance	24846.61	23734.16
Observations	21	21
Pearson Correlation	0.999532	
Hypothesized Mean Difference	0	
df	20	
t Stat	3.781704	
P(T<=t) one-tail	0.000586	
t Critical one-tail	1.724718	
P(T<=t) two-tail	0.001171	
t Critical two-tail	2.085963	

Turbulence Intensity, Iv was derived from equation $I_v = \frac{k_l}{c_o(z)\ln(\frac{z}{z_0})}$ 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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