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Full Length Research Paper

The appropriate model geometrical characteristics suitable for use in an Eiffel type subsonic boundary layer wind tunnel

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The study of parameters selection of model sizing appropriate for use in an Eiffel type subsonic boundary layer wind tunnel was investigated. Using a prototype-model dimension scale of 1:300 on a 72 m, 20 storey prototype building with ratio 1:2:6; a width of 43.33, 83.33 mm breadth and 240 mm height was produced as physical model. The result of the application of methods of global modular ratio and area moment for parameters selection shows that the moments of inertia obtained from the two methods only differ by 2.33%. The proposed hollow rectangular configuration recommended as walls for the physical model are 10.5 and 20.2 mm thick along the width and breadth, respectively. The assemblage of the models was rigidly bounded together by using a set of G-clamping devices and special adhesives such as epoxy resin and liquid glue. The base of the physical model was mounted on flat steel plate which serves as an infinitely rigid foundation resting on the polystyrene that represents the soil conditions which was assumed to behave as elastic base. At the end, the model was rigidly fastened to the wind tunnel using solenoid wires. The original data generated for the dimensions were intrinsically linear and they were linearized using a natural logarithm model. Ordinary least square regression analysis conducted on the result show that y = 1.94 + 0.433x. The analysis of variance (ANOVA) is significant at P = 0.001 but the corresponding standard deviation, correlation coefficients and student's T are S = 0.1561, R - Sq = 93.9% and T = 9.23, respectively. All these statistically confirm that there are no serious trends to show that the proposed models are inappropriate.

Key words: Model sizing, strouhal number, Bernoulli universal constant, wind tunnels, model scaling, multi-storey, building, aerodynamics loadings.

INTRODUCTION

To develop parameters for governing equations and geometrical characteristics of a physical model suitable for capturing the physico-mechanical and the ambient environmental conditions of a multi-storey building to be investigated in a boundary layer wind tunnel; the common approach is to consider the mean and fluctuating components separately when assessing their effects on such buildings and other structures. In recent years, more attention has been given to studying the characteristics of the fluctuating wind force component which causes structural vibrations. This has become more evident considering the difficulties and physical effects that unsteady measurements have on the damping and frequencies response of flexible tall structures and multi-storey buildings. The use of sophisticated mathematical theories involving probability and statistical methods have been proposed for predicting the dynamic response of a structure and some of these have been incorporated into codes of practice on wind effects. It is these rationally developed theories which provide general and sound

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background knowledge in this field (Lam and Lam, 1981). Current and most recent contemporary researches in this area are focused on more accurate determination of various parameters to which these theories are being applied. These rigorous analytical results are now being compared with results obtained from wind tunnel test. In some countries, empirical results are developed from the hybrid values of results obtained from the analytical and tests conducted in wind tunnel.

According to Bergmann et al. (2003) although, the mean and fluctuating wind load components are commonly considered separately and the total structural response is obtained by super-position. It is clear that the fluctuating component is not entirely independent of the mean. This is due to the variable effectiveness of different sizes of gust, which have been shown to change with the mean wind speed. The properties of the mean wind load would seem to merit more attention on account of the influence of mean wind load on the time-average and time-dependent structural response components.

In the absence of comprehensive full-scale tests results from the wind tunnel, the mean wind load is generally determined by adopting force coefficients or shape factors obtained from wind tunnels. Thus, whenever possible, full-scale measurements of the wind effects on buildings and structures should be accompanied by wind tunnel experiments for comparison. It is only through such comparative studies that accurate modelling can be assured (Lam and Lam, 1981).

Halabian et al. (2003) in their work on “reliability analysis of wind response of flexibly supported tall structures” observed that, proper wind response analysis of a tall building supported on soft soil requires adequate consideration of two aspects: (a) the footing flexibility caused by the deformation of soil around and underneath the footing arising from energy transmission between the structure and soil; and (b) the response of the soil-structure system to wind loads as dynamic loading. The former, which is called soil–structure interaction (SSI), has a significant effect on the behaviour of structures such as free-standing towers and is not present in the case of a structure on rock. The SSI allows for energy dissipation through radiation of waves into the soil medium.

There are two approaches to account for SSI in the dynamic analysis of structures. In the direct approach, the stiffness of the global system that includes the structure, foundation and the supporting medium is assembled and the response is obtained in one step. Alternatively, the substructure method can be used in which the global system is subdivided into two subsystems: superstructure and substructure. The dynamic analysis for the superstructure is performed using the impedance functions of the substructure. Both approaches use the soil dynamic parameters such as soil shear wave velocity to characterize the soil flexibility. In low and moderate level seismological zones, the design of tall structures is governed by wind loading. Dynamic wind forces (drag loads) on flexible structures depend on the induced structural response and thus are sensitive to the dynamic characteristics of the structure, which are largely influenced by the model flexural rigidity and soil–structure interaction phenomenon (Halabian et al., 2003).

Kumar (2009) in his work “a comparison between analytical methods in international building codes and wind tunnel testing for the purpose of effective tall building design” noted that, wind tunnels are widely used to reliably predict the wind loading on the cladding and glazing as well as on the structural frames of tall buildings. He concluded that, Code analytical provisions typically, but not always, give wind loads that are higher than what the building will really experience in the storm of design intensity. He however remarked that, it is hearth warming that most recent methods also allow and, in fact, recommend wind tunnel tests be undertaken where more accurate wind loads are desired. He also noted that, it is not surprising that the loads derived are often conservative and most often defined as minimum design loads. Therefore, loads derived from Code analytical methods represent an upper envelope covering the majority of cases for standard building shapes. Unlike wind tunnel tests, Codes have difficulty accounting for project specific factors such as:

- The aerodynamic effect of the actual shape of the structure;
- The influence of adjacent buildings and topography;
- Detailed wind directionality effects;
- Aero-elastic interaction between the structural motion and airflow.

However, wind tunnel studies take all of these issues into account.

Similarly, Kijewski and Kareem (2001) in their work on “Dynamic Wind Effects: A Comparative Study of Provisions in Codes and Standards with Wind Tunnel Data” considered nine (9) building cross sections representing a host of typical building shapes. Three different rigid balsa wood models were constructed each from the nine (9) building cross-sections, with heights of 16, 20 and 24 inches, yielding 27 different model buildings of varying aspect ratio and shape, providing some indication of the influence of these factors on aerodynamic loads (Kareem, 1992). The light-weight models were then affixed to an ultra-sensitive force balance and subjected to wind tunnel testing. These authors showed that, high-frequency force balance may be used for determining the dynamic wind-induced structural loads from scale models of buildings and structures.

They concluded that, this techniques have dramatically reduced both the time and cost required to obtain estimates of wind loads and structural response levels. The force balance provides dynamic load information for a specific building geometry and setting which may be used to calculate loads and response levels for a wide range of structural characteristics, damping values, and building masses. They also observed that the force balance technique has some shortcomings, e.g. only approximate estimates of the mode-generalized torsional moments are obtained and the lateral loads may be inaccurate if the sway mode shapes of the structure deviate significantly from a linear mode shape. The aerodynamic loads on buildings may be obtained by mapping and synthesizing the random pressure fields acting on the building envelope. The structure of random pressure fields through simultaneously monitored multiple-point realizations of pressure fluctuations and measurement of the local averages of the space-time random pressure fields by means of spatial and temporal averaging techniques can be mapped (Kareem, 1989).

Kareem (1990) in another work on “measurements of pressure and force fields on building models in simulated atmospheric flows” also observed that, the localized point-to-point pressure fluctuations are important for the design of cladding and their attachments, but they do not provide directly useful information concerning the integral loads. In this case, the statistics of the local spatial averages of the random pressure field become more relevant. The overall loads are synthesized through space-time structure of the local averages of the pressure field which takes into account the lack of spatial and or temporal correlation (Kareem, 1989). The space-time averaging may be accomplished by a pneumatic averaging technique which through a pneumatic manifolding procedure determines time varying local averages of aerodynamic loads. Alternatively, a sensitive multi-component force transducer may be utilized to measure base shears or moments that can be related to the mode-generalized load spectra associated with the linear or uniform mode shape (Kareem, 1987). However, these approaches fail to provide information on the spatial distribution of aerodynamic loads acting over the surface of the building model under study. A second-generation multi-level, multi-component force balance may provide distribution and correlation of dynamic wind loads (Reinhold and Kareem, 1986).Therefore, researches into spatial distribution of aerodynamic loads acting over the surface of the building has been very popular in dynamic analysis of structures as demonstrated by investigations conducted by other international scholars among which are (Kareem and Cermak, 1979; Tschanz and Davenport, 1983; Reinhold and Kareem, 1986; Boggs and Peterka, 1989).

Description of the wind tunnel and the material for the model

In Nigeria, available aerodynamics teaching and research equipment are limited to the educational wind tunnels. The quality and features of the wind tunnels make them ideal for teaching and research. The comprehensive subsonic wind tunnel is without a rotating disc. It is a compact, realistic and high-quality wind tunnel with a wide range of standard instrumentation and models such aero float simulation but none of the models are appropriate for simulation of multi-storey buildings. The wind tunnel is mostly used for student project work and research purposes in mechanical engineering fluid related simulations. The wind tunnel has the disadvantage that, it is small, since it has a test chamber capacity of 300 mm in height, 450 mm width and 600 mm breadth, respectively. It may achieve air velocities with Mach 0.23 which is the incompressible limit and capable of transonic tests, in a single test section. The air velocity may be set to any Mach number in seconds using a simple manual control.

Wind tunnels are either an open-circuit or closed circuit type with a working section and a working length. Because it does not have a rotating disc and the test chamber has a limit height, it therefore possess a challenge that appropriate analytical methods must be employed to achieve adequate and effective model scales and sizes (that is prototype - model dimension, site- tunnel wind speeds and prototype-model frequencies etc.).

It is all these challenges and limitations that provoked the research for the development of parameters for equations and geometrical characteristics of a physical model suitable for capturing the physico-mechanical and the ambient conditions of the environment of a multi-storey building to be investigated in an Eiffel type subsonic boundary layer wind tunnel. This present study is therefore, a comparative study of the dimensional selection for the model needed for the determination of global deformation, load distribution on a full-scale building and a wind tunnel tested model. Experiments have shown that, wind pressure distribution on the bluff building can be adequately represented by the result of the model studies in an Eiffel-Type boundary layer wind tunnel with elegant turbulence simulation. The current research used the boundary layer wind tunnel for a more accurate prediction of various parameters (e.g. model dimension sizing, model deflection etc.) to which the theories of fluid computational dynamics or dimensional analysis are being applied. The Afara wood was used for the construction of the superstructure. Afara is relatively easy to machine to the desired shapes and sizes. The Afara wood was used for the construction of the model to ensure that a low mass is produced. The low mass of the model is necessary to ensure that the natural frequency of the model-balance system is well above any expected wind forcing frequency. A primary advantage of this approach is that, modal force spectra are obtained directly and can be used in subsequent analytical estimations of building response, if the geometrical dimensions do not differ significantly.

Wind tunnel scaling

According to Mendis, et al. (2007), aerodynamically bluff cross-sections shed vortices are frequencies governed by the non-dimensional Strouhal number, St.

$$St = \frac{f_s B}{U} \quad (1)$$

Where:

f_s = the shedding frequency (in Hz). The shedding of vortices generates a periodic variation in the pressure over the surface of the structure.
B = the least plan dimension (breadth) (that is for the along forces; the plan dimension parallel or along the directrix of the pulsating wind gust) and
U = fluid (wind) velocity.

According to Zhou and Kareem (2003), wind tunnel testing is a powerful tool that allows engineers to determine the nature and intensity of wind forces acting on complex structures. To achieve this, the following steps are necessary.

i) Determine the dimension or length scale and the model blockage, the turbulence length scale, and constraints on model construction, e.g., the working space and the mass requirement. Once selected, the model as well as the ambient structures and their locations need to be built in the same scale.
ii) After the selection of length scale, the wind velocity scale and the frequency scale can be intentionally adjusted to fit the capability of the tunnel facilities. The air density scale is fixed and usually very close to unity.
iii) Special attention needs to be paid when there is a significant difference between the model and the prototype in temperature or elevation.
iv) This method is particularly useful when the complexity of the structure and the surrounding terrain, resulting in complex wind flows, does not allow the determination of wind forces using simplified code provisions.
v) Wind tunnel testing involves blowing air on the building model under consideration and its surroundings at various angles relative to the building orientation representing the wind directions.
vi) This is typically achieved by placing the complete model on a rotating platform within the wind tunnel. Once testing is completed for a selected direction, the platform is simply rotated by a chosen increment to represent a new wind direction.

According to Joseph and William (2006), the prototype-scale sampling frequency associated with the pressure time series follows from the requirement of equivalence between the reduced frequencies at physical model scale and at prototype scale, which can be expressed as Equation (2):

$$\left(\frac{f_s B}{U}\right)_m = \left(\frac{f_s B}{U}\right)_p \quad (2)$$

Where B, denotes a characteristic dimension of the structure;f_s, denotes the sampling frequency for the mean wind velocity at a consistent height (e.g., roof height), and the subscripts m and p denote “physical model” and “prototype” scales, respectively. Letting

$$\lambda L = \frac{H_m}{H_p} \quad (3)$$

λL , denote the length scale of the wind tunnel model. H_m and H_p are physical model and prototype maximum heights. The prototype sampling frequency can be expressed as follows by rearranging.

$$f_p = \lambda L \left(\frac{U_p}{U_m}\right) f_m \quad (4)$$

The model-scale sampling frequency f_m, the model-scale wind speed $\left(\frac{U_p}{U_m}\right)$, and the length scale λL , are constantly determined by the wind tunnel testing conditions. In contrast, it is generally necessary to consider a range of values of the prototype-scale wind speed U_p, to reflect the statistical variability of the extreme wind speeds from each direction at the site of interest. Because the prototype-scale sampling frequency f_p is proportional to the prototype-scale wind speed U_p, each wind speed that is considered corresponds to a different sampling frequency f_p, this dependence of the sampling frequency on the wind speed is particularly important for dynamically sensitive structures, for which responses can be strongly affected by the frequency content of the loading. For average size tunnels testing tall buildings, the 1:400 or 1:300 scale model of the natural wind is usually generated using “the augmented growth method”. This method generates large-scale turbulence using devices such as trip boards and spires upstream of the fetch length. Carpet or roughness blocks are used along the fetch length to generate the required velocity profile (Joseph and William 2006).

According to Mendis et al. (2007), to model the natural wind successfully, and maintain dynamic similarity between model and full-scale results, the following non-dimensional parameters are kept as near to constant as possible between the natural wind and the wind tunnel. They are:

- The velocity profile $U(z) / U(z_o)$, that is the variation of velocity with height normalised with respect to the values at height z_o,
- The height of the building under investigation;
- The turbulence intensity and
- The normalised power spectral density, which defines the energy present in the turbulence at various frequency.

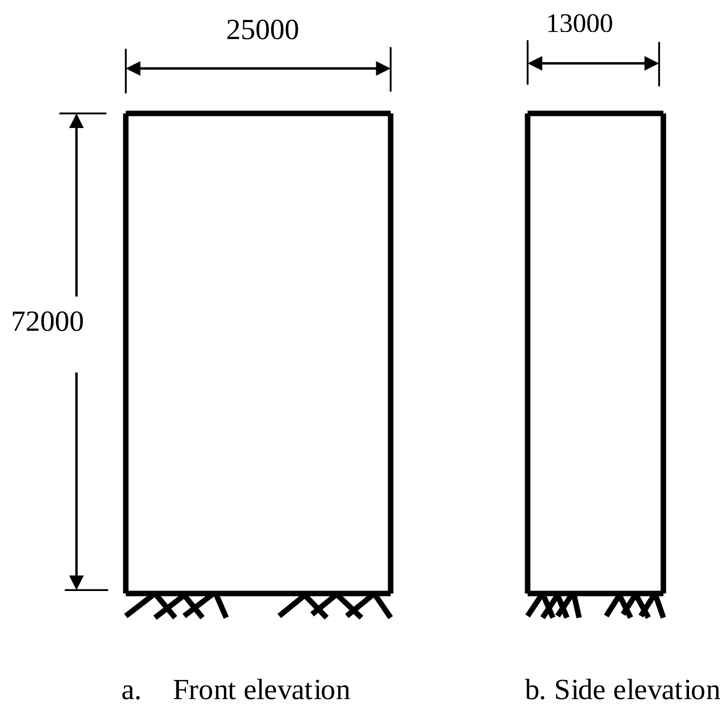

Figure 1. Description of the model. (a) Front elevation and (b) side elevation.

Reynolds number is not an important parameter in this case as a sharp edged model is used.

The design wind speed is based on meteorological data for the given city or area which is analysed to produce the required probability distribution of gust wind speeds. By appropriate integration processes and application of necessary scaling factors, directional wind speeds for the wind tunnel testing can be determined. Therefore, for this study, the Council for Regulation of Engineering in Nigeria (COREN) recently adopted modified wind speed for structural purposes in Nigeria was used (Onundi, 2010).

METHODOLOGY

Description of the prototype multi-storey building

The building under investigation is a proposed 20-storey steel framed structure measuring 13 m by 25 m in plan and 72 m in height. Figures 1a and b are the front and side elevations of the building. It is enclosed at all sides to its full height, with steel-framed glass curtain walls. The partition walls are effectively constructed with light weight stud walls. The proposed building is assumed to be situated on a slightly gentle hilly terrain in an open area after the School of Environmental Design, Tafawa Balewa University, Bauchi in Nigeria where it is exposed to winds blowing from all directions.

Figure 1c, shows the plan and the structural distribution of the load bearing elements for the building. These consist of three (3) primary and six (6) secondary frames which are located along the transverse direction of the building. The three (3) main or primary steel framed shear walls are strategically positioned to resist the vertical live and dead loads within their planes and all horizontal or aerodynamic loads from the pulsating wind gust. They are named Frame number 1 (that is section 1-1) and when viewed from the left of the building, they are located at 6.5, 12.5, and 18.5 m, respectively. Apart from these, the six (6) other secondary frames named Frame number 3 (that is section 2-2), located at 0, 3.5, 9.5, 15.5, 21.5 and 25 m, respectively are mainly positioned to resist all remaining dead and live vertical loads acting on the structural system. An expansion joint is centrally located at 12.5 m. (Frame No 2) are four (4) frames located at 4.8 m (grid B) and 8.2 m (grid C), respectively are also positioned along the longitudinal direction to resist horizontal or aerodynamic loads that might act along that direction.

Since it is conventionally recognized to be more economical to transfer greater loads acting on structural systems through the shorter spans; the three primary frames (Frame number 1) are considered as most critical for analysis. Therefore, the research work is directed towards and limited to the analysis and design of the steel framed shear walls.

Figure 1d, shows a typical cross section of the primary steel framed shear walls and the secondary steel frames (Figure 1e), which show the skeletal frame works of the distribution of the stanchions, beams or girders and the diagonal bracings that all together form an elastically deformable co-planner structural systems. The frames are labelled with horizontal and vertical dimensions. The nodal numbers are also shown to facilitate easy digitization of the local and global coordinates for the generation of the stiffness matrices.

Construction of the physical model

Model materials

The model dimension scale is 1: 300. The materials used for the construction of the model are as follows:

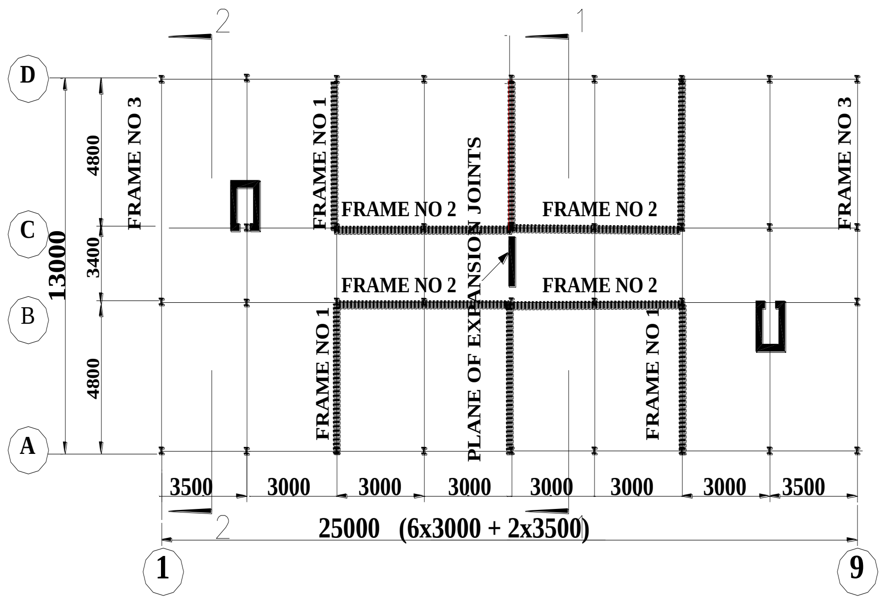

Figure 1c. The plan of the building and the structural layout

i) Afara wood
ii) Mild steel and
iii) Polystyrene

The Afara wood was used for the construction of the superstructure. Afara is relatively easy to machine to the desired shapes and sizes. The Afara wood was used for the construction of the model to ensure that a low mass is produced. As long as the structural geometry does not change, the forces can be used to analyze the effects of internal structural design changes without the need for further wind tunnel tests (Mendis et al. 2007). The mild steel is used for the base plate which represents an infinitely rigid foundation. This rests on the polystyrene and is assumed to behave as soil on elastic foundation. Its impact on the structural integrity of the model also influences the degree of damping incorporated within the system. The physical and mechanical characteristics of these materials are shown in Tables 1 and 2, respectively.

The model geometric dimensions and construction

Two parallel methods were used to determine the geometric dimensions for the physical model. The first method is a proposed approach, while the second is a modified version of a familiar approach but used to validate the correctness or accuracy of the proposed method.

The method of global modular ratio: The principle of dynamic similarity was assumed for the development of the method of global modular ratio. The principle of dynamic similarity relates the ratio of length scale and wavelength when a time scale is referred. When these conditions are satisfied, then, the ratio of a characteristic frequency of the fluctuating flow to a frequency of vibration of an elastic body, or the ratio of a characteristic wavelength of a fluctuating flow to a body dimension will be the same in a wind tunnel and in the atmosphere (Simiu and Scanlan, 1996). Therefore, the ratio between the model moment of inertia (I_m) and that of prototype (I_p) is related; using the global modular ratio (α_g) as follows:

$$\frac{I_m}{I_p} = \alpha_g \qquad (5)$$

$$I_m = I_p\, \alpha_g \qquad (6)$$

$$\alpha_g = 0.0100145\, \frac{E_t\, E_p\, U_m\, H_m}{E_s\, E_s\, U_p\, H_p} \qquad (7)$$

Where:

$a_1 = a_{0.02} = 0.0100145$ corresponding to tunnel for open level country with roughness length ($z_{o1} = 0.02$m)

E_t = Modulus of Elasticity of timber
E_s = Modulus of Elasticity of mild steel (base plate)
E_p = Modulus of Elasticity of polystyrene
U_m =17.5 ms^{-1} = Prevailing wind speed in the wind tunnel (model)
U_p = 52.5 ms^{-1} = Prevailing wind speed in Bauchi (prototype)
H_m = Model height
H_p = Prototype height

$$\frac{E_t}{E_s} = \propto_{ts} = \frac{6310}{205000} = 0.0307804$$
$$\frac{E_p}{E_s} = \propto_{ps} = \frac{3310}{205000} = 0.0161463$$
$$\frac{U_m}{U_p} = \frac{17.5}{52.5} = 0.3333333$$
$$\frac{H_m}{H_p} = \frac{240}{72000} = 0.0033333 \qquad (8)$$

Substituting the values from eqn. (8) into eqn. (7), the global modular ratio is given as:

$$\propto_g = 5.5301 \times 10^{-9}$$

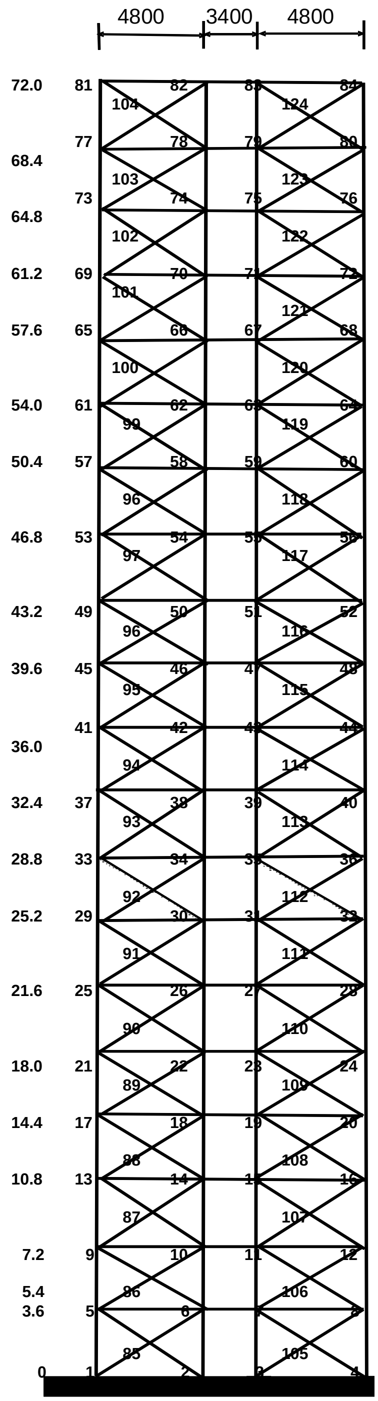

Figure 1d. Cross – Section of the Framed Shear Wall for the Model Showing the local and global coordinates for digitization and Analysis (Frame No 1).

The generalised moment of inertia (I_p) for the substitute cantilever for a framed steel shear wall (that is Prototype) corresponding to the first mode of vibration for a multi-storey building (Onundi et al. 2010).

$$I_p = \frac{m H_p (19409.53 + 3.976 H_p)}{E_p} \qquad 9$$

m_i is the linear modal mass which is given:

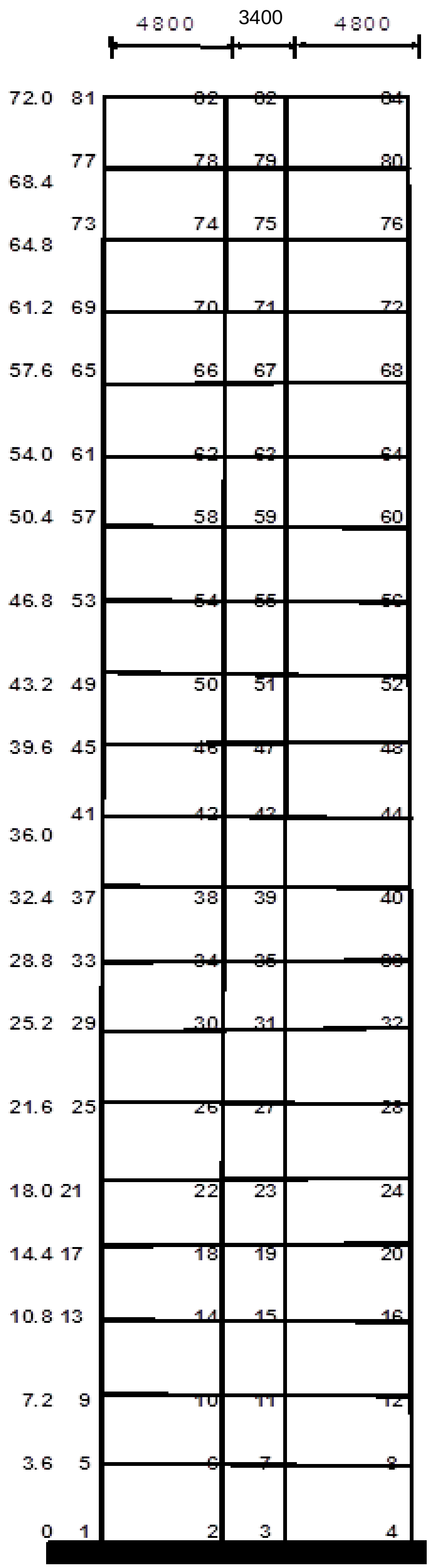

Figure 1e – Cross Section (2-2) of the secondary shear wall for the proto-type model (Frame No 3).

Table 1. Physical and Mechanical properties of Afara timber (Terminalia Superba).

S / No	Property	Values
1	Density (kg /m^3)	464
2	Moisture Content (%)	18
3	Strength Group	N_6
4	Impregnation	Readily
5	Shrinkage	Small
6	Bending and Tension Parallel to grains (N /mm^2)	9.0
7	Compression Parallel to grains (N /mm^2)	7.0
8	Compression Perpendicular to grains (N /mm^2)	1.80
9	Shear Parallel to grains (N /mm^2)	1.12
10	Modulus of Elasticity, E_T , (N /mm^2)	6300
11	Confirmation test for Modulus of Elasticity, (N /mm^2)	6310

Source: Nigerian Standards Organization (1973). "Structural Use of Timber" NCP2, Part 3

Table 2. Characteristics of Polystyrene, Mild Steel Plate and Brazen Tube Materials.

S / No	Material	Property	Values
1	Polystyrene	Tension Strength (N /mm^2)	40 – 60
2	Polystyrene	Elongation (%)	3 – 4
3	Polystyrene	Modulus of Elasticity (N /mm^2)	3000-3600
4	Polystyrene	Modulus of elasticity, confirmation test (N /mm^2)	3310
5	Mild Steel Plate	Density (kg/m^3)	7850
6	Mild Steel Plate	Tension Strength (N/mm^2)	430
7	Mild Steel Plate	Yield Strength (N/mm^2)	220
8	Mild Steel Plate	Modulus of Elasticity (N /mm^2)	205000
9	Mild Steel Plate	Ductility	0.21
10	Mild Steel Plate	Fracture Toughness (Mpa m$^{1/2}$)	140

Sources: Characteristic of Polynomial Materials, Http://en .wikipedia.org/wiki/polystyrene, (2008) and Ashby and Jones, (1998)

$$m_i = \frac{WB\rho H_p}{3 n_{sw} h_{sh}} \qquad 10$$

Multiplying Equation (10) by n_s (number of storeys), we have total modal mass of the building per unit displacement

$$m = n_s m_i = \frac{WB\rho H_p n_s}{3 n_{sw} h_{sh}} = \frac{WB\rho n_s^2}{3 n_{sw}} \qquad 11$$

Where:

H_p = Maximum height of the multi-storey building (that is 72 m)

n_{sw} = Number of shear – walls (that is 3)

n_s = Number of storeys (that is 20)

h_{sh} = H_p / n_s storey height of the shear – wall (that is 3.6 m)

B and W Plan dimensions of size of the building (that is 13 × 25 m)

ρ = Density of the building (0.264 kN/m^3) (Taranath 2005)

E_p = Modulus of Elasticity of Steel Shear-Wall (205 kN/mm^2)

From Eq. (9), prototype generalised moment of inertia (I_p) is

$$I_p = 26.379 m^4 \ (2.6379 \times 10^{13} \ mm^4)$$ and

From Eq. (6), the principle of dynamic similarity and global modular ratio, we obtain the generalised moment of inertia (I_m) as

$$I_m = I_p \, \alpha_g = 145879 \ mm^4$$

The area moment method: According to Allwood et al. (1978), the area-moment method of analysis as used at present is usually attributed to Mohr who published his method of elastic load in 1868. However, it was Professor C.E. Greene, of the University of Michgan who, in 1872, introduced the principles as they are now known. Subsequently, another German, Professor H.F.B. Muller – Breslau, extended the method to highly indeterminate structures. It is conventionally well known that:

$$\frac{M_m}{E_m I_m} = \frac{1}{R_m} \qquad 12$$

Where:

M_m = the physical model bending moment

E_m = the modulus of elasticity of the material (model)

I_m = the physical model moment of inertia

R_m = the radius of curvature of the member (physical model)

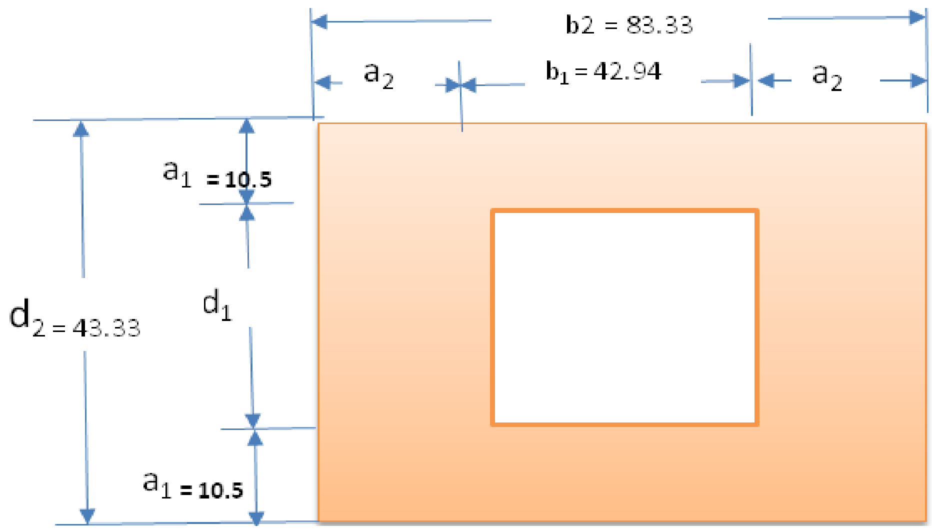

Figure 2. Dimensioning the CROSS-section of the physical model for multi- storey building.

The second theorem of area moment method shows that “the deflection of a point on a straight member under flexure, in the direction perpendicular to the original axis of the member, measured from the tangent at a second point on the same member is equal to the statical (first area) moment of the bending moment diagram, divide by EI, taken about the first point” (Allwood et al. 1978).

The wind pressure is assumed constant from ground level to a height of 10 m and subsequently increases with increase in height. Therefore, the wind pressure diagram is expected to be trapezoidal from 10 m to the model height. The trapezoidal diagram was therefore factor by peak gust time and height coefficients and then sub-divided into a rectangle (17.5 ms^{-1}) and triangle (0 ms^{-1} at 33.33 mm and 8.95 ms^{-1} at 240 mm) and the following formulae apply:

For the rectangular and triangular loadings, the moment of inertia I_m for a cantilevered model is given Eq. (14)

$$I_m = \frac{0.31883\, U_m^{\blacksquare} E_m H_m^3}{E_p \eta_L (\delta_{max})} + \frac{0.017238\, U_m^{\Delta} E_m \left(1333.2 H_m^2 + 55544.4 H_m + 740518.5\right)}{E_p \eta_L (\delta_{max})} \quad (13)$$

Where,

$U_m^{\blacksquare}$ = Rectangular load from wind speed

U_m^{Δ} = Triangular load from wind speed

H_m = Maximum height of the physical model

$\eta_L = \frac{1}{\lambda L} = \frac{H_p}{H_m}$ = 300 (Dimension Scale)

$\eta_u = \frac{U_p}{U_m}$ = 3 (Wind Scale)

$\eta_{dia} = 0.01 =$ Mechanical dial gauge precision

$$\delta_{max} = \eta_{dia}\left(\frac{\text{Dial gauge readings in the wind tunnel x Diamension scale}}{\text{Wind scale}}\right) = 19.68\text{mm} \quad (14)$$

δ_{max} = Maximum deflection measured at the top of the physical model

The approximate solution for rectangular wind loadings is:

$$I_m = 120625 \text{ mm}^4$$

The exact solution for trapezoidal wind loadings is:

$$I_m = 142544 \text{ mm}^4$$

Since the scaling factor for the prototype-model dimensions is 1: 300. The wind speed for Bauchi (prototype) is 52.5 ms^{-1}, (Soboyejo Isopleths Map for Nigeria, 1971 and Onundi 2010). Since the machine cannot produce a wind speed as high as 52.5 ms^{-1}. Therefore, using a site-wind tunnel scale of 1:3, the prevailing wind speed for Bauchi is related to that of the wind tunnel to give a speed of 17.5 ms^{-1} (that is 52.5 / 3). These have resulted in the generalised characteristic moment of inertia of 1.45879 × 10^5 mm^4 for the physical and 2.6379 × 10^{13} mm^4 for the prototype models, respectively.

The model dimensions: Figure 2 shows the cross sectional area of the physical model. The corresponding required generalised moment of inertia (I_m) is related to the cross sectional area by Eqs. (15a, b, c and d), respectively. Since b_2 and d_2 are known using the dimension scale. Therefore, the unknown quantities are a_1, a_2; b_1 and d_1 are shown in Figure 2 and related as follows.

$$I_m = \frac{(b_2 d_2^3 - b_1 d_1^3)}{12} \quad (15a)$$

$$12\, I_m = (b_2 d_2^3 - b_1 d_1^3)$$

$$\text{if,} \quad k = \frac{b_1}{d_1} = \frac{b_2}{d_2} \quad (15b)$$

$$d_1 = 0.5552\left(\sqrt[4]{d_2^4 - \frac{12\, I_m}{k}}\right) \quad (15c)$$

Table 3. Dimensions of the cross-section of the physical model for multi- storey building

Scales		Models					
		Prototype			Physical		
Dimensions	**Wind**	**Height**	**Breadth**	**Width**	**Height**	**Breadth**	**Width**
η_L	η_u	**H_p; mm**	**B_p; mm**	**W_p; mm**	**H_m; mm**	**b_2; mm**	**d_2; mm**
300	3	72000	25000	13000	240	83.333	43.333
Moment of Inertia;	**mm^4**	**k**	**a_1;mm**	**a_2;mm**	**b_1;mm**	**d_1;mm**	
Global Modular Ratio	145879	1.923	10.50	20.20	42.94	22.33	(Average =0.812%)
Area Moment	142544	1.923	10.48	20.16	43.02	22.37	
% Difference	2.330	0.000	0.19	1.98	0.19	0.18	
Linearization model				**Natural Logarithm**			
Global Modular Ratio		2.351	3.006	3.106		3.760	
Area Moment		2.349	3.004	3.108		3.762	
Regression Equation;				y = 1.94 + 0.433x			
Std Dev., Correl. Coeff. and T		S = 0.1561	R-Sq = 93.9%	R-Sq(adj) = 91.5% and T = 9.23			
		Source	**DF**	**SS**	**MS**	**F**	**P**
Analysis of variance (ANOVA)		Regression	2	1.87749	0.93874		
		Error	5	0.12189	0.02438	38.51	0.001
		Total	7	1.99938			

Where; 0.5552 is a height coefficient derived from the influences of height coefficient, wind peak gust time, creep and shrinkage for a prototype model not exceeding 100 m. For a prototype model more than 100 m, the corresponding height coefficient must be re-accessed.

From Figure 2, the unknown quantities a_1, a_2, b_1 and d_1 are determined using the same method. Let us consider as special case when $a_1 = ka_2$ and $b_1 = kd_1$:

$$2a_1 = (d_2 - d_1) \quad \text{and}$$

$$a_1 = \frac{(d_2 - d_1)}{2} \qquad 15d$$

Model construction procedure: The physical model with a ratio of 1:2:6 (width, breadth and height) was constructed. The components parts of the model were machined to the desired shapes and sizes shown in Figure 2. The cross sections of the walls for the models are a_1 = 10.5 mm, d_1 = 22.33 mm, d_2 = 43.33 mm thick along the width and a_2 = 20.15 mm, b_1 = 43 mm and b_2 = 83.33 mm along the breadth respectively. The assembly of the models was fastened together by using a set of G - clamping devices. The hollow rectangular cross-sections of the walls were joined together using special adhesives such as epoxy resin and liquid glue. The base of the physical model was mounted on flat steel plate. This is expected to serve as an infinitely rigid foundation resting on the polystyrene. This arrangement represents the soil conditions and is assumed to behave as elastic base. At the end, this model was fastened to the wind tunnel test chamber using solenoid wires.

RESULTS AND DISCUSSION

Using a model dimension scale of 1:300 on the width 13 m, breadth 25 m and a height of 72 m, 20 storey prototype building; it produced a width of 43.33 mm, 83.33 mm breadth and 240 mm height for the laboratory test. The result of the application of the global modular ratio and the area moment methods for dimensioning of the cross-section for the physical model is presented in Table 3. The moment of inertia for the global modular ratio is 145879 mm^4 while the area moment method gave 142544 mm^4 but they differ by only 2.33%. Therefore, the proposed model (that is the global modular ratio) was considered as rationally and structurally better; because it captured the composite properties of the sub and super structural physico–mechanical geometry and materials along with the ambient characteristics of the environmental conditions of the site. For the hollow rectangular configuration (Figure 2), these values of moment of inertia, gave corresponding sizes of various dimensions with an average of their differences limited to 0.812%.

The original data generated for the dimensions were intrinsically linear and the data was linearized using a natural logarithm model. The corresponding a regression equation of y = 1.94 + 0.433x was obtained when the ordinary least square regression analysis was conducted on these comparative data. Where, y is the predicted variable, 1.94 is intercept or constant, 0.433, the slope and x is the linearized values of the sought dimensions. The analysis of variance (ANOVA) is significant at P = 0.001 but the corresponding standard deviation, correlation coefficients and student's T are S = 0.1561, R-Sq = 93.9%, R - Sq (adj) = 91.5% and T = 9.23, respectively. As can be seen in Table 3, the analysis of

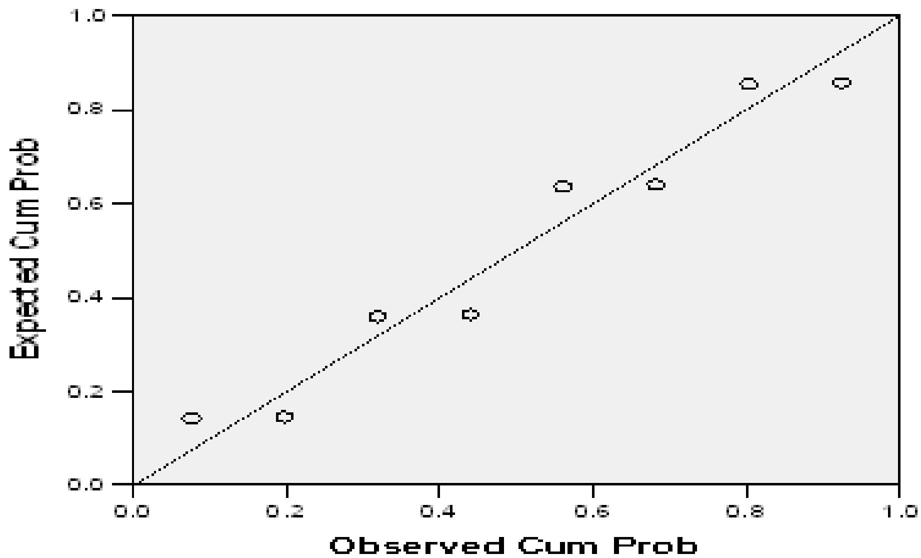

Figure 3. Normal regression standardized residual.

variance (ANOVA) was also tested for lack of fit or test of hypothesis for rejection (that is significance of the ANOVA). The total number (n) degrees of freedom (DF) were 7 but options tested were 2 while the residual error was 5 (that is n - 2). The error sum of squares (SS_e = 0.12189) and total sum of squares (SS_t = 1.99938). The unbiased estimator of variance gave 0.02433. Finally, the ANOVA was therefore significant at P = 0.001 (that is < 0.005). Hence the test for lack of fit is insignificant and can be considered satisfactory. Similarly, in Figure 3, from the normal regression standardized residual, there are few large residuals (Field, 2002; Elinwa and Mohamood, 2002) and hence limited apparent outliers (Rasaq and Wong, 2004). All these confirm that there are no serious trends to show that the models are inappropriate.

The measured deflection was 19.68 mm from the physical model. Therefore, using the principle of simple proportion and similar triangles with a factor of 1.58 (Simiu and Scanlan, 1996) the corresponding prototype deflection was computed from $\boldsymbol{\delta}_{\mathbf{pmax}} = \mathbf{0.0158}\,\boldsymbol{\eta}_{\mathbf{L}}\,\boldsymbol{\delta}_{\mathbf{max}} = \mathbf{0.0158 \times 300 \times 19.68 = 93.2832\ mm}$ or using the tunnel – site wind scale $\boldsymbol{\delta}_{\mathbf{pmax}} = \mathbf{1.58}\,\boldsymbol{\eta}_{\mathbf{u}}\,\boldsymbol{\delta}_{\mathbf{max}} = \mathbf{1.58 \times 3 \times 19.68 = 93.2832\ mm.}$

Relevant results using the constructed model

To test the constructed physical model, the authors considered the volume of analytical work required as more than the scope of this paper; therefore on results of this work to be published are highlighted as follows:

1.) In another paper "Dimensional analysis as wind tunnel experimental data assessment tool for dynamic analysis of a multi-storey building subjected to aerodynamic loadings". Using the same model and data generated (that is wind speeds and model deflection) the following summary of the results were obtained:

a.) As an initial condition, $\mathrm{F_{cqs}} = \mathrm{F_{cdy}}$ is a stationary component of the aero-dynamic force caused by the influence of the kinetic energy of the pulsating wind in collusion with the model (multi-storey building) occurs when the upper limit for the reduced frequency (that is Strouhal Number, $St = \frac{fB}{U}$ = 0.5) is reached. This value corresponds to a reduced frequency of either a low rise building or a high rise building when the wind gust (that is U = U_s = 2fB) has not reached its ultimate (maximum) value. This is important for noting since; if a harmonic load (wind) increases and reaches its maximum value and vanishes in a time less than the generalized fundamental natural period of a structure, the wind load is dynamic, else it is static (Taranath, 2005). The static or stationary component of the aerodynamic loading was defined by the product of the Bernoulli's aerodynamic pressure and net pressure coefficient C_{pi} which is dependent on the structure type and topographical or site terrain, etc., directionality factor C_d, shape factor C_{sh} and the height effect and terrain coefficients C_{ht}. (that is 0.5 ρ C_g U^2 C_{pi} L_o A_s = 0.613 C_{sh} C_d C_{ht} C_{pi} L_o $U^2 A_s$ = C_{gl} Fqs = F_{cqs}. This quantity can be directly measured along the model height from the influence of the wind speed produced in the wind tunnel or directly from the site conditions.

b.) Similarly, with progressive increase in aerodynamic loadings, the dynamic or non-stationary component of the

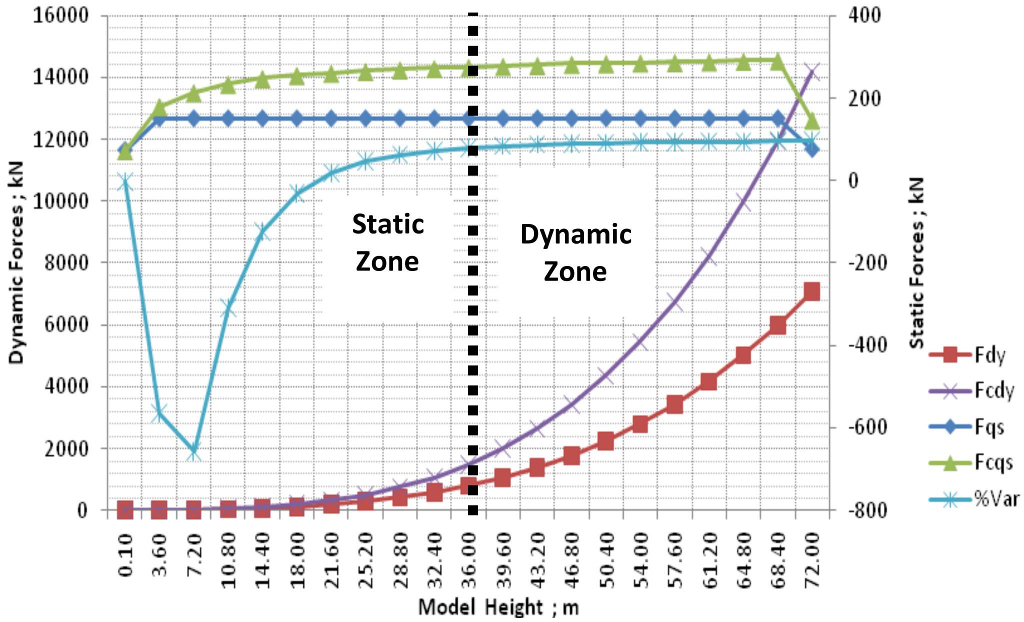

Figure 4. The statistic and dynamic forces along the model.

force was defined as

$$\frac{F_{cqs}}{9.81}\delta_{max}\left(\frac{z}{H}\right)^{\eta} = 0.10194\, C_{fij}F_{cqs} = F_{cdy}$$
.

While η = 2.2932, assisted in the determination of the normalized coefficient of first characteristic mode of vibration given by $C_{fij} = \left(\frac{z}{H}\right)^{2.2932}$ which is defined as the level fractional contribution to the total structural vibration at storey level i and corresponding height j caused by unit acceleration g but z is the variation of the model height at point j. The quantity $C_{fij}F_{qs} = F_{dy}$ is however the dynamic or non-stationary component of the force without the global factor (C_{gl}).

c.) As can be seen in Figure 4, the regions under the static and dynamic forces were clearly defined from the result of a 72m, 20-storey prototype model used as an illustrative example. It was concluded that, an increase in model height lead to decrease in the model frequency. Therefore, at approximately 38 m (0.528 H) from the base of the structure (that is prototype model) forces within the region remains static and above this point they are dynamic. The percentage variation (%Var.) between the static and the dynamic forces is considered to be largely caused by the parameters influencing forces close to the base of tall structures.

2.) In another paper “An Experimental Determination of Damping Ratio of a Multi-Storey Building Subjected to Aero-Dynamic Loadings”.

Figure 5 shows the interactive influence between the model height and Strouhal number (St.) have on the damping ratio (ξ). The study shows that; depending on the phase of the force with respect to motion, self-excited forces can be associated with the displacement, velocity or acceleration of the structure. Due to the influence of these associations, these forces can be thought of as “aerodynamic contributions” to stiffness, damping and mass respectively. Therefore, the building flexural rigidity and deflection are considered to be very important structural quantities and qualities which affect structural phenomena such as forces, moments, velocity and acceleration or human perception criteria of occupants using multi-storey buildings. The damping ratio also proved to have been influenced by dependent variables such as stiffness, amplitudes of vibration, H / B ratio, Strouhal and L_o numbers at a basic wind speed of 52.5 ms^{-1} as elaborately explained in that paper.

Conclusion

Using a prototype-physical model dimension scale of 1:300 on a 72 m, 20 storey prototype building with ratio 1:2:6; a width of 43.33 mm, 83.33 mm breadth and 240 mm height was produced as a physical model. The result of the application of methods of global modular ratio and area moment for parameters selection on the physical model show the moment of inertia obtained from the two methods only differ by 2.33% but the method of modular ratio proved better because it considered both the composite properties of the sub structural and super structural physico – mechanical geometry and materials along with the ambient characteristics of the environmental conditions.

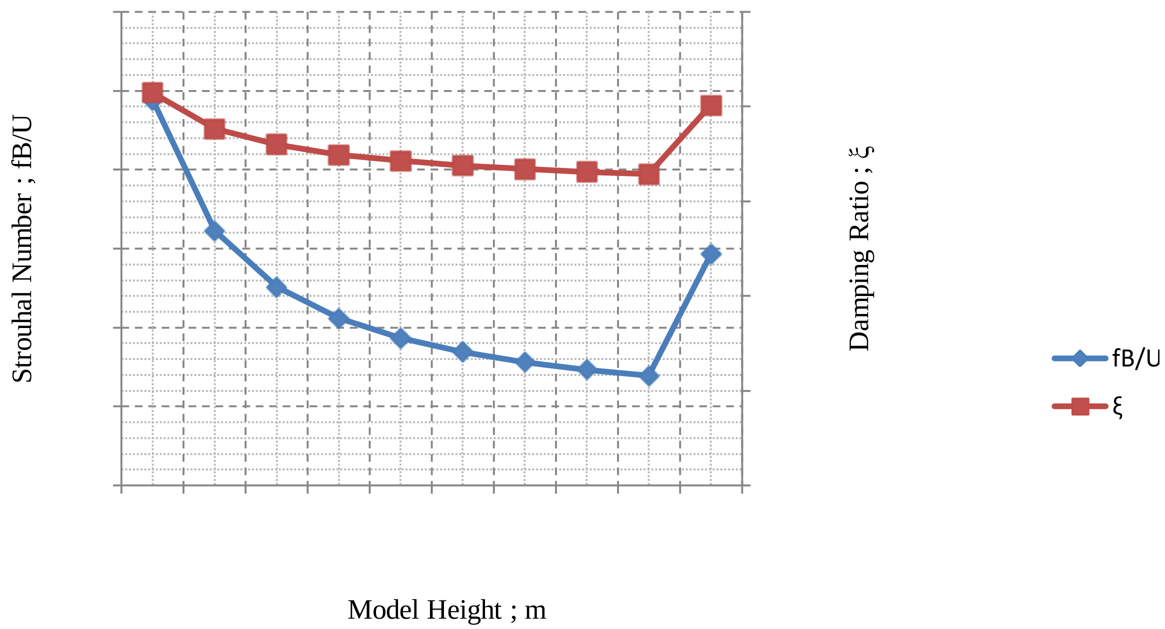

Figure 5. Model height Vs strouhal number and damping ratio.

The recommended walls for the physical model are 10.5 and 20.2mm thick along the width and breadth respectively; but the proposed hollow rectangular configurations of the two formulae gave dimensions that differ with an average of 0.812%.

The assemblage of the models was achieved by binding together two separate configurations using a set of G-clamping devices to form a hollow rectangular cross-section of the walls with the aid of special adhesives such as epoxy resin and liquid glue. The base of the physical model was mounted on flat steel plate which serves as an infinitely rigid foundation resting on the polystyrene that represents the soil conditions assumed to behave as elastic base. At the end, the model was rigidly fastened to the wind tunnel using solenoid wires.

The original data generated for the dimensions were intrinsically linear but the data was linearized using a natural logarithm model. The ordinary least square regression analysis conducted on the result show that $y = 1.94 + 0.433x$. The ANOVA is significant at $P = 0.001$ but the corresponding standard deviation, correlation coefficients and student's T are $S = 0.1561$, R-Sq = 93.9% and $T = 9.23$, respectively. All these statistically confirm that there are no serious trends to show that the proposed models are inappropriate.

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